The response of pile foundations subjected to shield tunnelling

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ABSTRACT: This paper presents a unique case history in Singapore on the study of the effects of shield tunnelling on the adjacent pile foundations. As part of the construction of the MRT North East Line contract 704, a viaduct bridge was planned in conjunction with the tunnel advancement. The bridge which consists of 2 abutments and 39 piers was constructed in parallel alignment with the new twin tunnels configuration. The piers were supported by groups of four to six 1.2 m diameter bored piles. Along the alignment, 6.5 m diameter tunnels were located very closely, i.e. 1.6 m clear distance, to the pile foundation. As part of the monitoring requirement, an instrumentation programme was implemented at six piers. A total of twelve piles were installed with strain gauges at various levels of foundations to monitor the piles' response during tunnelling. The data was then linked to settlement markers installed in the vicinity of the piles. The results show that the piles were subjected to large dragload particularly when higher volume loss was observed. Bending moments developed in piles were found to be much higher in the transverse direction (bending perpendicular) than in the longitudinal direction (bending parallel to tunnel alignment).

1 INTRODUCTION

The demand for transportation underground systems has increased in urban areas such as Singapore (Yong & Pang, 2004), Hong Kong, London and Amsterdam. Space constraint in these areas has made construction of tunnels in close proximity to superstructures inevitable. Extensive research has been carried out in the UK on the effects of tunneling near buildings, particularly from the Jubilee Line Extension (Burland et al., 2002) and North/South Line in Amsterdam (Netzel & Kaalberg, 2003). However, most of the structures observed were supported on shallow foundations and very little data has been collected on the response of pile foundations subjected to tunnelling in close proximity. This is due to the fact that most structures were built long before the tunnels were planned. Lack of instrumentation inside pile foundations led to concerns of the response of pile foundations to tunnelling (Mair, 2003). This paper therefore presents a unique case history in Singapore on monitoring the effects of tunnel construction adjacent to full-scale working piles. As part of the construction of the MRT North East Line contract 704, a piled-viaduct bridge was planned in conjunction with the shield tunnel advancement. Owing to the early planning, in-pile instrumentation was possible and some strain gauges were installed in the piles to monitor the axial force and bending moment that developed during tunnelling. This paper also discusses the monitoring results reported earlier by Coutts & Wang (2000).

2 MRT NORTH EAST LINE C704

Of the 12 civil contracts awarded (C701 to C712), this case history here focuses on C704. This contract involved the construction of two cut and cover stations i.e. Woodleigh Station and Serangoon Station, 992 m twin tunnels from Woodleigh Station to Serangoon Station (Ser-Wdlh) and 1522 m twin tunnels from Serangoon Station to Kovan Station (Ser-Kov). In addition to the tunnels and stations, the contract included the construction of a 1.9 km long dual-lane viaduct bridge. The viaduct bridge which consists of 2 abutments and 39 piers was constructed in parallel alignment with the new twin tunnels configuration and was located in between the tunnels. The bridge piers are supported by pile group of four to six bored piles of 1.2 m and 1.8 m in diameter which were installed to depths up to 62 m below ground level. In this alignment, the 6.5 m diameter tunnels, at about 21 m depth were located very closely, i.e. 1.6 m clear distance to the pile foundation, (Fig. 1). In the project, most of the piles were installed before the tunnels were excavated.

3 GEOLOGY AND GROUND CONDITIONS

The tunnels in C704 were excavated through two different geological formations, namely the Bukit Timah Granite and the Old Alluvium. The major portion of the viaduct bridge was constructed in the Bukit Timah Granite, a type of formation typically found in the central part of Singapore. Being one of the oldest formation in Singapore, Bukit Timah Granite was formed during the early to mid Triassic period, about 200 to 250 million years ago (Leong et al., 2003). Both the tunnels and pile foundations were mainly in completely weathered material (residual soil). The residual soil is classified as Grade VI material according to the British Code of Practice (BS5930, 1981) or better known as a G4 material according to the classification system by Dames & Moore (1983). G4 materials are predominantly reddish brown, sandy, silty clay.

