EFFECTS OF CONSTRUCTION OF UNDERGROUND MASS RAPID TRANSIT ON NEARBY PILED-STRUCTURES

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Abstract: The increase demand for underground systems in urban cities particularly for mass transportation has led to many tunnels being constructed in close proximity to buildings and structures. Two case histories on the construction of the Mass Rapid Transit (MRT) tunnels adjacent to structures supported on pile foundation are presented. The first case history is from the North-East Line Contract 704 where an existing piled viaduct bridge was located in between the construction of twin tunnels and aligned in parallel direction. The second case history is from the Circle Line Contract 825 where two vertically stacked tunnels were constructed under an existing piled-building. In-pile instrumentation, field measurements and 3D finite element simulation of the construction showed the response of piled-structures to the tunnel advancement for two different tunnel-pile configurations.

INTRODUCTION

In land scarce Singapore, over 10% of the land is already taken up by roads and related facilities. The key to a sustainable transport system lies in the development of a comprehensive and efficient public transport system without substantial land take. The Land Transport Authority (LTA) was inaugurated in September 1995 to spearhead and integrate all land transport developments in Singapore, including road networks, expressways and rail infrastructures. Today, about 60% of all passenger daily trips are made on the Mass Rapid Transit (MRT), Light Rail Transit (LRT), public buses and taxis, and the target is to reach 75%. To achieve this, the existing Rail-based Transit System (RTS) network will be expanded and the future trend is to build this system below ground, even in outlying areas, to conserve surface land for other developments. The construction of large scale infrastructure projects such as the Mass Rapid Transit System has opened up tremendous scope for tunnelling and underground works in Singapore. Recent underground transportation projects in soft ground, highly variable and mixed ground conditions, and densely developed urban areas have provided interesting geotechnical challenges (Yong & Pang, 2003).

The current MRT system (Fig. 1) comprises three main lines – East-West Line (EWL) including the Changi Airport Line (CAL) extension, North-South Line (NSL) and the recently completed North-East Line (NEL). These lines cover 65 stations and 110 km of rail-based transit. The construction of the first MRT system, the EWL and NSL, commenced in 1983 (Phase 1) followed by Phase 2 (1985), NEL (1997) and the Changi Airport Line extension (1999). The whole of the 67km line and 42 stations under the first two phases were carried out between 1983 and 1990, of which 19km of line and 15 stations were underground. The construction of 5.2m ID bored tunnels in these two phases was carried out with a wide variety of equipment to cater to the varying and mixed ground conditions. These included Greathead type shields to full faced Tunnel-boring machine (TBM) and Earth Pressure Balance (EPB) machines (Krishnan, 2000). Lessons learnt from the early tunneling works have significantly increased the use of EPB to curb surface settlement. In NEL, over 90% of tunneling works were carried out by EPB machines which were significantly different from that used in the earlier Phase 2 construction. In the latter, EPB shield was used mainly to tunnel in soft ground whereas most of NEL shields were versatile for both soft ground and rock.

GEOTECHNICAL CHALLENGES OF THE MRT SYSTEM IN SINGAPORE

North-East Line (NEL)

The S$5 billion (1995 prices) NEL project was carried out between 1996 and 2003, and comprised 16 stations and 20 km of twin tunnel works. The civil engineering works were divided into 12 contract packages on a design and build basis. Of these, 3 contracts were for the construction of major interchange stations, Dhoby Ghaut Station and Chinatown Station, and the construction of a rail depot. The other 9 contract packages include the construction of stations and tunnels, with bored tunnelling specified on 6 of the packages. The various civil contracts were packaged to ensure that each package was sizeable enough to be attractive to the contractors and at the same time ensure it was not too large so that risks could be contained and managed.

Major geotechnical challenges in NEL were the nearby buildings and underground utilities. Buildings were of varying age, with some more than 100 years old. One 75-year building, Foochow Methodist Church, comprised three sections, each of which had been built during a different period with a different type of foundation (footings, bakau piles and RC piles). During excavation, the building settled differentially because of the different foundation systems. The church was temporarily relocated to allow strengthening of the building. Cracks also appeared on 15 shop houses near the construction of Outram Station. The cause of the problem was settlement of soft clay associated with the excavation, aggravated by its weak shallow foundation. LTA installed A-frame steel supports on the walls of the shop houses as well as an underground wall to prevent MRT work affecting these shop houses. Chinatown turned out to be most challenging station with poor and variable ground conditions, pre-war buildings, canals and major thoroughfare – the adjacent roads were closed twice because of cave-ins.

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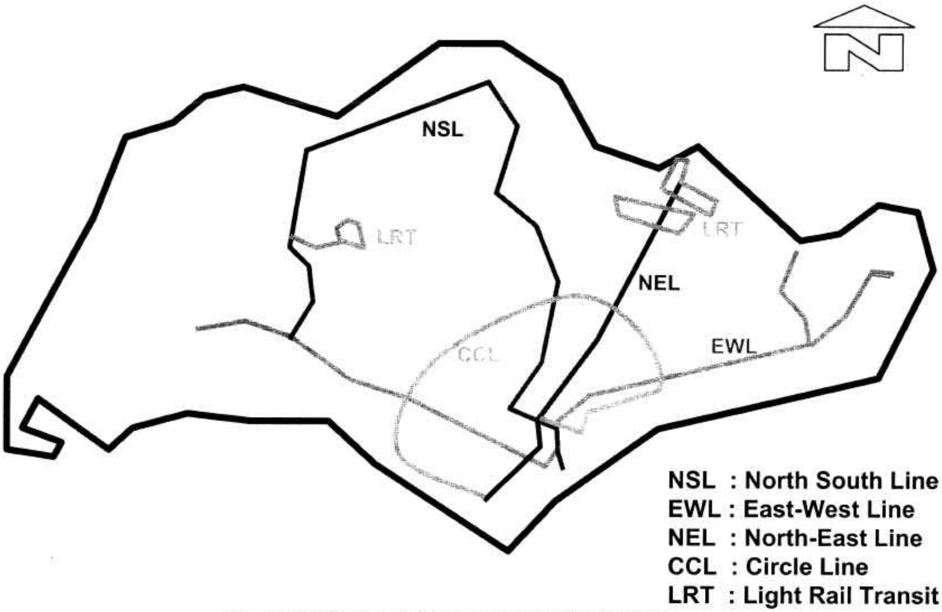


Fig. 1 MRT System in Singapore (courtesy of LTA)

In general, the tunnels are aligned below major road corridors and do not pass directly under buildings. Underpinning was carried out as a precautionary measure at some buildings. A housing block at Braddell and an existing bus ramp at Singapore Changi Airport were underpinned by diaphragm walls and the chapel at the Convent of Holy Infant Jesus, Victoria Street was underpinned by mini and micro piles.

