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Effects of tunnelling on existing support systems of perpendicularly crossing tunnels

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ABSTRACT

The interactions between perpendicularly crossing tunnels in the Sydney region are investigated using a full three-dimensional (3D) finite element analysis coupled with elasto-plastic material models. Special attention is paid to the effect of subsequent tunnelling on the support system, i.e. the shotcrete lining and rock bolts, of the existing tunnel. The results of the analysis show that in a region such as Sydney, with relatively high horizontal stresses, installation of the new tunnel causes the shotcrete lining of the existing tunnel to be in tension in the side facing towards the tunnel opening and in compression at the crown and invert. The pre-stressed rock bolts are usually tensioned more in the sections closest to the tunnel opening. For this particular study, if a new tunnel is driven perpendicularly beneath an existing tunnel, significant increases are induced in the bending moments in the shotcrete lining at the lateral sides of the existing tunnel and in the axial forces at its crown and invert. The increase in side bending moments causes further tensile cracking but the crown and invert stresses remain within the thresholds for both compressive failure and tensile cracking for shotcrete lining of typical concrete quality. Moreover, the driving of the new tunnel causes the tensile forces in the existing side rock bolts to increase and those in the existing crown rock bolts to decrease. In contrast, if the new tunnel is driven perpendicularly above the existing tunnel, compressive failure of the existing shotcrete lining is induced at the crown of the deeper tunnel for concrete of typical capacity and a significant tensile force increase of the existing rock bolts around the crown. It is concluded that in order to ensure the stability of the existing tunnel, local thickening is needed at the sides of the existing shotcrete lining if the shallow tunnel is installed first and local thickening is needed at the crown if the deep tunnel is installed first.

1. Introduction

Largely because of the need to develop more efficient and environmentally friendly infrastructure in congested urban cities, the number of constructed tunnels has been gradually increasing during past decades. Thus, it is highly likely that some new tunnels may need to be designed and constructed nearby existing tunnels. For example, the Jubilee Line Extension Project in London involved the construction of sections with twin tunnels running side-by-side and one above the other [1]. In this case, it is important to investigate the effects of tunnelling on the support systems of existing adjacent tunnels so that the existing tunnels can continue to operate safely both during and after construction of the new tunnel.

Interaction between closely-spaced tunnels has been studied in the past using a variety of approaches: field observation, physical model testing, empirical/analytical methods, and finite element modelling. Kim [2] presented a good summary of the studies conducted before 1996. Thus, only the investigations since 1996 are reviewed in this paper.

Field observations of the interactions between closely-spaced crossing tunnels on the Jubilee Line Extension in London were conducted by Kimmance et al. [3] to measure the deformation created in existing tunnels at the crossing point caused by adjacent and cross-cutting excavations. They concluded that there would always be settlement of an existing tunnel when a new tunnel passed beneath it. Yamaguchi et al. [4] analysed the behaviour of the tunnels and the surrounding ground, compared the solutions with monitoring data obtained during the construction of four subway tunnels that run close to each other in the Kyoto City, and concluded that a redistribution of the ground stress was caused by shield excavations when a shield was passing an existing tunnel. Asano et al. [5] proposed an observational excavation control method for adjacent mountain tunnels involving monitoring the relationship between the wall displacement of a new tunnel and the incremental stress in the concrete lining of an existing tunnel.

A series of physical model tests on closely-spaced tunnels in kaolin clay samples was performed by Kim [2]. In his tests, three tunnels were constructed in which the two new tunnels were

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either parallel to the existing tunnel or perpendicular to it. The results indicated that the interaction mechanisms depended on the geometry of the tunnels, e.g. side-by-side and parallel, or crossing and perpendicular. For side-by-side and parallel tunnels, construction of a new tunnel would lead to changes in the lateral stresses acting on the existing tunnel. For perpendicular tunnels, settlements might be induced in the upper tunnel if a new excavation was constructed beneath an existing tunnel. Finally, he concluded that the interaction mechanisms between adjacent tunnels were extremely complex and further studies were needed.

Field observations remain the key to understanding the interaction behaviour between adjacent and crossing tunnels, but often distortion and settlement data are incomplete and comparisons with free-field movements are unavailable. Moreover, it is clear that model testing can only be used to study limited interaction features. Empirical/analytical methods that deal with multiple tunnels have been used to obtain solutions by superimposing the independent transverse settlement profile [6] predicted for each individual excavation to obtain the final accumulated settlement profile [7]. Thus empirical/analytical methods ignore the presence of an existing tunnel and are therefore not realistic. The use of realistic finite element modelling seems to be a promising way to investigate the interaction between adjacent and crossing tunnels.

Most finite element analyses of the interactions between adjacent tunnels [1,8] involve a two-dimensional (2D) plane-strain approximation in which a section perpendicular to the tunnel axis is considered. While 2D finite element analyses have been both convenient and useful in the past, they suffer some major drawbacks: (1) empirical factors are involved in various 2D methods such as the convergence-confinement method, the progressive softening method, the volume loss method and the gap method; (2) 2D analyses provide no information on the tunnel heading and in particular on the behaviour of support systems and the surrounding ground as the tunnel face advances. As the behaviour of the tunnel heading is essentially three-dimensional (3D), it is not possible to reproduce such behaviour accurately in a 2D plane-strain analysis. Thus, there has been considerable interest in recent years in developing 3D numerical models to investigate the tunnel interaction problem.

Ng et al. [9] carried out a series of 3D finite element analyses to investigate the interactions between large parallel hypothetical twin tunnels constructed in stiff clay using the new Austrian tunnelling method (NATM). It was found that the lag distance between the twin tunnel faces had a stronger influence on the horizontal movements than on the vertical movements of each tunnel, and there was a transfer of load from the lagging tunnel to the leading tunnel. Liu et al. [10] investigated the effects of tunnelling on existing support systems of adjacent side-by-side, piggyback and staggered parallel tunnels in the Sydney region using full 3D finite element modelling. It was concluded that the effects strongly depended on the relative locations of the existing and new tunnels, and in a region such as Sydney with high horizontal regional stresses, the driving of a new tunnel in a piggyback position or staggered parallel to the existing tunnel more readily caused adverse effects on the existing tunnel support system than a new side-by-side tunnel parallel to the existing tunnel.

In summary, the literature reviewed above indicates that an extensive amount of research has been carried out on tunnel interactions. Field observations and physical model testing suggest that the mechanisms governing the interaction between adjacent tunnels are highly complex. The empirical/analytical methods ignore the presence of an existing tunnel and are not realistic. Simplified 2D finite element modelling has some major shortcomings as mentioned above. Moreover, the 2D model can only be used to study interactions between parallel tunnels without rock bolt support and cannot be used to investigate interactions between perpendicular tunnels. With the rapid development of computing power, interactive computer graphics, topological data structures and storage capacities during the past few years, 3D finite element modelling has become a powerful tool for investigating the interaction between new tunnels and existing structures [10–13]. However, until now, no research has been reported for investigating the interactions between perpendicularly crossing tunnels using full 3D numerical techniques.

The present research program is intended to study the effects of new tunnelling on the support system of an existing adjacent perpendicularly crossing tunnel in the Sydney region, with a high regional stress regime, using full 3D numerical techniques, i.e. ABAQUS and TUNNEL3D presented in a previous paper [10]. The main objectives are: (1) to obtain the initial internal force and deformation fields of the existing support system by simulating the construction process of the existing tunnel; (2) to investigate the complete response of the existing support system to the installation of an adjacent perpendicularly crossing tunnel; (3) to study the differences between the behaviour of the existing support system when the new tunnel passes perpendicularly over or beneath the existing tunnel; and (4) to qualitatively compare the numerical results with the physical model test data presented by Kim [2] on perpendicularly crossing tunnels.

2. Geometrical and numerical models

The layout of perpendicularly crossing tunnels is depicted in Fig. 1a. The shallow tunnel has a cover depth of 15 m (Fig. 1b) and the deep tunnel is at a depth of 27 m (Fig. 1c). Thus, the centre-to-centre spacing between the two perpendicular tunnels is $S = 12$ m and the pillar width is approximately 3.5 m. The crossing area is of particular interest, where the installations of tunnels are modelled in a step-by-step procedure, i.e. first excavated and then supported using a shotcrete lining and rock bolts step-by-step. The characteristic dimensions, i.e. the radius ($R = 5$ m) of both polycentric tunnels, the thickness ($t = 250$ mm) of the shotcrete lining, the length ($l = 3$ m), and the diameter ($d = 20$ mm) of the rock bolts, are shown in Fig. 1d.