4 CONSTRUCTION SEQUENCE

In C704, construction activities have been coordinated in such a way that the bridge viaduct construction does not interfere with the tunnel advancement or affect the tunnel lining. Therefore, piles had to be installed prior to the tunnel advancement. During the advancement, various levels of casting for the viaduct bridge were in progress at different piers. The piles were either partially loaded or not loaded at all. Two earth pressure balance machines (EPBM) were used to bore the South bound (SB) and North bound (NB) tunnels. Both EPBMs were launched from Serangoon Station and advanced towards Woodleigh Station. The tunnelling work started in April 1999. Upon breakthrough at Woodleigh Station, the EPBMs were transferred back to Serangoon Station and continued their advance towards Kovan Station. The SB tunnel was driven first, followed by the NB tunnel which was approximately 200 m to 300 m behind the SB tunnel. In general, the SB and NB tunnels were located at almost the same level along the alignment, with their depths ranging from 16 m to 29 m below ground surface.

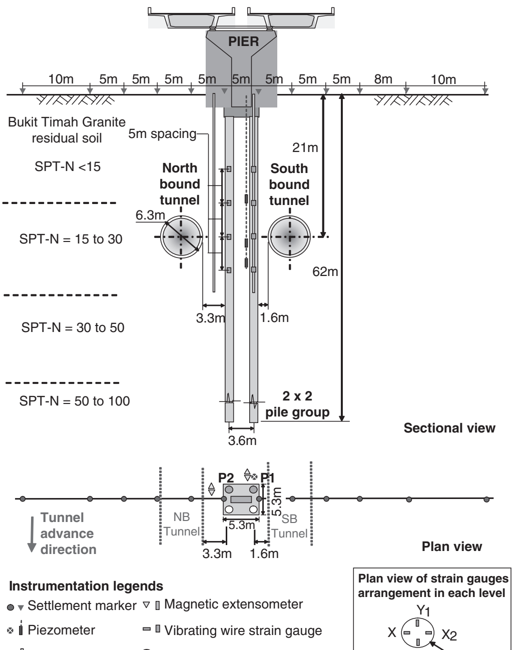

Figure 1. Typical section and instrumentation layout.

5 FIELD MONITORING

Extensive instrumentation was installed on the surrounding ground along the tunnel route to monitor the performance of tunnel advancement and to verify the design. More than 800 settlement and building markers, 48 inclinometers, 12 deep extensometers, 10 tiltmeters, 26 tape extensometers and 55 piezometers/standpipes were installed along the tunnel route (Knight-Hassell & Tan, 2000). One of the most noticeable instrumentation was the in-pile strain gauges installed to monitor the pile response during tunnel advancement.

As part of the monitoring requirement, instrumentation programmes were implemented at six piers (Pier 11, 14, 20, 32, 37 and 38). A total of twelve piles (with two piles at each pier) were installed with strain gauges at various levels. The strain gauges were located at four different levels with 5 m spacing in between. They correspond approximately to the tunnel springline, invert, crown and 5 m above the crown. Four strain gauges in pairs of two, were installed on four sides at each level with one pair located parallel to and the other pair located perpendicular to the tunnel direction. The strain gauges were used to calculate both the bending moment (i.e. transversely and longitudinally) and axial force in the piles.

Additional in-ground instrumentation such as settlement markers, inclinometers, magnetic extensometers and piezometers were installed in the vicinity of the piles. Figure 1 shows the typical instrumentation layout at one of the instrumented piers. Similar instrumentation arrangement was also implemented at other piers.

In this paper, only the monitoring results of ground movement and piles subjected to single tunnel advancement (i.e. SB tunnel) are presented due to space constraint.

6 MONITORING RESULTS

6.1 *Ground surface settlement*

The surface settlement measured at the Pier 20 during SB tunnel advancement is shown in Figure 2. The measured data has approximated reasonably well to Gaussian curve. The assumed trough width parameter (K) of 0.5 was found to adequately represent the soil. The immediate measured settlement trough corresponds to a volume loss of 1.38%. Further comparison of the tunnel advancing records will allow the data regarding the contribution from each phase of the tunnelling process to be obtained. Approximately 49% out of the total volume loss was due to face loss, followed by 28% attributed to shield loss and 23% due to tail void closure. Face loss seems to be the major contribution towards the surface settlement in this section.