Tunnelling below Penang Lane in the city for NEL was another challenge. The tunnels run in close proximity to Oxley flyover abutment, several old structures and also run beneath existing NSL MRT tunnels. Several measures such as stabilization of the loose soils by grouting, controlled excavation of the face, compressed air application and enhanced supervision had to be implemented to complete the work safely. Settlement of the existing NSL was mitigated by the advance installation of a grouted pipe-roof arch above the NEL tunnels being constructed.

Circle Line (CCL)

The Circle Line (CCL), currently under construction (Fig 1), is a 55km long medium capacity transit system line with a 5.8m ID twin tunnel system. This is an orbital line linking all radial lines running into the city and would provide interconnectivity between these lines outside the perimeter of the downtown core as well as between suburban town centres. It is fully underground and will interface with the existing lines, crosses beneath several underground structures and runs beneath or adjacent to several buildings, thus giving even greater geotechnical challenges than the NEL This line is expected to add another 20 km to 30 km to the transit system with a major portion of it below ground. Chew (2003) gave a good account of the building of CCL in five stages.

Stage 1 comprising a 5.4 km underground line with six stations is the most challenging as it is constructed in a densely developed and heavy traffic part of the civic district with historical monuments, aside from having difficult soft ground and boulder clay formations. It also includes Dhoby Ghaut Interchange Station that is being constructed totally below Orchard Road and the existing NEL Dhoby Ghaut Station, and abuts the existing NSL Dhoby Ghaut station and a row of conservation shop houses. The construction of the 30m deep Museum Station next to the historical Singapore Art Museum with stained glass windows is a very sensitive and delicate task that required stiff diaphragm wall and support system installed with minimal vibration.

Stage 2 extends CCL by 5.5 km and 5 stations and the challenges are its 30m deep stations constructed in thick deposits of soft clay, the interface with the underground Kallang-Paya Lebar Expressway, the Deep Tunnel Sewerage System (DTSS) tunnel and a power cable tunnel. Another 5 km and 5 stations will be added into CCL during Stage 3. Complex interfaces at the existing NEL and NSL stations in Serangoon and Bishan respectively are expected. In addition, the twin tunnels will be spaced within 2 to 3 m in some areas and run into difficult rock-soil interface. Construction of Stages 4 and 5 of CCL will add a further 10 km to CCL and similar problems are expected.

TUNNELLING NEAR PILED-STRUCTURES

The increase demand for underground systems in urban areas particularly for mass transportation purpose can be seen in Singapore as well as big cities like Bangkok, Hong Kong and London. This has led to many tunnels being constructed in close proximity to buildings and structures. Extensive research has been carried out in the United Kingdom particularly on the Jubilee Line Extension (Burland *et al.*, 2002) and North/South Line in Amsterdam (Netzel & Kaalberg, 2003) on the effects of tunnelling near buildings. However, most of these structures are supported on shallow foundations and very little work has been reported on the response of pile foundation subjected to tunnelling. This is due to the fact that most structures were built long before the tunnel is planned. Instrumentation inside pile foundation is prohibited for further investigation, the result of which is a lack of understanding of the response of piled-structure to tunnelling (Mair, 2003).

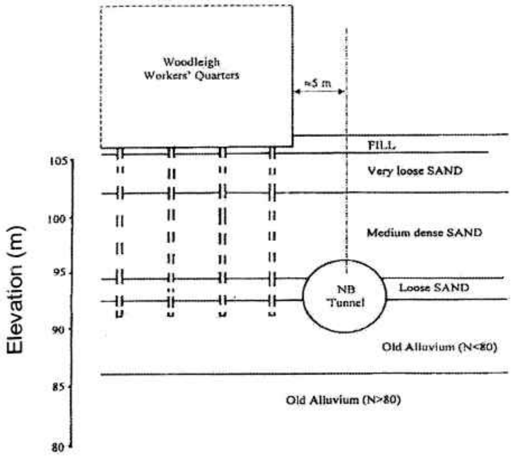


Fig. 2 Tunnelling adjacent to Woodleigh Workers' Quarters for the MRT NEL Contract 705 (Tham & Deutscher, 2000)

In the NEL and CCL construction, the tunnels alignments were close to super-structures and its foundation. Figure 2 shows one of the cases reported by Tham & Deutscher (2000) for NEL Contract 705 where the MRT tunnels were constructed adjacent to the Woodleigh Workers' Quarters. The building is a four-storey concrete structure and is supported on bored piles of 450mm diameter. The North bound (NB) tunnel was bored as near as 5m away from the building edge to its centreline. The tunnel was located at a depth of 14m below ground which correspond to approximately the same length of the piles. Comprehensive monitoring by building settlement markers showed that the building did not suffer detrimental effect from the tunnelling. Settlement of only up to 7mm was measured at the edge of the building. The success of the operation was due to the good tunnelling control where volume loss was only 0.4%. Figure 3 shows another critical section of the tunnelling work in Singapore where complex tunnel-tunnel interaction and tunnel-pile interaction were encountered.

For superstructure supported on pile foundation, there is a need to carry out proper assessment of the pile response when subjected to the new tunnel construction. Tunnelling induced movement on nearby pile foundation is complex. Depending on the pile-tunnel relative position, additional loading from downdrag caused by soil settlement and bending moment caused by lateral soil movement can be undesirable. Figure 4 illustrates the typical responses of pile foundation when subjected to tunnelling adjacent to and under it in addition to the loading from superstructure. In general, the zone of influence due to tunnelling extend typically at 45° from the tunnel towards the ground surface. The criticality of pile response therefore lies in the position of pile relative to the zone. The possibility of tunnel location near pile as identified from the actual case histories reported can be categorised into four different positions as shown in Figure 5.

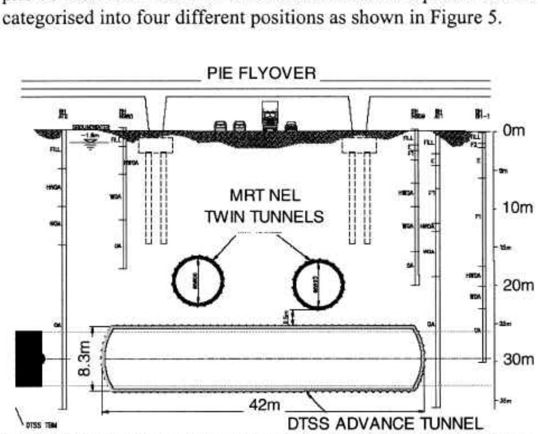


Fig. 3 Interaction between tunnel-pile-tunnel in the construction of MRT and DTSS tunnels (Barraclough *et al.*, 2004)

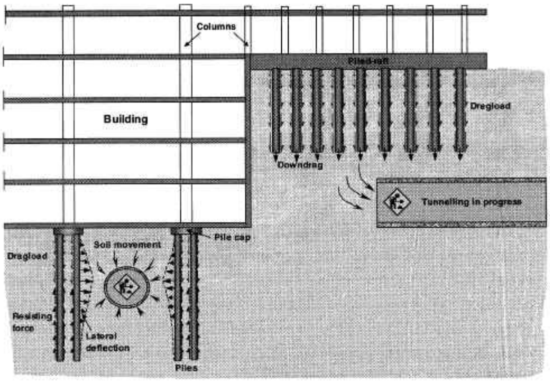


Fig. 4 Illustration of the responses of pile foundation when subjected to tunneling induced movement