Fig. 2a presents the finite element model for the perpendicularly crossing tunnels. A total of 36,708 eight-noded isoparametric hexahedral elements with reduced integration points and hourglass controls (C3D8R) are used to represent the rock mass. The reduced integration scheme ensures that the shear locking phenomena usually observed in the eight-noded hexahedral element does not occur and the hourglass control is implemented to avoid the zero-energy mode, which usually happens in the coarse mesh of this type of element. A total of 1440 four-noded shell elements with reduced integration points and five section points in the thickness direction (S4R) are used to model the shotcrete lining, and 1386 two-noded truss elements (T3D2) are used to model the 462 rock bolts, as shown in Fig. 2b. Thus, there is a total of 132,105 degrees of freedom. In the analysis, the shell elements are constrained to move with the exposed tunnel surface, i.e. the surface of the eight-noded isoparametric hexahedral elements around the interior of the tunnel opening has a shell element attached to it after the tunnel excavation is simulated. The truss elements are embedded into the eight-noded isoparametric hexahedral elements to model the interaction between the rock bolts and the surrounding rock mass.

According to Pells's study [14], in the metropolitan area of the Sydney region, most of tunnels are constructed in the Hawkesbury sandstone, a benign medium with intact unconfined compressive strength of 25–45 MPa, Brazilian tensile strength of 2–3 MPa, and Young's modulus of 2.5–8 GPa. However, the intensive weaknesses (bedding discontinuities, joints and faults) in the Sydney region

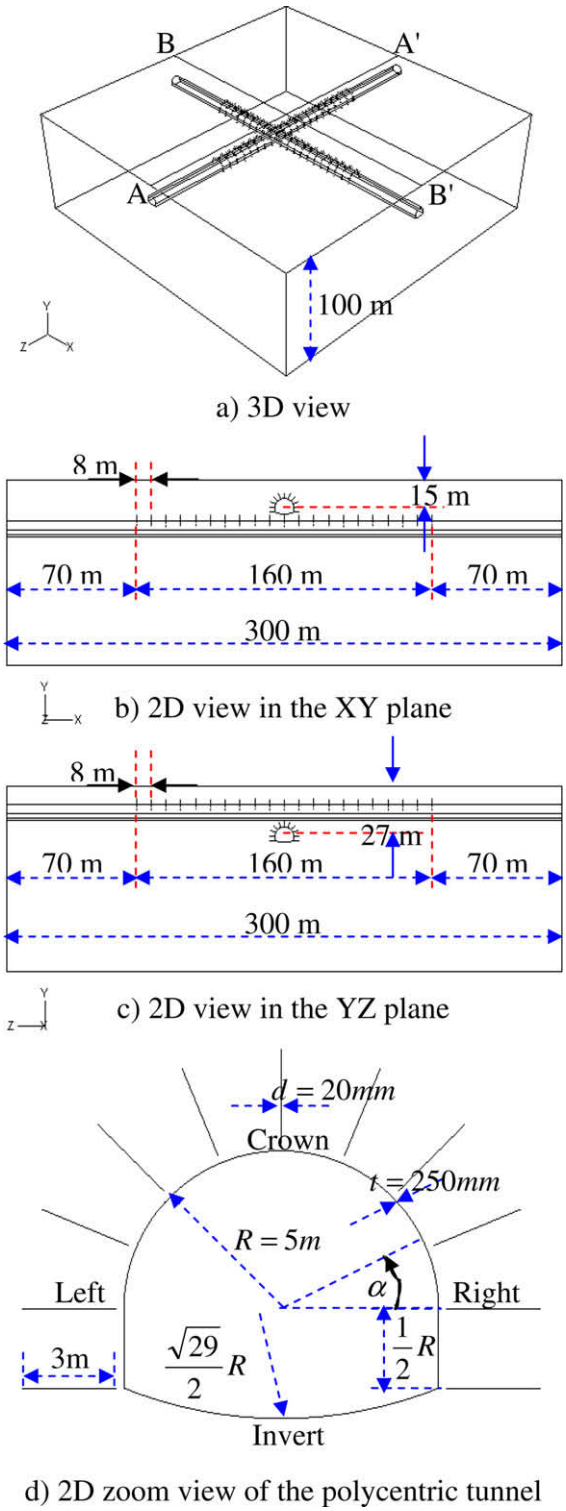


Fig. 1. Geometrical model for perpendicularly crossing tunnels.

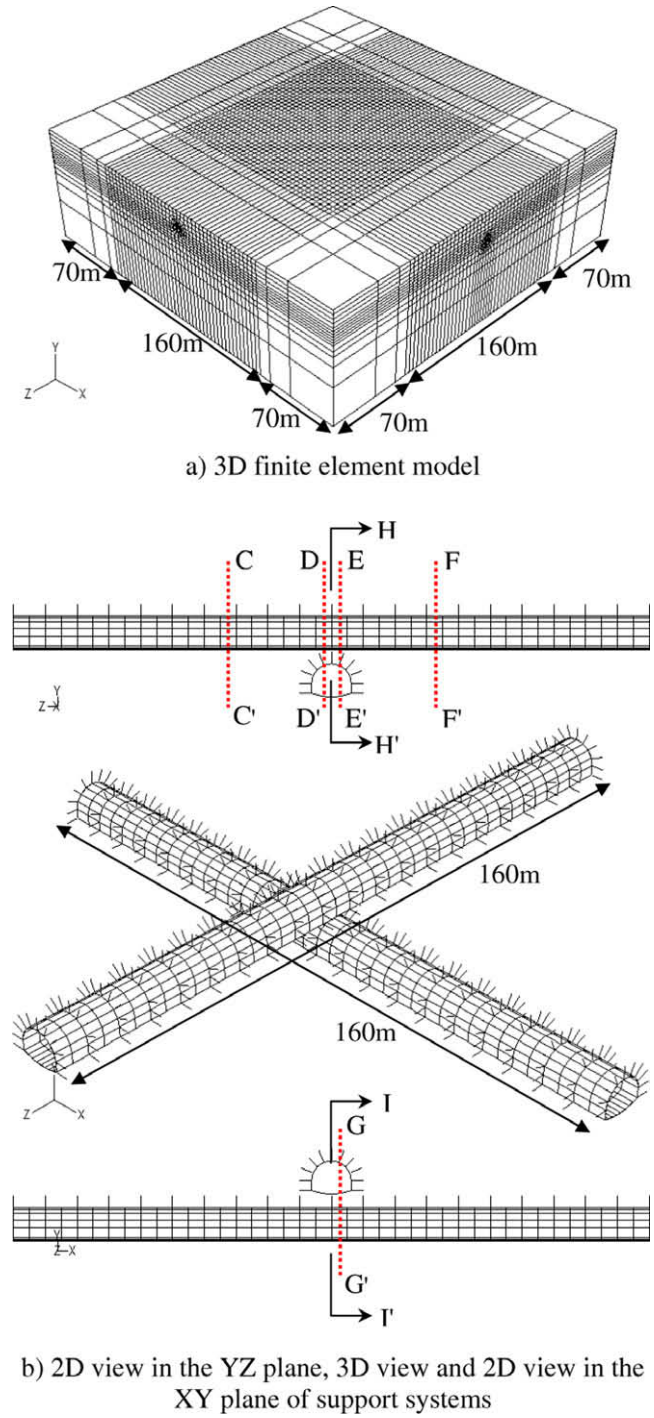


Fig. 2. Finite element model for perpendicularly crossing tunnels and their support systems (i.e. shotcrete lining and rock bolts).

make the substance parameters of the rock mass at the "tunnel scale" very low and there is no agreements on the field scale parameters of the rock mass to be adopted in practice. In this study, the behaviour of the rock mass is modelled by an elasto-plastic constitutive relationship based on the Mohr–Coulomb criterion, with a non-associated flow rule. The main physical–mechanical parameters of the rock mass are a Young's modulus $E = 200$ MPa, a Poisson's ratio $\upsilon = 0.3$, a cohesion $c = 0.5$ MPa, a friction angle $\varphi = 38°$, and a dilatancy angle $\psi = 19°$. Thus, the weaknesses in the rock mass are implicitly modelled by reducing the parameters of the rock mass, which simplifies the problem but at the same time limits this study. Moreover, it is worth noting that the rock mass is assumed to be dry and ground water is not considered in this study. The practice of shallow tunnel shotcrete lining in the Sydney region usually involves steel fibre reinforcement so that it can support the loose rock blocks in the jointed rock mass. Since the weaknesses in the jointed rock mass are implicitly considered in a simplified model used in this study, the shotcrete lining is also simplified as plain concrete without reinforcement. The behaviour

of the shotcrete lining is assumed to be governed by an elastic perfectly-plastic relationship using a Mises yield surface with a Young's modulus $E = 35$ GPa, a Poisson's ratio $\upsilon = 0.25$, and a yield stress $\sigma_{yield} = 20$ MPa. However, it will be shown in Section 3 that the shotcrete lining remains basically elastic according to the numerical simulations using this elastic perfectly-plastic model and the plastic part of the model is actually irrelevant. Thus, the behaviour of the shotcrete lining will be further assessed by comparing the modelled bending moments and axial forces with its typical capacities. The behaviour of the rock bolts is also assumed to be elastic perfectly-plastic with a Young's modulus $E = 200$ GPa, a Poisson ratio $\upsilon = 0.3$ and a yield stress $\sigma_{yield} = 400$ MPa. It is assumed that the rock mass has a mass density of 2400 kg/m^3 and by comparison that of the shotcrete lining and rock bolts is considered to be negligible and has been ignored.