Similarly to Pier 20, the volume loss is computed from the surface settlement markers at the other instrumented piers locations. The volume loss ranged from as low as 0.32% to as high as 1.45%.

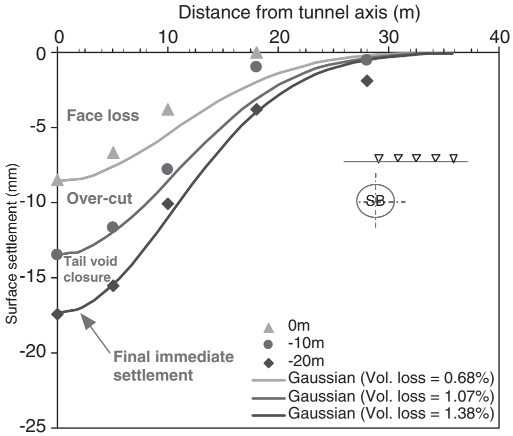

Figure 2. Surface settlement trough measured at Pier 20 array.

6.2 *Axial force in piles*

In general, soil settlement caused by tunnelling (i.e. volume loss) would induce additional axial force i.e. drag load in the nearby piles. As the soil settles, negative skin friction acts on the piles and therefore leads to a drag load. The response is evident from the measured strain gauges data. Figure 3 shows a typical development of axial forces in the front pile (P1) and rear pile (P2) with distance of tunnel approaching and leaving Pier 20. The negative (−ve) value of the axial force indicates compressive force (dragload). For a clearer visualisation, only the maximum measured axial force at one level of the strain gauges is presented. The maximum axial force in the pile was measured either at the same level of the tunnel springline or invert. As can be observed, the piles were not initially subjected to working load (i.e. casting reached pilecap level during tunnel advancement). The dragload only started to increase when the EPBM was at a distance of 25 m approaching the piles. Thereafter, the dragload increased gradually. When the EPBM face was adjacent to the centre of the pile group, whereas pile P1 recorded a dragload of 3000 kN where as pile P2 recorded 2000 kN. The dragload continued to increase even after the EPBM left the piles. Peak dragload was

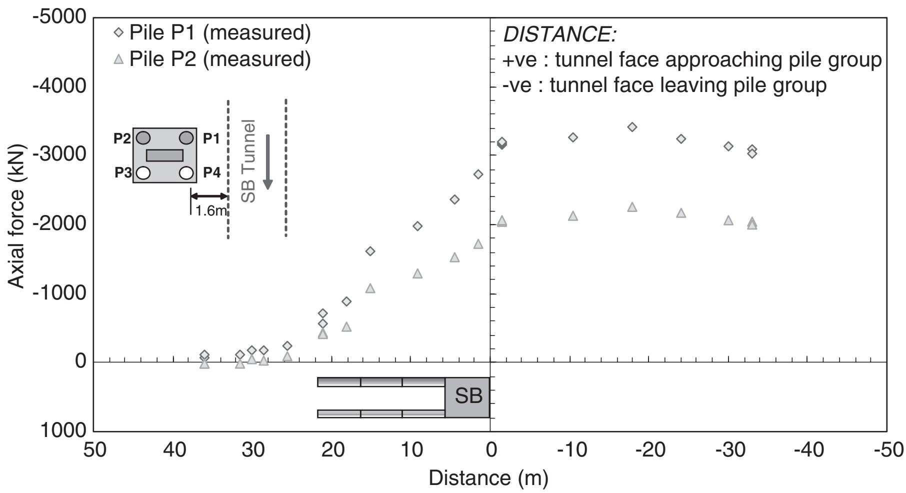

Figure 3. Development of maximum axial force in piles at Pier 20 with tunnel advancement distance.