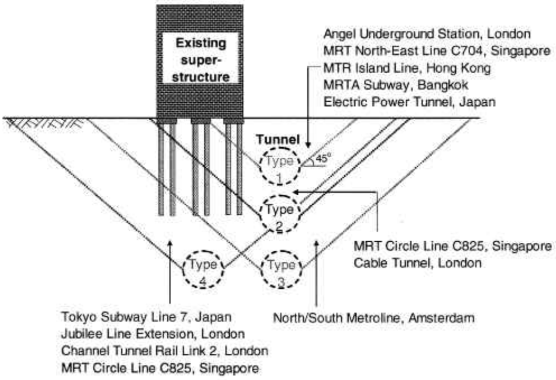


Fig. 5 Tunnel-pile relative position observed in real cases

MRT NORTH-EAST LINE - CONTRACT 704

Background and Overview

Out of the 12 civil contracts awarded (Contract 701 to 712), the case study described here is from Contract 704 which involved the construction of two cut and cover stations i.e. Woodleigh Station and Serangoon Station, 992m twin tunnels from Woodleigh Station to Serangoon Station and 1522m twin tunnels from Serangoon Station to Kovan Station.

Besides the tunnels and stations, part of the package included in the contract was to construct a 1.9km long dual-lane viaduct bridge. The viaduct bridge which consisted of 2 abutments and 39 piers was constructed in parallel alignment with the new twin tunnels configuration and located in between the tunnels. The typical relative position of the pile foundation and the tunnels is shown in Fig. 6 for one of the piers. The piers are supported mainly on bored piles of 1.2m diameter in a group of four piles. Along the alignment, the tunnels were located as close as 1.6m between the extrados and pile edge. The tunnels were bored with two Earth Pressure Balance machines (EPBM) of 6.5m diameter through mainly weathered Granite.

Basically, there were two construction sequence options:-

Option 1 - Tunnel to be constructed first followed by the piled bridge viaduct

Option 2 - Piled bridge viaduct to be constructed first followed by the tunnel

The key concern of *Option 1* was the effects of bored pile installation on the tunnel lining. Very little research has been reported so far on these effects and the confidence on this option was ruled out to avoid additional loading on the tunnel lining. *Option 2* was chosen as the understanding on this type of interaction is better compared to *Option 1*. Studies have been previously carried out by centrifuge testing (Loganathan et al., 1998; Bezuijen & van der Schrier, 1994; Jacobsz *et al.*, 2004) and finite element method (Vermeer & Bonnier, 1991; Mroueh & Shahrour, 2002). Mair (1993) reported a tunnel in London which was successfully constructed with only 1m spacing between pile and tunnel without adverse effect.

Geology and Ground Conditions

Tunnels in C704 were driven through two different geological formations, namely the Bukit Timah Granite and the Old Alluvium. However, most of the piles supporting the viaduct were constructed in the Bukit Timah Granite. This type of formation is found typically in the central part of Singapore. Being one of the oldest formation in Singapore, the Bukit Timah Granite was formed during the lower to middle Triassic period which was about 200 to 250 million years ago (Leong et al., 2003). Both the tunnel depth and pile foundation were located mainly in the completely weathered material (residual soil). The residual soil is classified as Grade VI material according to BS5930 (1981) or better known locally as the G4 material according to a classification system by Dames & Moore (1983). The G4 material is predominantly reddish brown, sandy silty clay.

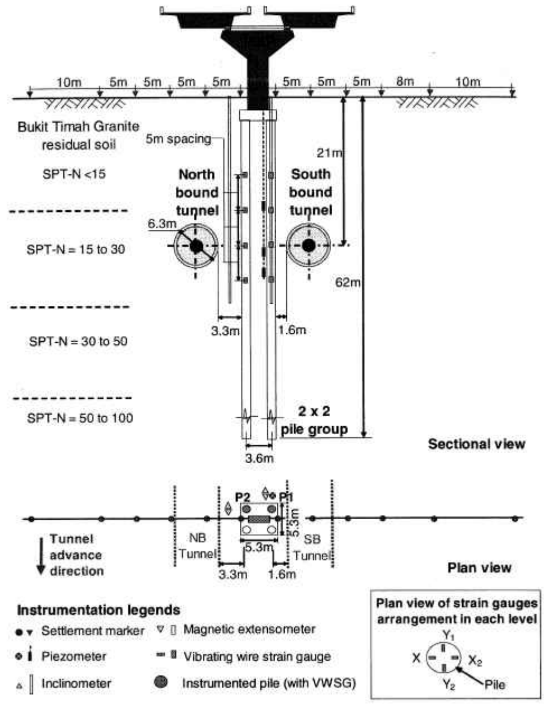


Fig. 6 Typical section and instrumentation layout

Construction Sequence

In C704, construction activities had been coordinated in a way that the bridge viaduct construction did not interfere with the tunnel advancement or in another word, affect the tunnel lining as mentioned above. Therefore, all piles had to be installed prior to the tunnel advancement. Besides, during the advancement, various levels of casting for the viaduct bridge were in progress at different piers. Two EPBMs were used to bore the SB and NB tunnels. Both the 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 advance towards Kovan Station in the opposite direction. The SB tunnel was first driven and followed by the NB tunnel which was approximately 200 to 300m behind the SB tunnel. In general, the SB and NB were located at almost the same level with depths ranging from 16 to 29m below ground surface along the alignment.

Field Monitoring Scheme

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

As part of the monitoring requirement, an instrumentation programme was implemented at six piers location (Pier 11, 14, 20, 32, 37 and 38). A total up to twelve piles were installed with strain gauges at various levels. The strain gauges were located at four different levels spaced at 5m apart which correspond approximately to the tunnel springline, invert, crown and 5m above the crown. Strain gauges were installed at 4 sides (i.e. 2 pairs) in each level with each pair located parallel and perpendicular to the tunnel position. The strain gauges were used to calculate both the bending moment (i.e. transversely and longitudinally) and axial force in the piles.

Besides, in-ground instrumentation such as settlement marker, inclinometer, magnetic extensometer and piezometer were installed at the vicinity of the piles. Figure 6 shows the typical instrumentation layout at one of the instrumented piers. Similar instrumentation arrangement was also implemented at other piers. Further details of the instrumentation results can be found in Pang (in prep) and Coutts & Wang (2000).