Moreover, according to Pells's study [14], the regional stresses in the metropolitan area of the Sydney region can be approximately represented using the following equations:

$$\sigma_x = \sigma_{NS} = 1.5 + 1.2\sigma_V \quad \text{MPa} \tag{1}$$

$$\sigma_y = \sigma_V = 0.024H \quad \text{MPa} \tag{2}$$

$$\sigma_z = \sigma_{WE} = 0.5\sigma_x \quad \text{MPa} \tag{3}$$

where σ_V is the vertical stress, which can be calculated from the self-weight of rock mass and the depth H to any point, σ_{NS} is the horizontal regional stress parallel to the axis of the deep tunnel, and σ_{WE} is the horizontal regional stress parallel to the axis of the shallow tunnel. The external vertical boundaries of the finite element model (Fig. 2a) permit only vertical displacements. The bottom boundary is fixed in the vertical direction and the degrees of freedom in the horizontal directions are free. All of the degrees of freedom at the top surface are free, i.e. the top surface is unrestrained.

The numerical procedure developed in a previous paper [10] is used to model the tunnel construction process. ABAQUS is implemented to solve the non-linear finite element equations. TUNNEL3D is developed using Visual C++, FORTRAN and OpenGL, with the aim of it being a virtual reality system for application to tunnel engineering. TUNNEL3D is used to progressively modify the input data, control ABAQUS to perform the finite element calculations, analyse the results, as well as retrieve and graphically display the analysis results. Since the numerical procedure was described in detail in a previous paper [10], only a brief description is given here. Special attention is paid to the pre-stressing of the rock bolts because in the previous paper the rock bolts are not pre-stressed.

The tunnel construction process is modelled using a step-by-step approach. Elements in front of the tunnel face are removed to simulate tunnel excavation. In each step, the excavation length increment is $L_{exec} = 4$ m. The elements of the shotcrete lining are subsequently reactivated behind the tunnel face to simulate the support provided by the shotcrete lining. It should be noted that not all of the exposed tunnel surfaces are supported in the current step and the unsupported length is 4 m. The unsupported surfaces near the advancing face of the tunnel allow some deformations to occur before the installation of the support system. Moreover, according to Pells's study [14], rock bolts are normally used as supports in combination with the shotcrete lining in underground construction in the metropolitan area of the Sydney region. Thus, the linear elements representing the rock bolts are also reactivated behind the tunnel face in every other step in order to simulate the support provided by the rock bolts, as shown in Fig. 2b. The primary purpose of the rock bolts is to increase the stiffness and strength of the rock mass with respect to tensile and shear loads. In order for the rock bolts to significantly contribute towards resisting the failure of the rock mass without any associated appreciable displacement in the rock mass, the rock bolts are usually pre-stressed with tensile forces of between 20 kN and 60 kN applied in the Sydney region. One way to induce a pre-stress into the rock bolts is through the use of thermal contractions. After the rock bolt elements are reactivated, the temperature of these elements is reduced to generate a tensile stress of approximately 100 MPa (≈30 kN).

The entire analysis is divided into 82 analysis steps. Steps 1–42 are used to simulate the construction process of the existing tunnel so as to obtain the initial internal force and deformation fields for the existing support system. Steps 43–82 are implemented to model the driving process of the new tunnel so that the effect of this subsequent tunnelling on the existing support system can be quantified.

3. Effects of tunnelling on existing support system

This section presents the results of the interaction behaviour between two perpendicularly crossing tunnels observed when a new deep tunnel is driven beneath an existing shallow tunnel. If the bending moment tends to put the side of the shotcrete lining facing towards the tunnel opening into tension and the side facing the rock mass into compression, it is regarded as positive. Otherwise, it is negative. Positive and negative values of axial force refer to tension and compression, respectively.

3.1. Effects of tunnelling on internal forces of existing support system

During modelling, several locations, as marked in Fig. 2b, are monitored to quantify the effects of tunnelling on the existing support system.

Fig. 3 depicts the variations of circumferential bending moments monitored at the right (leading) side (refer to Fig. 1d), the crown, the left (far) side, and the invert of the shotcrete lining of the shallow tunnel, at four sectional planes (CC′, DD′, EE′ and FF′ in Fig. 2b) perpendicular to the axis of the shallow tunnel during the driving of the crossing tunnels. In order to compare and validate the results, the four sectional planes of the shallow tunnel are located symmetrically to either side of the centre of the model in the Z direction. It can be seen that as the face of the shallow tunnel advances, the bending moments gradually become stable after experiencing several loading and unloading phases. Before the driving of the deep tunnel, i.e. at Step 42, the bending moments at corresponding locations (i.e. left, crown, right and invert) of the four sectional planes have the same values, which reveal that plane-strain conditions are approximately achieved in each sectional plane and the tunnel has been constructed over a sufficient length. Moreover, the bending moments at the sides (i.e. left and right) of the shotcrete lining are equal since the model is symmetrical about the axis of the tunnel. Because of the relatively high horizontal regional stress in the X direction Eq. (1) compared with the vertical stress, the shotcrete lining at the left and right sides of the tunnel experiences tensile stresses in the side facing the tunnel opening. At the crown and invert it is compressed in the side facing the tunnel opening, which is indicated by the positive bending moments at the left and right sides and the negative bending moments at the crown and invert, respectively, as shown in Fig. 3 before Step 42. In actual tunnelling in the Hawkesbury Sandstone, Pells [14] also noted the compressive stress concentrations at the crown due to the high horizontal stress field in the Sydney region.

During the driving of the deep tunnel (i.e. from Step 42 to Step 82 in Fig. 3), different trends are observed at the different measuring points of the different sectional planes. As the face of the deep tunnel advances, the change of bending moments on the existing shotcrete lining is observed first at the right (leading) side, then

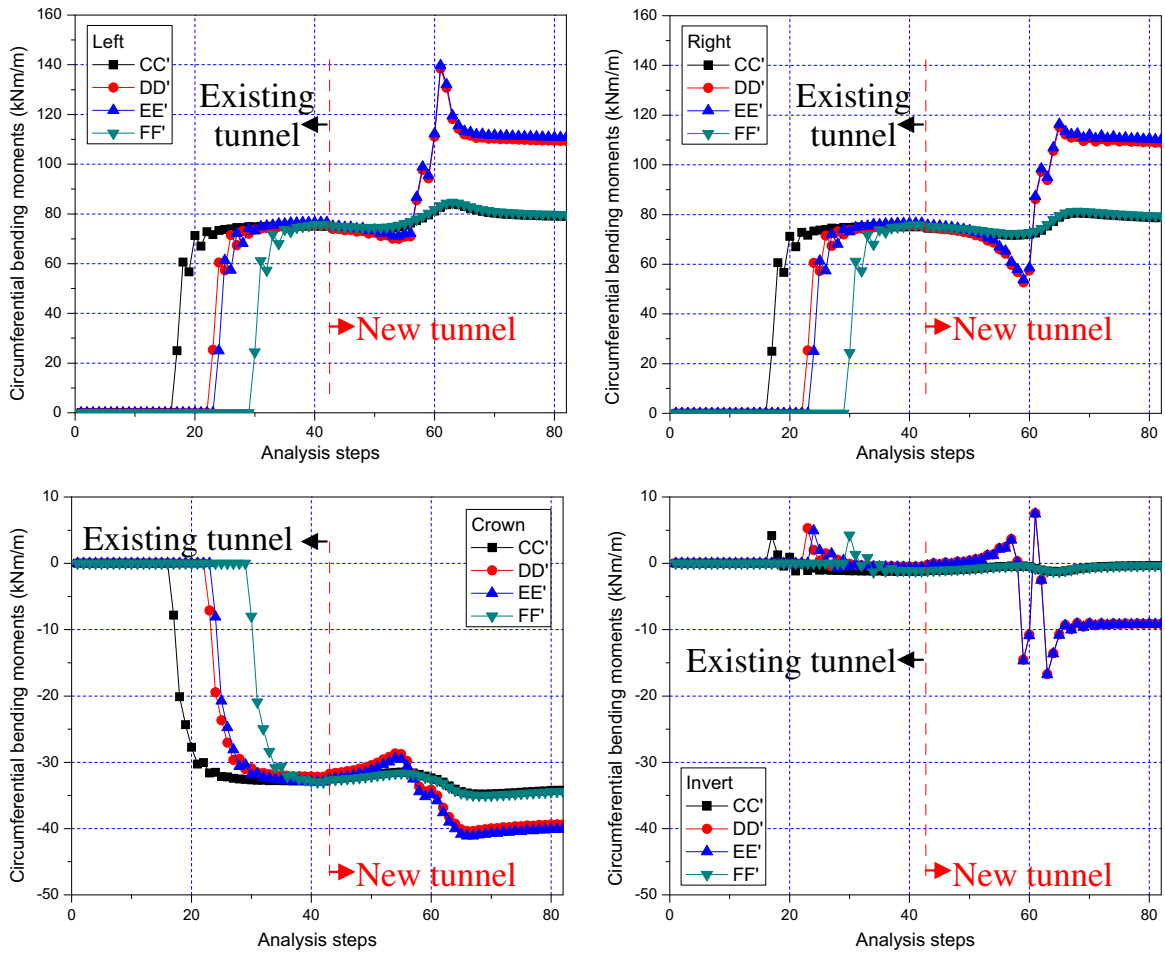


Fig. 3. Variation of bending moments at existing shotcrete lining during tunnelling (Refer to Figs. 1c and 2b for locations).

at the invert, then at the crown, and finally at the left (far) side of the shallow tunnel.