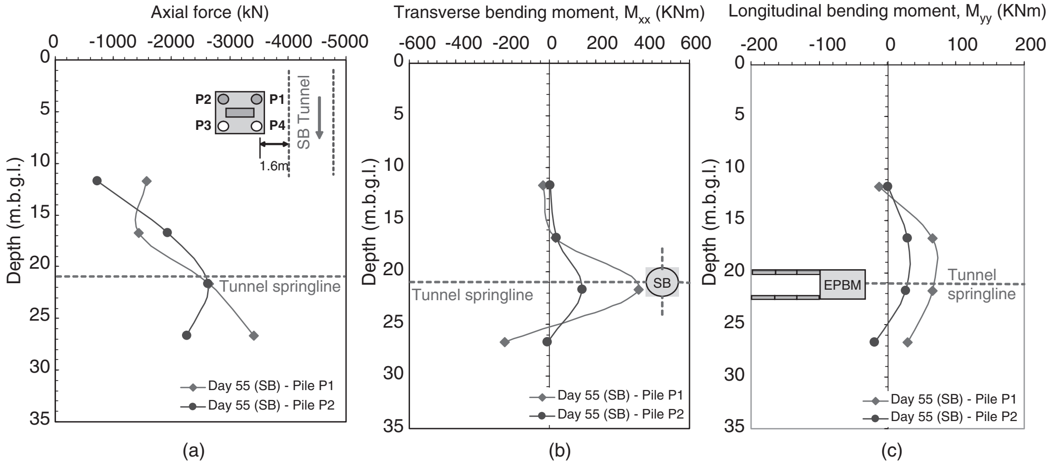

Figure 4. Response of pile foundation at Pier 20 (a) axial force (b) transverse bending moment (c) longitudinal bending moment.

recorded when the EPBM was approximately 20 m away from the piles.

Figure 4(a) plots the maximum measured axial force against depth in piles P1 and P2 when subjected to SB tunnel advancement. The data shows that the axial force increased with depth up to the tunnel springline or the invert level. With a limited number of strain gauges located below the tunnel springline, a reduction of axial forces was not very apparent below the tunnel springline. Furthermore, it can be observed that pile P1 experienced a higher dragload of 3400 kN compared to the dragload of pile P2 which was 2600 kN after the SB tunnel advancement. This result was expected since the distance of pile P2 is further away from the SB tunnel; the further the distance is from the tunnel axis, the more soil settlement decreases. Another likely cause for the lower axial force in pile P2 is the interaction of piles within the pile group.

Most of the instrumented piles indicated a profile similar to the piles at Pier 20. Nevertheless, piles at Pier 11, 32 and 38 recorded tensile force particularly at the highest level of the strain gauges. However, it is not conclusive since the strain gauges located further above the piles were insufficient, which could otherwise have recorded a higher tensile force. The measurement shows that the bending effect could lead piles to both compression and tension within the same pile group.

As was noticed in almost all the instrumented piles, the dragload reached the peak when the EPBM was adjacent to the piles. As the EPBM moved further away from the piles, the dragload either remained the same (i.e. Pier 20) or reduced. The reduction of dragload is more drastic in piles at Pier 11, 14, 37 and 38. Due to space constraint, this issue will not be addressed in this paper.

The maximum dragload for all the instrumented piles were compared with their structural capacity. Allowable structural capacity of 11,000 kN and 25,000 kN were calculated respectively for 1.2 m and 1.8 m diameter piles assuming concrete strength of $f_{cu} = 40\ N/mm^2$. In summary, the maximum dragload due to SB tunnel advancement ranged from 9% to 46% of the structural capacity. It should be noted that the value is due to SB tunnel advancement only and further increase which will occur after NB tunnel advancement will be more significant (not discussed in this paper).