Monitoring Results

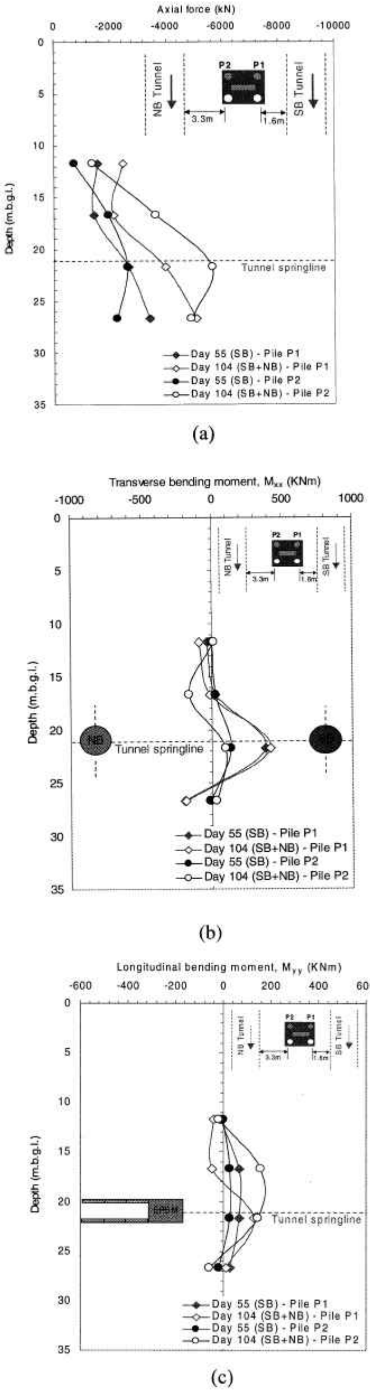
Tunnelling induced axial force

In general, the soil settlement caused by tunnelling (i.e. ground loss) would induce additional axial force i.e. drag load in the nearby piles. As the soil settles, negative skin friction is acting on the pile and therefore leads to drag load. The response is evident from the observation of measured strain gauges data.

Figure 7a plots the maximum measured axial force against depth in the front (P1) and rear (P2) piles at Pier 20 when subjected to both the SB tunnel and SB+NB tunnels advancement. The negative (-ve) value indicated a compressive force. For ease of discussion, the date reported in this paper is converted to 'Day' where Day 0 refers to 1 April 1999 (i.e. the starting day of tunnelling work for C704) as the reference. The results on axial force showed a trend of increasing magnitude with depth up to the tunnel level. With limited strain gauges located below the tunnel springline, a reduction of axial force was apparent below the tunnel springline in pile P2. This reduction implies that positive skin friction was developed to support the dragload. Furthermore, it can be observed that pile P1 experienced higher dragload of 3400kN compared to pile P2 which is 2600kN after the SB tunnel advancement. This is expected since the distance of pile P2 is further away from the SB tunnel. Another likely cause for the lower axial force in pile P2 is the interaction of piles within the pile group. After the advance of NB tunnel, the dragload increased to 5090kN and 5630kN respectively in pile P1 and P2. Furthermore, it can be observed that the increase of slope of the load transfer curve during NB tunnel advancement indicated that the pile shaft friction had not been fully mobilised after the SB tunnel advancement.

Tunnelling induced transverse bending moment

Bending moment is induced in pile when the EPBM is driven close to its proximity owing to the horizontal soil movement. The pile can be subjected to longitudinal bending moment (M_{yy}) and transverse bending moment (M_{xx}). Figure 7b shows a typical plot of the measured M_{xx} 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



(a)

(b)

(c)

Fig. 7 Measured response of pile P1 and P2 at Pier 20 (a) axial force (b) transverse BM (b) longitudinal BM

M_{xx} of 403kNm and 163kN were observed in the front (P1) and rear (P2) piles respectively. Difference of almost three times is apparent in the piles of the same pile group. Similarly as observed in the axial force developed, the difference is likely to be due to the pile-soil-pile interaction and also the shadowing effect from the front pile. Besides, since the rear pile was 3.6m (3 times pile diameter) behind the front pile, a smaller lateral soil movement would be expected at the rear pile which cause a smaller bending. Subsequently, as the NB tunnel arrive at Pier 20, pile P1 shows no significant changes in bending. However, pile P2 shows a reversal of bending especially at the tunnel crown. Due to the limited level of strain gauges installed, investigation of the continuous bending response along the whole pile length was not possible.

Tunnelling induced longitudinal bending moment

Figure 7c 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 subjected to a higher bending of about 80kNm compared to the rear pile (P2) of 40kNm during the advancement of SB tunnel at Day 55. However, maximum measured M_{yy} at Pier 20 are 131kNm and 49kNm respectively for P1 and P2 occurred at Day 50.

Three-Dimensional Finite Element Back-Analysis

Further studies were conducted using a three-dimensional finite element model that was set up to back-analyse the case history. Owing to the variability of the soil condition, pile-tunnel relative distance and volume loss at different sections, a single analysis to represent the whole problem is impossible. For back-analysis work described in this paper, monitoring results of pier 20 were used for comparison since it is the most critical section in term of tunnel-pile distance (i.e. 1.6m) and volume loss (i.e. 1.38% for SB tunnel and 1.67% for NB tunnel).

Finite element mesh

The analysis described in this paper were performed using finite element program ABAQUS (Hibbitt, Karlsson & Sorensen Inc., 2001). Figure 8 shows the typical finite element mesh adopted for the simulation. Only the effect of single tunnel advancement is presented here. All the soils, tunnel lining, pile foundation were modelled using 20-node continuum solid element. The FE mesh consisted of approximately 5080 elements with total up to 19107 nodes. Assuming the pile foundation existed on both side of the tunnel, only half the mesh was created taking the plane of symmetry at the tunnel axis. In reality, this was not the case and it is then demonstrated through a full mesh analysis that there was no significant effect on the analysed results.

After a study on the boundary effects, the mesh was determined to be 74m on the vertical length, 62m on the transverse width and 90m on the longitudinal length. Out of the 90m length, the excavation was simulated up to 60m, which left 30m of soil in front of the tunnel face. In the model, lateral boundary fixity was assigned perpendicular to the vertical plane whereas both the lateral and vertical fixities were assigned to the bottom of the mesh. The pile group was located far enough such that the effect of tunnel approaching and leaving on the pile response could be studied. Therefore, the centre of pile group was modelled 33m away from the mesh front boundary. The piles were assumed wished-in-place. Actual pile diameter of 1.2m and length of 62m in pile group of four piles were modelled. A clear distance of 1.6m between the pile edge and the SB tunnel extrados was examined. The actual pile cap size of 5.3m x 5.3m with a thickness of 1.5m was also simulated. No interface element was prescribed between the pile and the soil. In the actual condition at Pier 20, the piles were only loaded up to pile cap level prior to tunnel advancement. Therefore, no loading was prescribed on the pile head in the analysis.