At the left and right sides of the shallow tunnel, the positive bending moment in the shotcrete lining gradually decreases as the tunnel face of the deep tunnel approaches the monitored points. The positive moment in the shotcrete lining at the right (leading) side decreases more than that at the left side. As tunnelling passes the monitored points, the positive moments gradually increase to their maximum values. It is noted that in contrast to the decrease occurring while the new tunnel face approaches the monitored points, the increase of the positive moment in the shotcrete lining at the left (far) side is larger than that at the right (leading) side. After tunnelling has passed the monitored points, the positive moments in the shotcrete lining decrease from their maximum values to stable values larger than their initial values, and finally an equilibrium stress condition is developed in the shotcrete lining.

At the crown of the shallow tunnel, the negative moments in the shotcrete lining decrease a little as the face of the deep tunnel approaches. As tunnelling passes the monitored points, the negative moments are observed to increase. At the invert, the moments change significantly as the tunnel face approaches and passes. After tunnelling has passed the monitored points, stable negative moments are developed at the crown and the invert of the shallow tunnel.

Moreover, it is found that the changes of bending moments of the existing shotcrete lining located in the crossing area (DD′ and EE′ in Fig. 3) are much larger than those in other areas (CC′ and FF′ in Fig. 3). The reason for this is that the shotcrete lining in the crossing area is closer to the tunnelling since the two perpendicular tunnels cross there. Thus, the interaction effects between two perpendicularly crossing tunnels mainly occur in the crossing area. In the areas relatively far from the crossing area, the interaction effects are not obvious, which is consistent with the 3D considerations suggesting that the magnitude of the interaction effects between perpendicular tunnels might be expected to be less than those developed between similar parallel tunnels. However, as described above, this is not the case in the crossing area.

Kim [2] performed a series of model tests to study the interaction between perpendicular tunnels in clay. The total stresses at the leading (corresponding to the "right" in this study) springline, crown, far (left) springline and invert of a shallow tunnel were measured when a deep tunnel was installed perpendicularly beneath a shallow crossing tunnel. His results indicated that as the tunnel face advanced, the stress change response was observed first at the leading (right) springline, then at the invert and crown, and finally at the far (left) springline. Thus, the behaviours of bending moments in the shotcrete lining from the present modelling shown in Fig. 3 and described above are consistent with Kim's observed variational law [2] of total stresses in physical model tests.

The variations of axial forces in the existing rock bolts (CC′, HH′ and FF′ in Fig. 2b) at the left side (refer to Fig. 1d), the crown and the right side of the shallow tunnel are depicted in Fig. 4. It can be seen that, as the face of the shallow tunnel advances, the installed rock bolts experience several loading and unloading processes and finally the axial forces become stable. During the

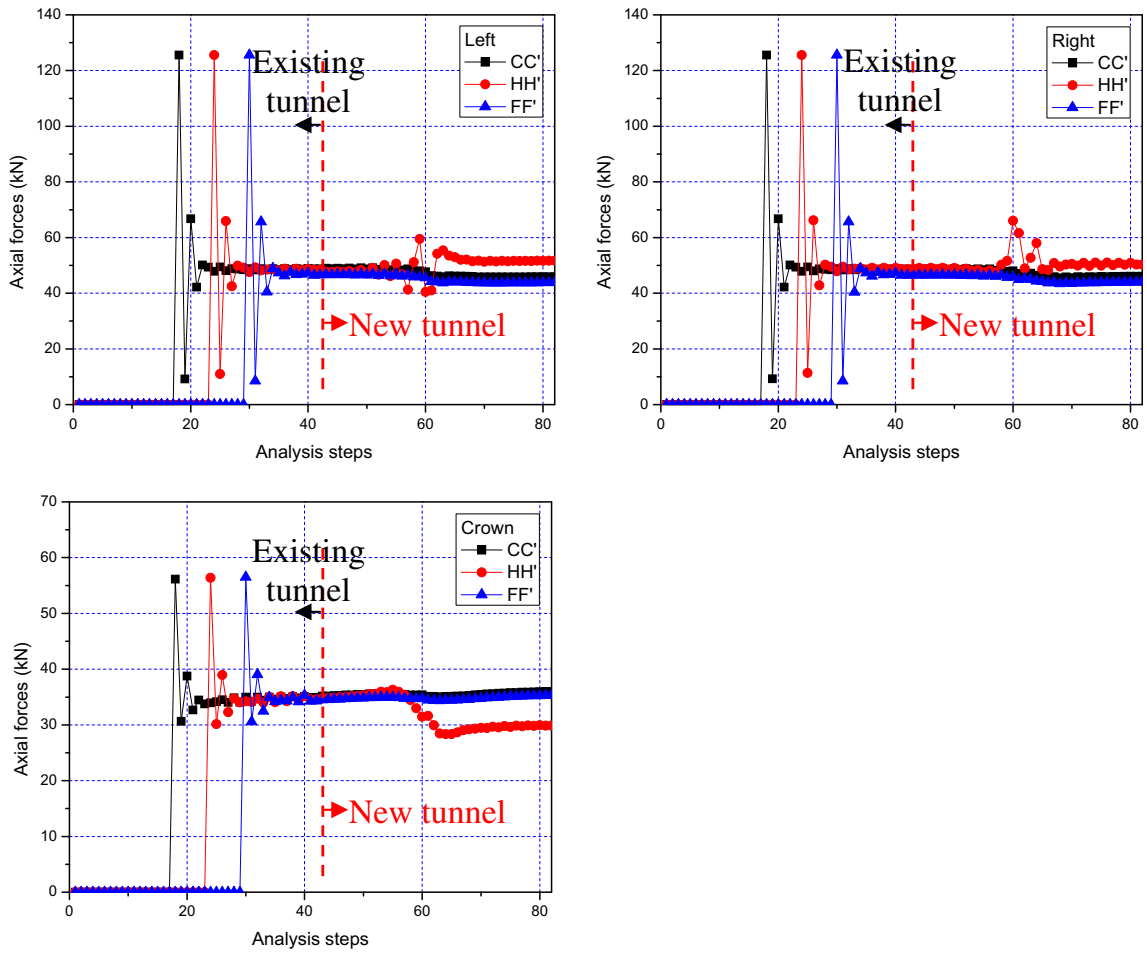


Fig. 4. Variations of axial forces at existing rock bolts during tunnelling (Refer to Figs. 1c and 2b for locations).

driving of the new tunnel perpendicularly beneath the shallow tunnel, the axial forces in the existing rock bolts change as the face of the new tunnel approaches and passes their locations. However, depending on the locations of the rock bolts and their distances from the crossing area, the axial forces change in different ways. In the crossing area, the tensile forces in the rock bolts (HH′ in Fig. 4) at the left and right sides of the shallow tunnel experience several decreasing and increasing cycles, and finally reach stable values higher than their initial values. The tensile forces in the rock bolts at the crown first increase a little, then decrease significantly, and finally reach stable values smaller than their initial values. If the rock bolts are located relatively far from the crossing area, the axial force changes are very small; the forces in the rock bolts at the sides (CC′ and FF′ in Fig. 4) decrease a little and that at the crown increase a little.