6.3 *Transverse bending moment in piles*

Bending moment was induced in the pile when the EPBM was driven in close proximity due to horizontal soil movement. The piles could be subjected to bending moment in two directions namely the longitudinal bending moment (M_{yy}) and transverse bending moment (M_{xx}). The two bending moments represent respectively the bending parallel and perpendicular to the tunnel alignment. Figure 4(b) shows a typical plot of the measured transverse bending moment against depth in piles at Pier 20. Generally, the maximum bending moment was recorded at the tunnel springline level. During the advancement of SB tunnel, maximum M_{xx} of 403 kNm and 163 kNm were observed in the front pile (P1) and rear pile (P2) respectively. The difference of almost three times was apparent in the piles of the same pile group. Similarly to what was observed in the axial force, the difference is likely to be due to the pile-soil-pile interaction and also the shadowing effect from the front pile. In addition, since the rear pile was 3.6 m (3 times pile diameter) behind the front pile, a smaller lateral soil movement would be expected at the rear pile, which would cause a smaller bending. Due to the limited level of strain gauges installed, investigation of the continuous bending profile along the whole pile length was not possible.

6.4 *Longitudinal bending moment in piles*

Figure 4(c) shows a typical plot of the measured longitudinal bending moment against depth in piles at Pier 20. As can be noted, the bending was towards the tunnel advancing direction. The front pile (P1) was again subjected to a higher bending compared to the rear pile (P2) during the advancement of SB tunnel. Maximum measured M_{yy} at Pier 20 was 131 kNm and 49 kNm respectively for P1 and P2 on Day 50 (not shown in Fig. 4c).

7 RELATIONSHIP BETWEEN SOIL MOVEMENT AND PILE RESPONSE

7.1 *Axial force vs. volume loss*

In order to examine the effect of soil settlement (or in general the volume loss) on the development of axial force in piles, the corresponding reading from strain gauges and surface settlement markers (converted to volume loss) from the same day were compared. Figure 5 plots a typical relationship between axial force (dragload) and volume loss for piles P1 and P2 at Pier 20. In general, the axial force increased as the volume loss increased. A volume loss of up to 1.38% was attributed to the advancement of SB tunnel. It is noted that when volume loss exceeded 0.9%, the axial force increased very little.

Figure 6 plots the maximum measured axial force against the volume loss for all the 1.2 m diameter instrumented piles when subjected to SB tunnel advancement. Only the results of 1.2 m diameter piles are presented. As discussed above, the soil is subjected to varying volume losses at different sections along the tunnelling stretch. There is an apparent linear relationship between the volume loss and axial force. It can be noted that as the volume loss increases, the axial force in the piles also increases. Most of the front piles show a higher axial force compared to the rear piles.

7.2 *Bending moment vs. volume loss*

Figure 7 plots the maximum developed transverse bending moment against the corresponding volume loss during the SB tunnel advancement. As can be observed, the bending moment has a tendency to increase as the volume loss increases. The bending

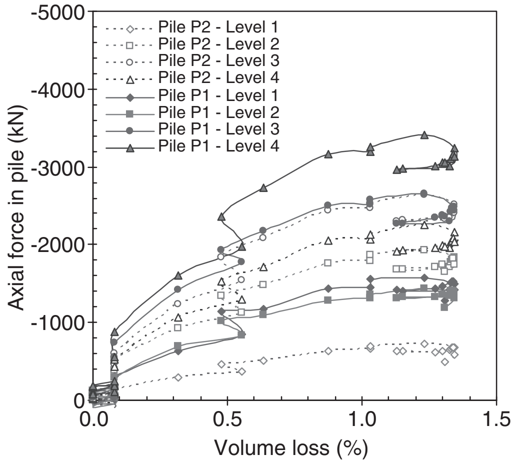

Figure 5. Relationship between volume loss and axial force developed for Pier 20.

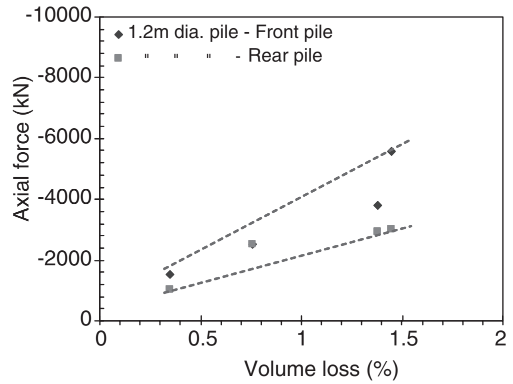

Figure 6. Relationship between axial force and volume loss.