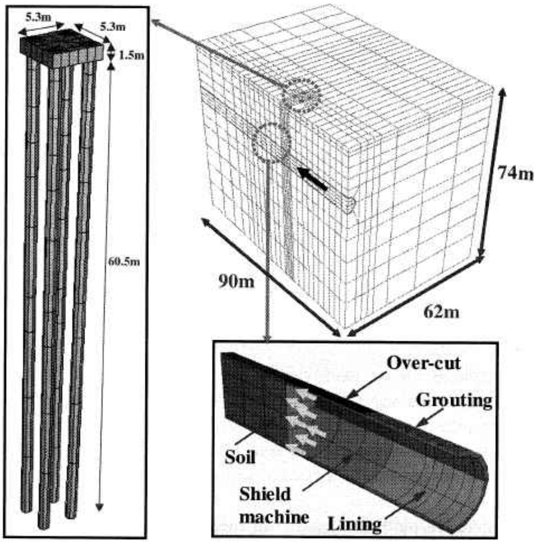


Fig. 8 Typical three-dimensional finite element used

Ground profile

The soil profile is modelled according to the boreholes nearest to Pier 20. At this section, only the Bukit Timah Granite residual soil was encountered. Figure 9 shows the idealised soil profile for analysis together with the position of tunnels and piles. Within the G4 material, further layering is classified according to the SPT-N value (Wang, 2003). G4a, G4b, G4c, G4d and G4e represent respectively the residual soil with SPT-N < 15, 15 to 30, 30 to 50, 50 to 100 and >100blows/30cm. At Pier 20, the top 16m consists of G4a material. The 1.5m of fill at the top soil layer was modeled as G4a. This was followed by 14m of G4b, 14m of G4c and finally G4d which was encountered all the way without encountering G4e material. The tunnels are located at a depth of 21m below ground surface. The water table is assumed to be located at 3m below ground surface. All the analyses were carried out with coupled consolidation model to simulate the rate of tunnel advancement.

Tunnel advancement simulation procedures

The magnitude of ground movement during the EPB shield tunnelling is highly related to the operational and human factor rather than the ground condition (Shirlaw, 2000).

To accurately simulate the effect of shield tunnelling on ground movement, four main phases of the construction process have to be considered in the model. This includes the face pressure, shield machine advancement (i.e. over-cutting), tail void closure and lining installation. In this paper, the tunnel was modelled using solid elements to represent the shield machine, overcut, grout and lining. The tunnel face was supported by application of horizontal pressure during the shield advancement. A pressure of 100kPa as measured (Knight-Hassell & Tan 2000) was assigned in the analysis.

In the FE analysis, advancement was simulated by step-by-step removal of soil element and activation of shield, lining and grout elements. The excavation was simulated with 3m per step. The advancement rate of 9m/day was simulated in the analysis. During each excavation step, soil elements were removed with simultaneous application of pressure on the tunnel face and activation of shield and over-cut elements. At the same step, the shield and over-cut elements at the back were removed to simulate the shield length of 9m. In addition, the initial grout and lining elements were activated there. In C704, the grout mix setting time was in the range of 5 to 12 hours (Shirlaw *et al.* 2004). With the tunnel advancing rate of 9m/day, the length of grout to be in fluid state would be between 1.875m and 4.5m. Therefore, the length of initial grout was modelled as 3m length. In the subsequent tunnel advancement, the initial grout element was replaced with grout of hardened property.

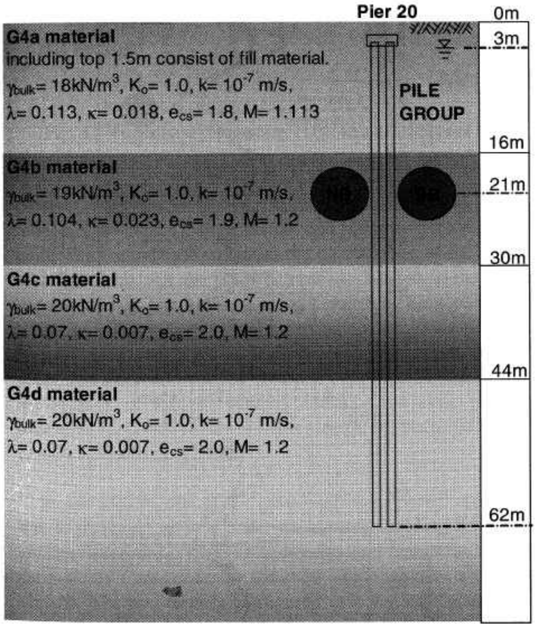


Fig. 9 Typical soil profile simulated in FE analysis

Material models and parameters

To consider the pre-failure non-linear stress-strain behaviour, the Strain Dependent Modified Cam Clay (SDMCC) model developed by Dasari (1996) was used. Within the elastic part, the model incorporated the variation of shear and bulk modulus with strain which follow the power laws. On the wet side of the critical state, the classical MCC yield surface was assumed. However, on the dry side of critical state, the Hvorslev surface and a no-tension cut off line were further incorporated into the SDMCC model. More details of the formulations and parameters selected can be found in Pang (in prep). The general soil properties and standard MCC parameters adopted for different soil layers are shown in Fig. 9.

The structural properties such as lining and shield machine were modelled as linear elastic material. The shield machine was modelled with solid elements with Young's modulus of 200GPa and Poisson ratio of 0.25. The concrete lining was modelled with Young's modulus of 28GPa, density of 24kN/m^3 and Poisson ratio of 0.2. The over-cutting was modelled using solid elements to induce deformation. Therefore, calibration with measured surface settlement during shield passage is necessary. After sensitivity studies, the elements were assigned Young's modulus of 1000kPa and a Poisson ratio of 0.2. Similarly for the grout, an initial Young's modulus of 1000kPa, Poisson ratio of 0.2 and density of 24kN/m^3 were assigned to the grout of liquid state. The hardened grout element which replaced the liquid grout was assigned stiffness of 50MPa. The pile was assumed to be linear elastic. From the back-analysis carried out on the instrumented pile load test at the C704 site, Young's modulus of 28 GPa was selected for the strain gauge interpretation and also for the finite element analysis. Poisson ratio of 0.2 was assigned.

Results of the Finite Element Analysis

Ground surface settlement

Figure 10 shows the comparison between the computed and measured transverse surface settlement trough after the tunnel face has passed the monitoring section by 27m (i.e. plane strain condition). A fairly good agreement of trough shape and magnitude is achieved although the trough is slightly wider numerically at the far-field. The effect of piles on the surface settlement can also be noted with the reduction in settlement extending as far as 25m away from the tunnel axis.

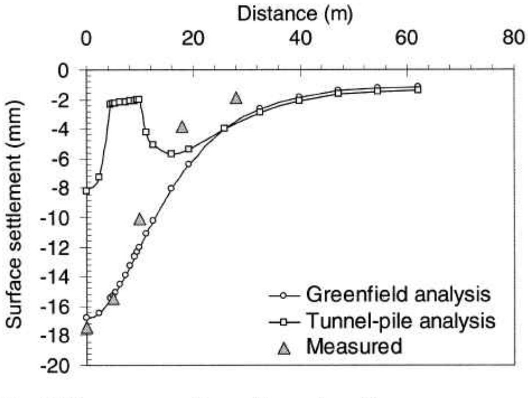


Fig. 10 Transverse surface settlement profile

Axial response of piles

Figure 11 shows the predicted axial and bending responses of the piles when the tunnel face is 27m away from the centre of the pile group (where plane strain condition has been achieved). The measured data for axial force, transverse BM and longitudinal BM are also included for comparison. As can be seen in Figure 11a, the axial force of piles in the same row i.e. P1 & P4 (front row) or P2 & P3 (rear row) behaved in the same manner and magnitude. The axial force is increasing with depth down to the tunnel springline which indicated the piles are subjected to downdrag from the soil settlement. Below the tunnel springline, the axial force reduced owing to the positive skin friction developed to support the dragload. The predicted axial force is slightly higher than the measured for both the front and rear piles. Piles settlements predicted are very small with maximum magnitude at the pile head up to 2.5mm only.