The variations shown in Figs. 3 and 4 indicate that the behaviour of the shotcrete lining and rock bolts in the crossing area is more significantly affected by the driving of the new tunnel than that at other places. In order to clarify these effects, the whole section (HH′ in Fig. 2b) of the existing support system in the crossing area is monitored and the variations are depicted in Figs. 5 and 7 for the shotcrete lining and the rock bolts, respectively. A polar coordinate system is used in Figs. 5 and 7, in which the angle approximately represents the locations of the shotcrete lining and rock bolts relative to the tunnel opening, as shown in Fig. 1d. The other coordinates are the magnitudes of the bending moments in kNm/m and the axial forces in kN/m or kN. Moreover, the modelled bending moments and axial forces in the shotcrete lining before and after the driving of the new tunnel are compared in Fig. 6, together with the typical capacity of concrete [15]. As shown in Fig. 5, the initial distributions of bending moments in the circumferential and longitudinal directions indicate that the shotcrete lining located at the sides of the tunnel is in tension (positive values) in the side facing the tunnel opening, because of the relatively high horizontal regional stresses compared with the vertical stresses. The largest bending moment in the circumferential direction in Fig. 5 is 76 kN/m, located at 11.25° and 168.75° but the circumferential axial force at those locations is relatively small (28 kN). The relationship between bending moments and axial forces shown in Fig. 6i indicates the values in the circumferential direction at those locations reach the limit of tensile cracking in the concrete if compared with the typical capacity of concrete and thus the existing shotcrete lining will start cracking at the sides. In the tunnelling practice, the shotcrete lining at this location should be locally thickened to ensure that the tunnel support system remains intact. However, during the simulations, no plasticity is observed in the shell elements of the shotcrete lining, which indicates that the elastic perfectly-plastic model with the Mises yield surface used in this study cannot model the behaviour of the shotcrete lining and a more robust material model should be developed in future studies. In the longitudinal direction, there are both relatively big bending moments and axial forces at the sides and thus the loads are still within the limit of both compressive failure and tensile cracking in the concrete according to the relationship in Fig. 6i. At other locations, the loads are also within the limit of the concrete capacity.

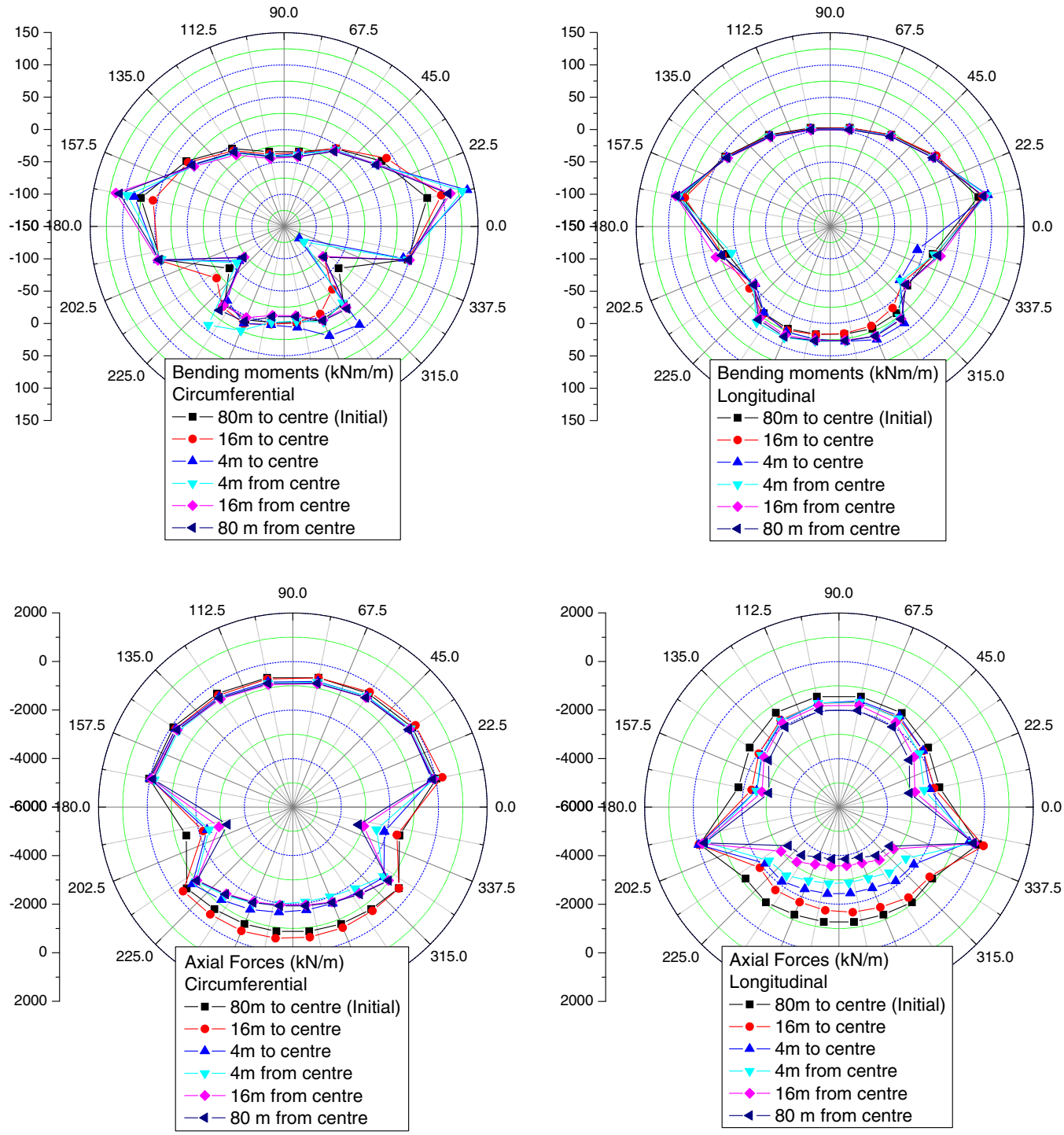
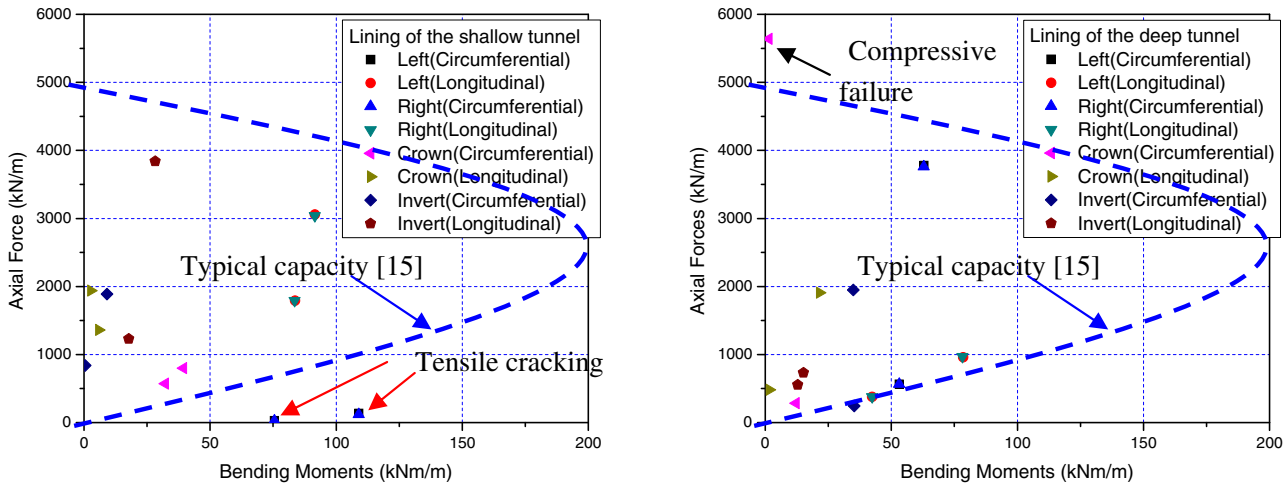


Fig. 5. Variation of bending moments and axial forces at the centre (DD′ in Fig. 2b) of existing shotcrete lining during tunnelling.

As the face of the new tunnel approaches the crossing area, the positive circumferential bending moments of the shotcrete lining located between 337.5° and 22.5° (the right side of the shallow tunnel) first increase, as do the negative circumferential bending moments of the shotcrete lining located between 315.0° and 337.5° (the right part of the invert), as shown in Fig. 5 with the face 4 m (on approach) from the centre of the model. At this stage, the circumferential bending moments show obvious asymmetrical characteristics. The relationship between bending moments and axial forces shown in Fig. 6i indicates that the increase in bending moment at the right side will cause further tensile cracking of the existing shotcrete lining there but the increase at the right part of the invert will not damage the existing shotcrete lining since the axial forces there also increase. As tunnelling passes, the negative circumferential bending moments at the crown and the positive circumferential bending moments at the left side also begin to increase, as depicted in Fig. 5 when the tunnel face is 4 m and 16 m from the centre. The increases at the left side cause further tensile cracking of the existing shotcrete lining there, but the increases at the crown do not damage the existing shotcrete lining because of the large axial forces there, as shown in Fig. 5. After tunnelling passes, the bending moments and axial forces around the shallow



i) Shallow tunnelling first followed by deep tunnelling (data from Fig. 5)

ii) Deep tunnelling first followed by shallow tunnelling (data from Fig. 12)

Fig. 6. Comparisons of modelled relationship between bending moments and axial forces and typical capacity [20] of concrete lining.

tunnel gradually come back to their approximately symmetrical shape, as shown in Fig. 5 when the tunnel face is 80 m from the centre.