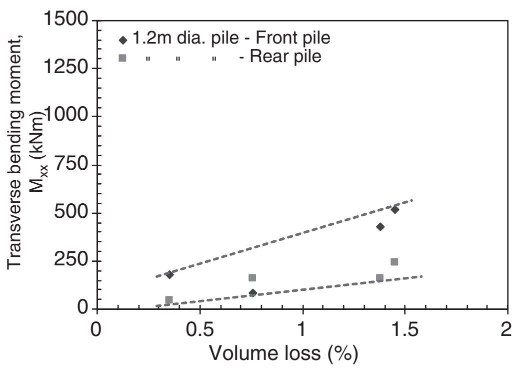

Figure 7. Relationship between transverse bending moment and volume loss.

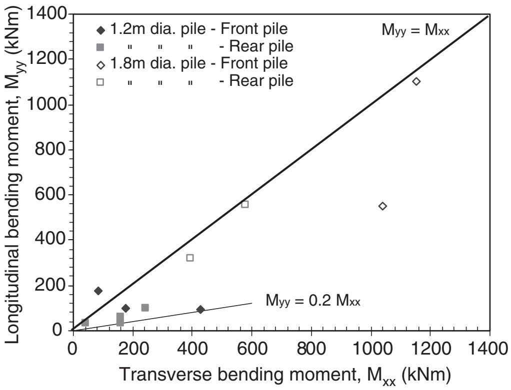

Figure 8. Relationship between transverse and longitudinal bending moments.

moment on most of the front piles was found to be higher than that of the rear piles. Similar trend was also observed for longitudinal bending. Despite the tendency of bending moments and axial force to increase as the volume loss increases, it should be noted that the tunnel-pile configuration at each of the instrumented section (i.e. tunnel-pile distance, pile length, tunnel depth and soil condition) was slightly different.

7.3 *Transverse BM vs. longitudinal BM*

As the piles were subjected to bending in both transverse and longitudinal directions, it is necessary to look into criticality. Figure 8 plots the maximum measured longitudinal bending moment against the transverse bending moment due to SB tunnel advancement for all the 12 piles. As can be noticed, the longitudinal bending moment had a tendency of being equal (i.e. $M_{yy} = M_{xx}$) or smaller (i.e. $M_{yy} < M_{xx}$) than the transverse bending moment. The measured data shows that the M_{yy}/M_{xx} ratio could be as low as 0.2. It is surmised that the ratio could depend on the amount of pressure applied at the tunnel face which could cause varying degree of stress relief in the longitudinal direction. Unfortunately, detailed EPBM records were not available for further evaluation.

8 CONCLUSIONS

A case history of the construction of the MRT North-East Line Contract 704 has been presented. This paper described the response of twelve working piles during the advancement of EPBM. Field measurements from in-pile instrumentation were included with only the results of a single tunnel advancement were reported herein. Some of the main points that could be concluded from the study are as follows:

- Soil settlement caused by tunnelling (i.e. from volume loss) causes downdrag acting on the pile shaft above the tunnel level.
- Axial force only started to increase when the EPBM was at a distance of 25 m approaching the piles. Peak axial force was recorded when the EPBM was adjacent to the piles.
- Axial force and bending moment in the front pile was measured to be higher than the rear pile due to the distance effect and also the pile-soil-pile interaction.
- Both the axial force induced by tunnelling and the bending moment in piles increased as the volume loss increased.
- Tunnelling induced bending moments were small although the piles were located very near to the tunnels (i.e. 1.6 m clear distance). Volume loss of up to 1.5% does not seem to have significant effect on the piles.
- Bending moment in the longitudinal direction was found to be either equal to (i.e. $M_{yy} = M_{xx}$) or smaller than the bending moment in the transverse direction (i.e. $M_{yy} < M_{xx}$).

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