Bending response of piles

Figure 11b shows the bending moment of piles in the transverse direction. The maximum magnitudes occur at the tunnel springline. The maximum computed lateral deflection of piles is only up to 13mm. The rear piles were subjected to a lower magnitude compared to the front piles. Looking in another direction parallel to the tunnel advancement i.e. longitudinal, the bending moment reduced by more than half the magnitudes observed in the transverse direction (Fig. 11c).

For clearer visualisation of the development of piles responses, the computed deformation of the three-dimensional analysis are extracted for different stages of the tunnel advancement as shown in Figure 12 for both the transverse and longitudinal planes. On the transverse plane, as the tunnel progressively approached the pile group, the deformation became noticeable particularly when the tunnel face passed by the pile group. As the tunnel face gradually leave the pile group, the piles in the same row, i.e. front piles and rear piles behaved similarly which indicated that plane strain condition has been reached. The response of piles were rather different in the longitudinal plane. Prior to the tunnel arriving at the pile group, the piles deflected towards the tunnel with maximum movement at the pile head level and not at the tunnel springline. Subsequently, as the tunnel advanced pass the pile group, the piles were gradually pushed forward in the same direction of the tunnel advancement. The response is caused by the lateral soil movement which moves in the same trend. Similar observation was also noted in measured data reported by Phienwej (1997) for shield tunnelling work. Besides, the piles in the same row were subjected to different magnitude of deflection even after plane strain condition has been achieved unlike in the transverse deflection.

Figure 13 plots the ratio of the maximum responses of the front pile to the rear pile in both the pile group analysis and single pile analysis. Apparently, the front pile is subjected to higher magnitude compared to the rear pile in both the axial and bending responses up to 2.5 times. However, the ratio of axial response is rather insignificant. The measured ratio also shows good agreement with the finite element analysis. In fact, the soil movement caused by tunnelling reduces as the distance is further away from the tunnel. The positive ratio is attributed to the fact that the front pile is subjected to more soil movement compared to the rear pile which is located three times pile diameter away (i.e. 3.6m). In addition, pile group interaction effect also influences the results.

MRT CIRCLE LINE 1 - CONTRACT 825

Background and Overview

The on-going Contract 825 project formed the first stage of the CCL construction (CCL1). The CCL1 line, also known as the Marina Line is part of the five stages to be built (Fig. 1). In the

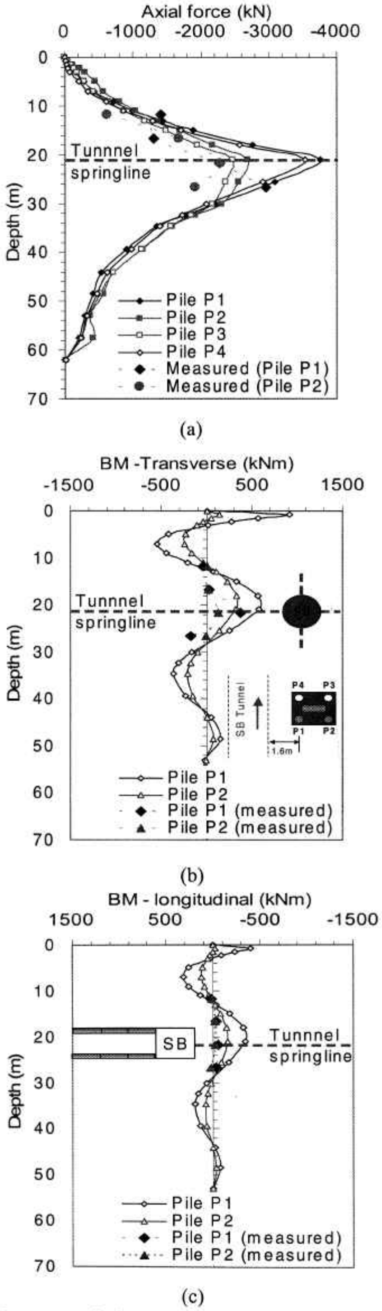


Fig. 11 Response of pile foundation (a) axial force (b) transverse bending moment (c) longitudinal bending moment

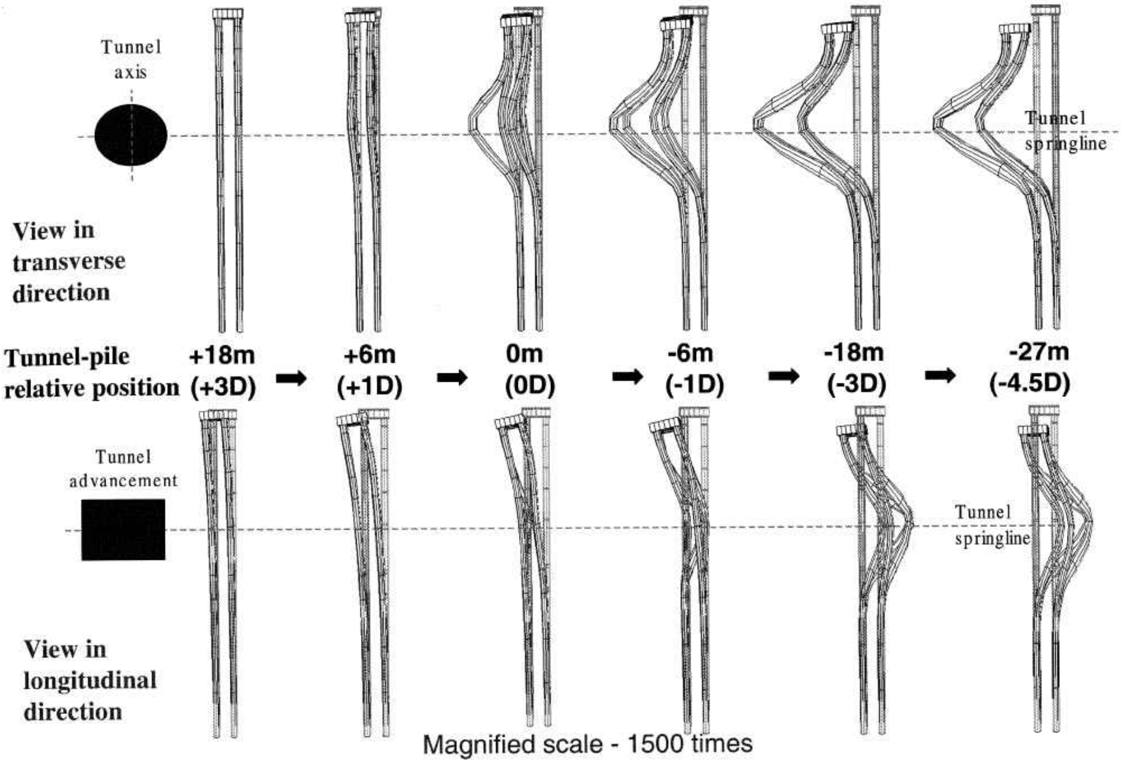


Fig. 12 Visualised pile group response at different stages of tunnel advancement computed from finite element analysis