Comparison of the existing shotcrete lining bending moments and axial forces in the circumferential and longitudinal directions shown in Fig. 5 indicates after the new tunnel is installed, the circumferential bending moments at the lateral sides and the longitudinal axial forces at the crown and invert increase significantly. The increase in the longitudinal axial forces at the invert is especially large. According to the relationship between bending moments and axial forces for concrete in Fig. 6i, the increase of the bending moments at the side will cause further tensile cracking of the existing shotcrete lining, but the increase of the axial force at the crown and invert will not cause compressive failure of the existing shotcrete lining there. Therefore, in order to ensure the stability of the existing tunnel, local thickening should in practice be applied at the sides of the existing shotcrete lining. Moreover, comparisons of the circumferential bending moments shown in Fig. 5 reveal that during the driving of the new tunnel, the circumferential bending moments of the existing shotcrete lining reach values larger than their corresponding final values at some locations. For example, the positive circumferential bending moments of the shotcrete lining at an angle of about 11.25° and the negative circumferential bending moments at an angle of about 326.25° in Fig. 5, with the tunnel face 4 m to/from the centre, are larger than their corresponding final values in Fig. 5 when the tunnel face is 80 m from the centre. This behaviour means that the interaction effects between perpendicular tunnels during the driving of the new tunnel may be larger than those after the new tunnel is installed. Thus, in the planning, design and construction processes of closely-spaced perpendicular tunnels in urban environments, special attention should be paid to the tunnelling process. If a new tunnel is constructed in close proximity to an existing tunnel, the support system of the existing tunnel

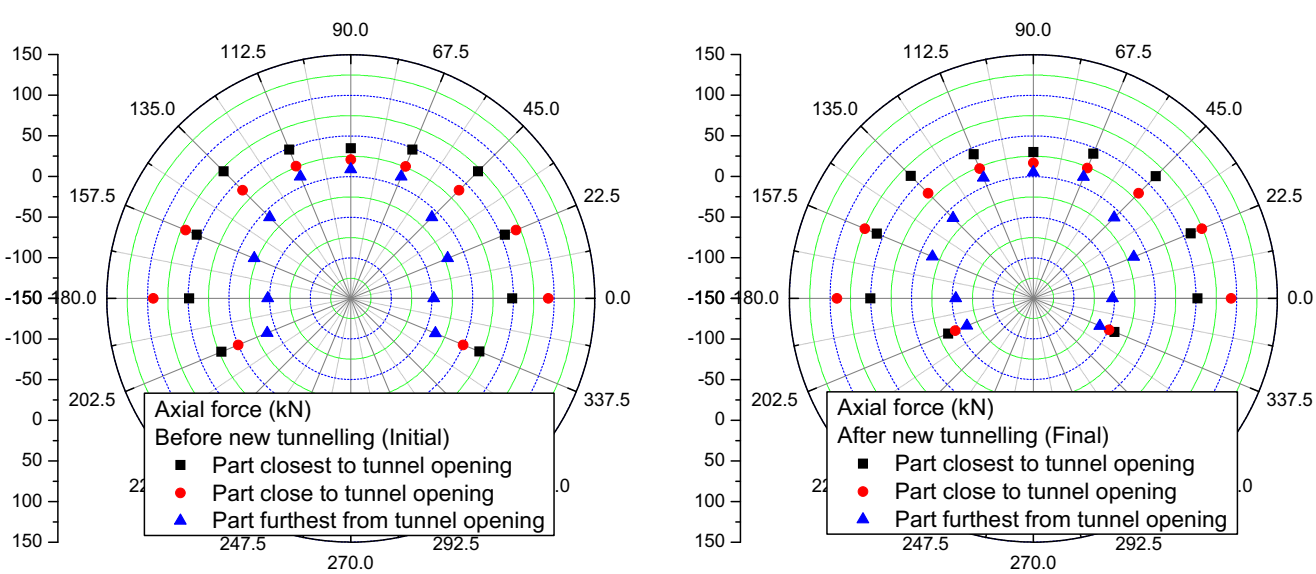


Fig. 7. Variation of axial forces at the centre (HH′ in Fig. 2b) of existing rock bolt during tunnelling.

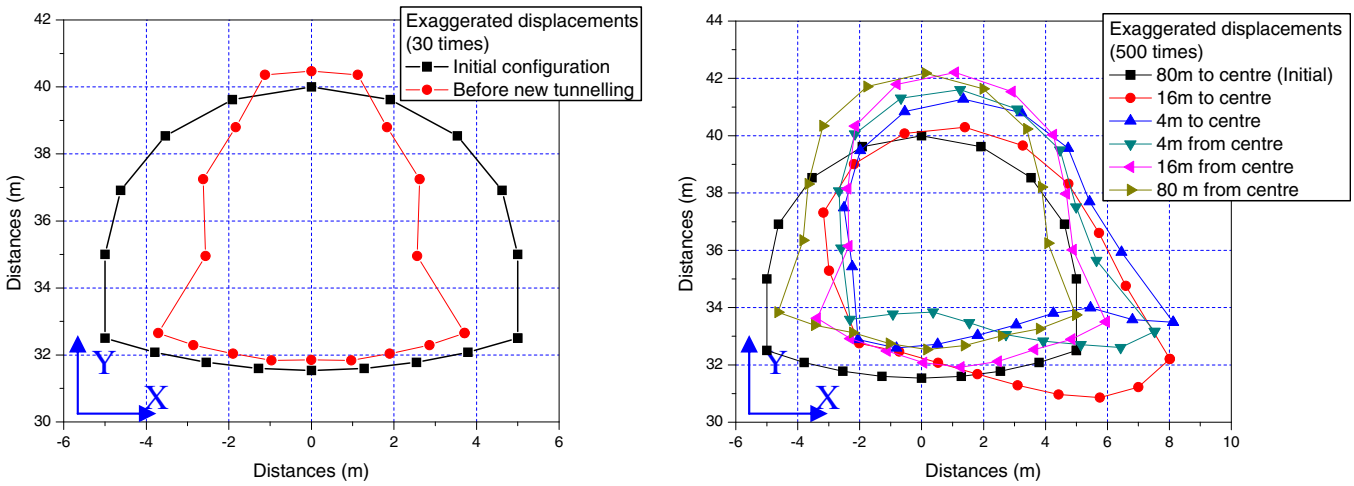


Fig. 8. Variation of exaggerated displacements at the centre (HH′ in Fig. 2b) of existing support system during tunnelling.

may not have been designed to withstand the resulting interaction effects. In this case, the additional loads induced by adjacent tunnel construction can lead to damage to the support system. In this example, during the driving of the deep tunnel, the increase of bending moments at the sides of the existing shotcrete lining causes tensile cracking there. It may, therefore, be necessary to strengthen the existing support system, such as by local thickening of the shotcrete lining at the sides, before the driving of the adjacent tunnel.

Fig. 7 shows the effect of driving the new tunnel on the axial forces in the existing rock bolts in the crossing area. Since the force variations shown in Fig. 3 indicate that the force changes in the existing rock bolts are much simpler compared with those in the existing shotcrete lining; for brevity, only the initial and final forces in the existing rock bolts are shown in Fig. 7. Before the driving of the new tunnel, most sections of the existing rock bolts are in tension, with the bolt sections closest to the tunnel opening under highest tension. After the driving of the new tunnel, major changes occur in the forces in the existing rock bolts located near the invert.

3.2. Effects of tunnelling on deformation of existing support system

The deformation of the existing support system around the shallow tunnel in the crossing area (HH′ in Fig. 2b) before the driving of the new deep tunnel is shown exaggerated 30-fold in Fig. 8. It should be noted that the deformation shown in Fig. 8 before driving the new deep tunnel is actually the deformation of the rock mass around the shallow tunnel, which includes the deformation of the support system. The support system is installed after a certain deformation has already occurred in the rock mass around the tunnel opening. Thus, the actual deformation of the support system after installation of the shallow tunnel is much smaller than that shown in Fig. 8. It can be seen that after the installation of the shallow tunnel, the sides of the tunnel move inward, while the crown and the invert heaves upwards because of the relatively high horizontal regional stress compared with the vertical stresses in the Sydney region. This is explained in more detail in a previous paper by comparing the results obtained in low and high regional stress regimes [16] and will briefly be discussed later in Section 4.3. Moreover, it is noted that the displacements at the crown and invert of the shallow tunnel are not uniform, which supports the assumption of the oval-shaped ground deformation pattern around a tunnel section [17] used in the empirical/analytical analysis to calculate the ground loss.

At the start of the driving of the new deep tunnel, the deformation caused by the driving of the existing shallow tunnel is reset to zero in the analysis. Thus, the calculated deformation of the existing support system around the shallow tunnel during the driving of new deep tunnel is actually the incremental deformation (which is exaggerated by 500-fold in Fig. 8) induced only by the driving of the new tunnel. As the face of the deep tunnel advances, the support system is pulled towards the new excavation, as shown in Fig. 8 with the tunnel face 16 m to the centre, i.e. the shotcrete lining at the right side of the shallow tunnel is pulled outward from the tunnel opening and that at the left side is pulled inward. As the deep tunnel passes the shallow tunnel (i.e. the tunnel face is 4 m to/from the centre in Fig. 8), the displacement in the vertical direction changes significantly; on advance, the shotcrete lining at the invert moves inward towards the tunnel opening and that at the crown moves further outward. As the deep tunnel passes, the shotcrete lining gradually moves back to its initial position in the horizontal direction (Fig. 8 with the tunnel face 16 m and 80 m from the centre). It can be seen that after the driving of the new deep tunnel, the existing support system moves further inward at the sides, and the invert and the crown undergo further heave. During the driving of the deep tunnel, the deformation of the shotcrete lining at some locations (e.g. the right side) during the driving process is bigger than that after the new tunnel is completed.

Kim [2] monitored the radial displacements at the leading (right) springline, crown, far (left) springline and invert of the shallow tunnel in his physical model tests when a new tunnel was installed beneath the crossing shallow tunnel. The two springlines were found to move outward from the tunnel centre, and the crown and the invert were observed to move downward due to the installation of the new tunnel. However, in our modelling, it is found that the sides move inward, and the crown and the invert heave after the driving of the new tunnel. We believe that the differences are caused by the relatively high horizontal regional stress compared with the vertical stress in the Sydney region ($K_{0-NS} = 5.37$ and $K_{0-WE} = 2.69$ in Eqs. (1)–(3)) used in this study. In Kim's physical tests [2], the regional stress regime is $K_0 < 1$ (approximately 0.64), i.e. the vertical stress is bigger than the horizontal stress, which causes the existing tunnel to settle down due to the excavation of the new tunnel. Moreover, Kim's results [2] indicated that the displacement interactions were larger at the crown and the invert, and were smaller at the lateral springlines, which is consistent with our modelling results shown in Fig. 8 with the tunnel face 80 m from the centre. The consistency between the

numerical results and physical observations is reasonable since the new tunnel is excavated at different depths beneath the existing tunnel in the two cases. If the new tunnel is driven at the same level as the existing tunnel, it is expected that the displacement interaction would be larger at the lateral springlines than that at the crown and invert.

4. Discussions

The modelling results presented in Section 3 provide important information about the effect of new tunnelling on the support system of an existing tunnel during the driving of perpendicularly crossing tunnels. In this section, various aspects related to the interaction mechanisms that are mobilised are discussed.

4.1. Influence of finite element meshes (element type and mesh coarseness)

It is generally accepted that finite element modelling results are affected by the finite element mesh adopted. In order to investigate the influence of the finite element mesh on the results presented in Section 3, another two models, including a single tunnel with the same cover depth as the shallow tunnel in Fig. 2, are considered. As shown in Fig. 9, model A is meshed using either linear or quadratic elements and model B is meshed using linear elements. The linear elements in the finite element model are the same as those described in Section 2. In the model using quadratic elements, the rock mass, the shotcrete lining, and the rock bolt are modelled using twenty-noded hexahedral elements with reduced integration points (C3D20R), eight-noded quadrilateral elements with reduced integration points (S8R), and three-noded quadratic truss elements (T3D3), respectively.

The tunnel in Fig. 9 is then excavated and supported according to the procedure described in Section 2. After installation, the bending moments and axial forces calculated at the centre of the existing shotcrete lining are compared with those obtained in Section 3 (before the driving of the deep tunnel) since the tunnel in Fig. 9 has the same cover depth as the shallow tunnel in Fig. 2. The comparisons shown in Fig. 10 indicate that the fine mesh in Fig. 2 and the coarse mesh in Fig. 9a using linear elements produce almost identical bending moments and axial forces at the centre of the existing shotcrete lining. Thus, the model in Fig. 2 is considered to be adequate. However, the results calculated using the linear (Fig. 2 and Fig. 9a) and quadratic (Fig. 9a) meshes with the same density show large variations in bending moments and axial forces with sharp transitions at the sides. Fortunately, as the density of the linear mesh increases (Fig. 9b), the linear and quadratic meshes predict similar bending moments and axial forces (for the quadratic element with several integration points, the values are interpolated at the centre of the element). Therefore, the eight-noded hexahedral element with reduced integration points and hourglass control is very useful for the tunnel modelling. The reduced integration scheme ensures that the shear locking phenomena does not occur and the analysis can be completed with acceptable computational time and resources. Moreover, the occurrence of the zero-energy mode, which usually happens in the eight-noded hexahedral element, is avoided by the hourglass control.

4.2. Achievement of steady-state on existing support system

During the driving of tunnels, the rock mass surrounding the excavated area will deform and create a local 3D state of stress around the excavated area. As the tunnel face advances step-by-step, the installed support system receives an additional loading at each excavation step. The additional loading gradually decreases as the tunnel face progresses. Thus, at a sufficient distance behind the tunnel face, the support system at each tunnel section will experience nearly the same stress. Such a state of stress is called the plane-strain condition. In the study on the interaction between perpendicular tunnels, the plane-strain condition must be approximately achieved at each tunnel, or at least at the monitored locations of each tunnel, otherwise, the monitored data cannot represent the complete interaction behaviour.

From the point of view of achieving steady-state, each tunnel should be constructed over as long a distance as possible, which will result in a very big finite element model. However, there are limitations ([12,18]) in terms of computational time and resources in increasing the number of elements in the finite element mesh.

Fig. 11a shows a plot of the development of settlement/heave profiles in the longitudinal direction of the shallow tunnel (Line AA′ in Fig. 1a) during the driving of the shallow tunnel. It can be seen from Fig. 11a that a horizontal plane of settlement/heave

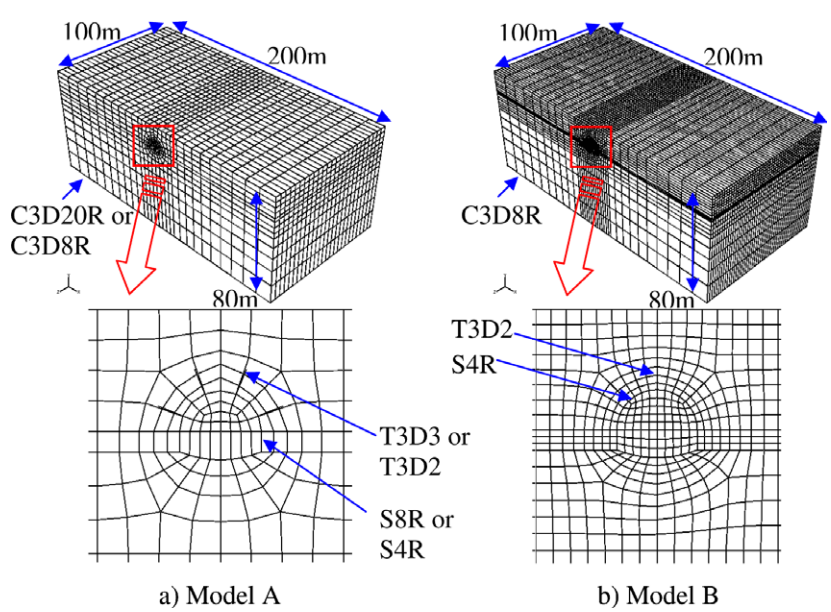


Fig. 9. Finite element models with different meshes.

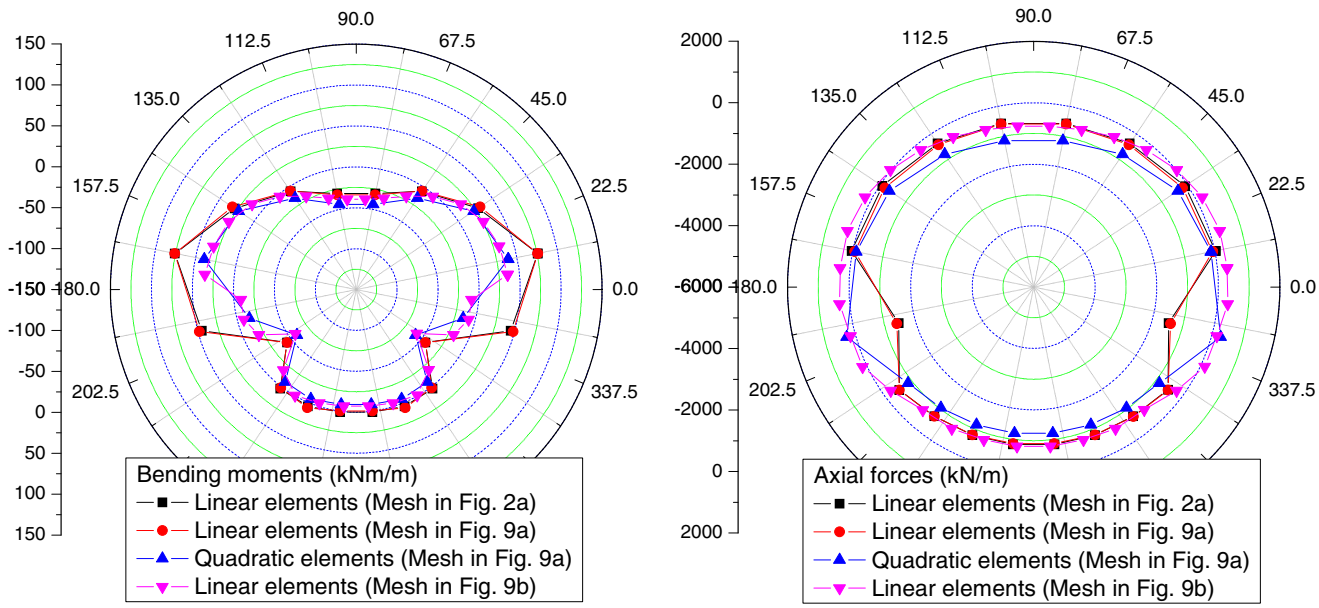


Fig. 10. Influences of finite element meshes (element types and mesh coarseness) on bending moments and axial forces at the centre of existing shotcrete lining.