contract, four stations namely the Dhoby Ghaut Station, Museum Station, Convention Centre Station and Millenia Station are to be built. The contract also includes the construction of twin tunnels of 1.5km long. All the constructions are located in the densely populated civic and business district centre of Singapore. Inevitably, the construction has to be carried out very near to existing heritage structures such as Raffles Hotel, Singapore Arts Museum, Cathedral and various high-rise buildings. Figure 14 shows the location of tunnels, stations and also the close proximity structures in Contract 825. Further details on the project can be found in Osborne *et al.* (2004).

In this project, one of the great challenges posed to engineers was to construct tunnels under an existing building beneath the Raffles Boulevard. Figure 15 shows the twin tunnels bored under the 5-storey frame concrete structure which includes a basement carpark. The two tunnels configured in a vertically stack alignment passed beneath the structure which link the Marina Square and the Pan Pacific Hotel. The structure was supported on driven Raymond Step-Taper steel piles of 324mm diameter. The piles are founded at a depth of approximately 11.5m below the basement. The main columns are supported on pile groups of four, eleven and seventeen piles whereas the wall sits on a stretch of single piles. The piles are located as close as 1.12m to the tunnel extrados. Two EPB shield machines of 6.58m diameter were used to bore the twin tunnels and were located very near to each other with a clear spacing of 3.84m from their extrados. The upper tunnel is located at a depth of 12.5m below the basement car park.

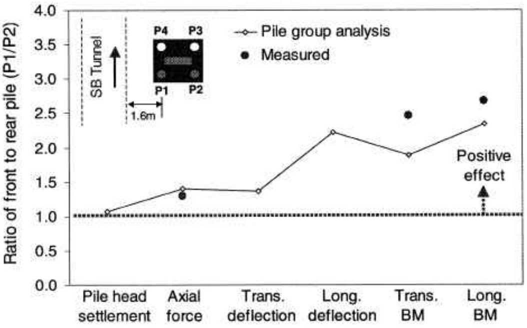


Fig. 13 Response ratio of front pile to rear pile

Geology and Ground Conditions

From the soil investigation carried out, the structure is generally founded on the Old Alluvium with the degree of weathering varying with depth. The Old Alluvium is an alluvial deposit that has been variably cemented and has the strength of weak rock (LTA, 2001). The Old Alluvium which composed of silty sandy clay can be classified into five classes, i.e. OA1 to OA5 which are defined by the SPT-N of <10, 10 to 30, 30 to 50, 50 to 100 and >100 respectively. However, the 7m of soil below the basement consists of mixed layers of fluvial sand (F1) and clay (F2), marine clay (M) and fill material, typically the Kallang Formation (Fig. 15). Ground water is close to the original ground level. The piles are generally founded on the dense Old Alluvium material (i.e. OA5). Material of OA3 to OA5 was encountered during the north bound tunnel advancement whereas the south bound tunnel encountered only OA5 material.

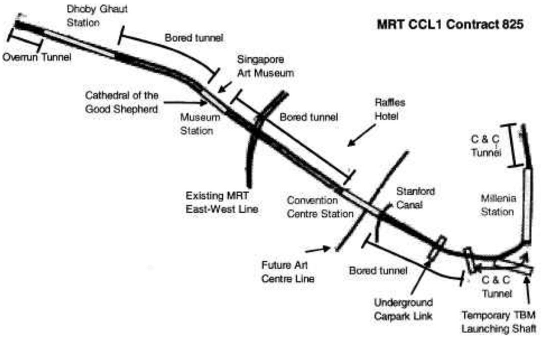


Fig. 14 Location of MRT CCL C825

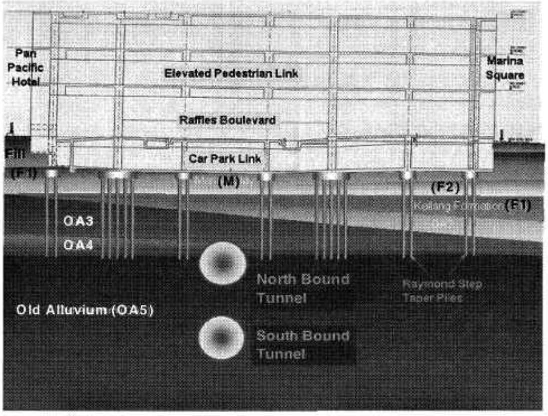


Fig. 15 Tunneling under the link structure between Pan Pacific Hotel and Marina Square

Construction Sequence

The tunnels were driven by two earth pressure balance machine (EPBM) manufactured by Herrenknecht and has an outer diameter of 6.58m and length of 8m. When the EPBM were under the building, good soil condition was encountered, therefore leading to good advance rate (i.e. approximately 50mm/min) and progress rate (up to 10 rings/day). A face pressure of 150kPa was maintained in the chamber to provide face stability although it is realised that the material encountered is generally stable and has a considerable stand-up time even without the pressure. The first EPBM (for south bound tunnel) was launched from Millenia Station on the 22 January 2003 and advance towards the Convention Centre Station. This is followed by the second EPBM (for north bound tunnel) which was launched two months later from the same launching shaft. The construction of the tunnels were scheduled such that the lower tunnel was bored first and followed by the upper tunnel to minimise the effect on the structure. Initially the tunnels started off in a horizontally parallel position for length of approximately 230m (Fig. 16a). However, the tunnels were then gradually shifted into a vertically stacked alignment when reaching the link structure due to space constraint from the pile foundation (Fig. 16b).

One of the main challenges in this section was the intersection of three numbers of piles with the upper tunnel (Fig. 17). Two piles were encountered at the front wall and one pile at the rear wall of the structure. Initially, only two piles were expected. However, an unexpected H-pile of 375mm x 375mm was encountered exactly adjacent to one of the piles to be expected during tunnelling. Approximately 3m length of each pile was to be removed to allow the tunnel machine to pass through. The EPBM was stopped allowing the piles to be cut-off manually. To avoid loading on the tunnel lining, polystyrene foam block was attached to the base of the pile (Fig. 18).

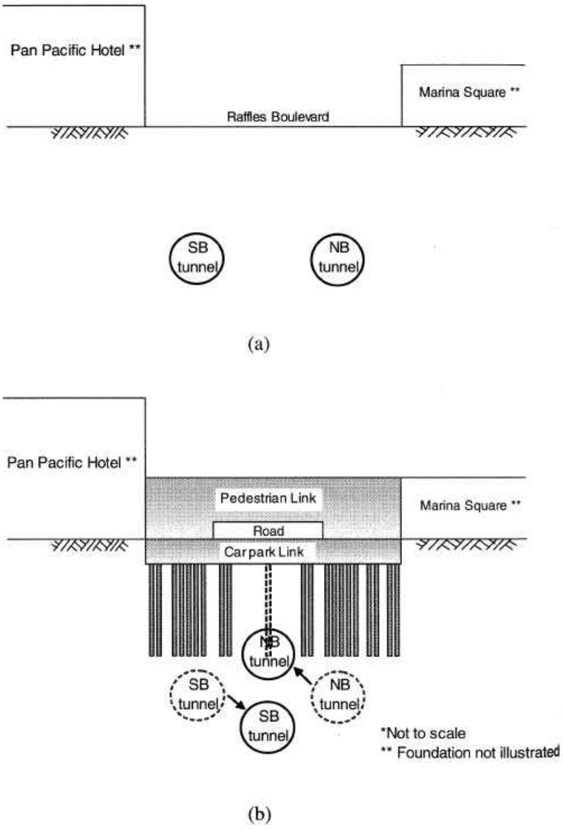


Fig. 16 Alignment of tunnels (a) before reaching structure (b) under structure

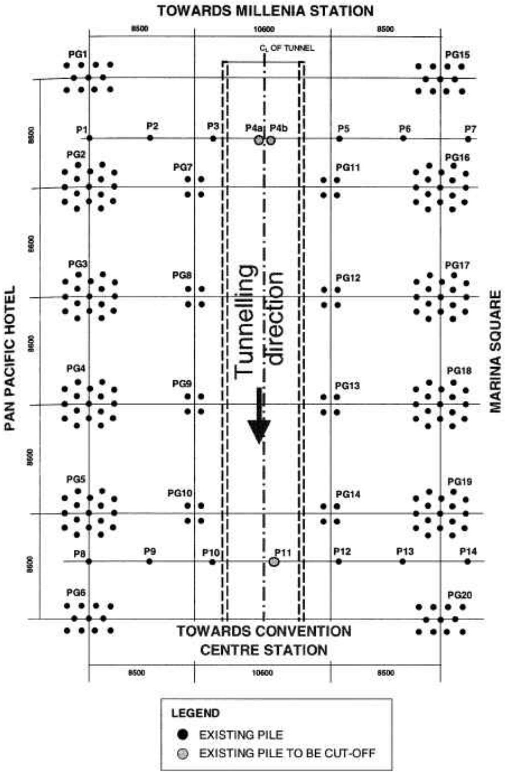


Fig. 17 Foundation layout of the link structure

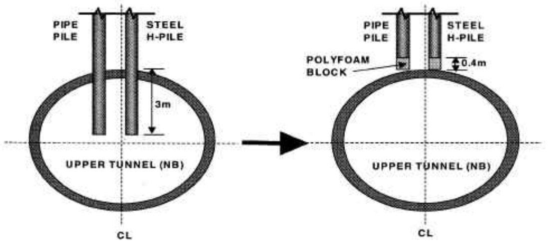
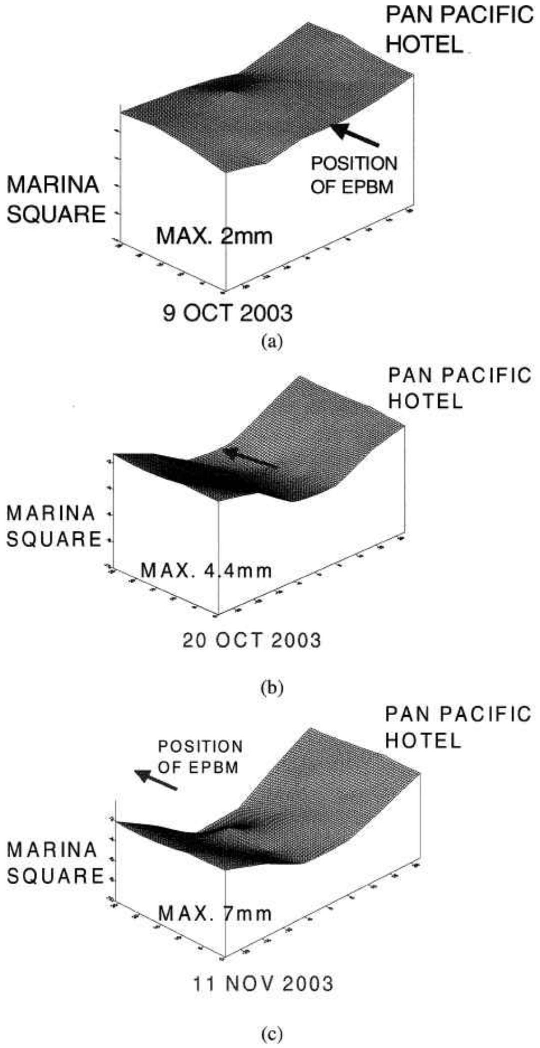


Fig. 18 Pile cut-off at one of the wall section



(a)

(b)

(c)

Fig. 19 Measured building settlement for NB tunnel advancement (a) EPBM at front wall (b) EPBM at rear wall (c) EPBM leaving the structure

Monitoring Scheme and Results

As part of the stringent requirement laid by the Land Transport Authority (LTA), the building was fully instrumented. Settlement markers were installed in almost all the columns at the basement level of the structure. In addition, tilt meters and tape extensometers were also installed in some of the columns and walls.

During the advancement of the SB tunnel, the maximum column settlement recorded is only up to 3mm. Subsequently, after the NB tunnel has advanced, the maximum accumulated settlement is up to 7mm. Figure 19 plots all the columns settlements in three-dimensional visualisation for cases when the face of the second EPBM (for NB tunnel) was (a) at the front wall of structure (b) at the rear wall of structure (c) at a distance of 10 times tunnel diameter away from the rear wall. With relatively good ground conditions and well controlled tunnelling procedure, the maximum and differential measured settlements were kept small.

CONCLUSIONS

Two case histories from the construction of the MRT North-East Line Contract 704 and Circle Line Contract 825 in Singapore are presented. The first case highlighted the construction of piled-bridge viaducts in between two parallel tunnels. Field measurement with in-pile instrumentation was implemented and some of the data are reported herein. Furthermore, a three-dimensional finite element model was created to back-analyse the problem. Good agreement between the measured and prediction was achieved. The second case history describes the effect of twin tunnels in vertically stacked alignment constructed adjacent and under the pile foundation supporting a 5-storey building. During the construction, three piles that intersected the upper tunnel were removed manually from the inside of the EPBM. Settlement of the columns was measured up to 7mm after the twin tunnels have passed.

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