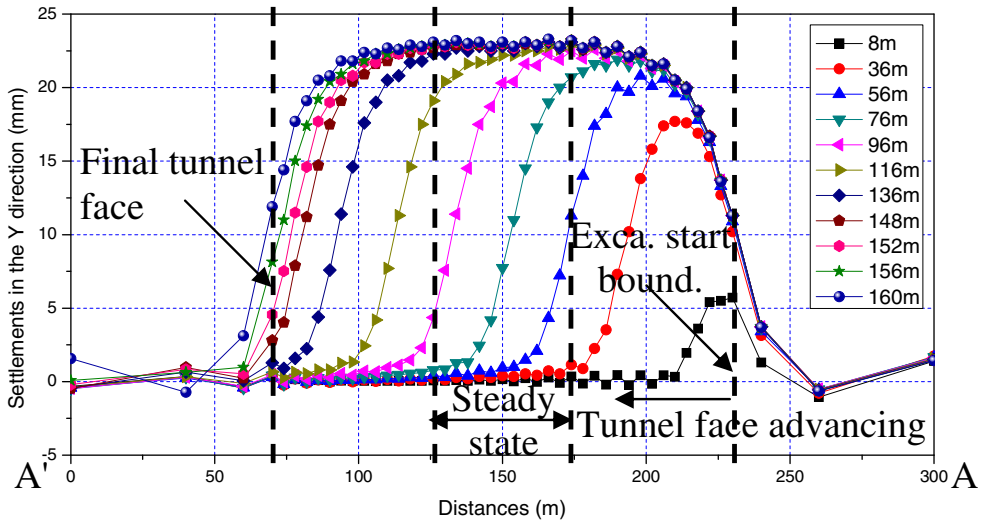
develops approximately 56 m (5.6*D*) from the excavation start boundary, i.e. 40 m (4*D*) behind the tunnel face, after the shallow tunnel is driven for a length of 96 m (9.6*D*). For a tunnel with a diameter $D = 4.15$ m, Franzius and Potts [18] reported that the steady-state settlement developed approximately 20 m (4.8*D*) from the excavation start boundary, i.e. 30 m (7.2*D*) behind the tunnel face, when the tunnel face had advanced 50 m (12.0*D*). For a tunnel with a diameter $D = 8$ m, Vermeer et al. [19] concluded that steady-state settlement was established approximately 40 m (5*D*) from the excavation start boundary, i.e. 40 m (5*D*) behind the tunnel face, when the tunnel face had advanced 80 m (10*D*). The comparisons indicate that our modelled distance (5.6*D*) from the excavation start boundary is larger than that predicted by Franzius and Potts (4.8*D*), and Vermeer et al. (5*D*), which may be related to the high stress regime in this study. Our modelled distance (4*D*) behind the tunnel face is smaller than that (7.2*D*) predicted by Franzius and Potts but is close to that (5*D*) predicted by Vermeer et al. In any case, Fig. 11a shows that steady-state settlement/heave has developed around the centre of the model, which is marked in Fig. 11a. Moreover, the monitored bending moments shown in Fig. 3 have also indicated that the plane-strain condition has been approximately achieved at the four sectional planes (CC′, DD′, EE′ and FF′ in Fig. 2b) of the existing support system. Thus, the bending moments, axial forces and deformations presented in Section 3 should represent the actual behaviour of the existing support system installed before the driving of the new tunnel.

In order to evaluate the complete interaction behaviour between perpendicular tunnels, the new tunnel should be driven for such a length that the start of the driving has almost no effect on the existing tunnel, affects it as the new tunnel face approaches and passes it, and finally has almost no affects at the end of driving. Fig. 11b shows the longitudinal development of settlement/heave profiles (Line BB′ in Fig. 1a) during the driving of the new tunnel. Due to the existence of the shallow tunnel, the settlement/heave profile presents different characteristics compared with that shown in Fig. 11a; being convex around the centre of the model. Because of the relative low stress regime in the *Z* direction Eq. (3) compared with that in the *X* direction Eq. (1), it seems that the steady-state can be more easily achieved in the longitudinal direction of the new tunnel and the heave is much smaller than that in Fig. 11a. Before the new tunnel face approaches the existing tunnel, the heave profile is approximately a flat plateau like that in the free-field, which indicates that the interaction is not significant during this stage. As the new tunnel face approaches and passes the existing tunnel, additional heave of the existing tunnel occurs, which results in the convex heave profile at the ground surface in the longitudinal direction of the new tunnel. The convex heave profile is actually caused by the existence of the shallow tunnel. Thus, the interactions between two perpendicular tunnels are obvious. As the new tunnel face moves away, the heave profile returns to the approximately flat plateau profile and the interaction behaviour between tunnels becomes stable. Therefore, the calculated settlement/heave profile during the driving of the new tunnel indicates the complete interaction behaviour between two perpendicular tunnels modelled in this study. The variation shown in Fig. 3 further supports this idea, since the monitored final bending moments become stable as the new tunnel face advances. Thus, the bending moments, axial forces and deformations presented in Section 3 should represent the complete response of the existing support system to the driving of the new tunnel.

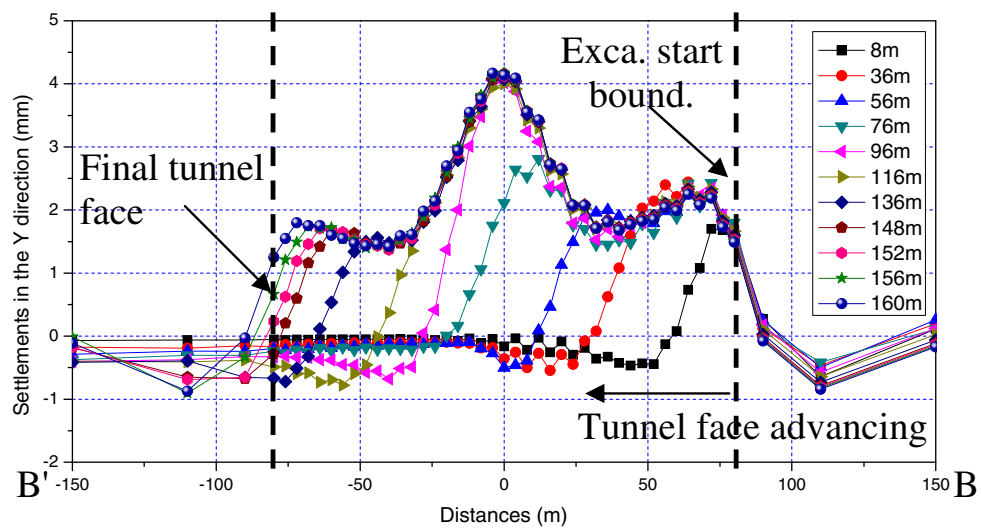
4.3. Effects of tunnelling procedures

In previous sections, the interaction between perpendicular tunnels is investigated by focusing on the effects of deep tunnelling on the support system of an existing crossing shallow tunnel. In this section, the alternative sequence of the deep tunnel being installed first and the crossing shallow tunnel being installed subsequently is modelled.

Fig. 12 compares the initial and final bending moments and axial forces at the centre (II′ in Fig. 2b) of the support system of the existing deep tunnel before and after the driving of the shallow tunnel. Similar to the bending moments shown in Fig. 5, the shotcrete lining facing towards the deep tunnel opening is in tension at the sides before the driving of the new tunnel. However, due to the relatively low horizontal regional stress in the *Z* direction Eq. (3) compared with that in the *X* direction Eq. (1), the positive bending moments at the sides in Fig. 12i are much smaller than those in Fig. 5. The comparison of the modelled relationship between



a) During driving of shallow tunnel

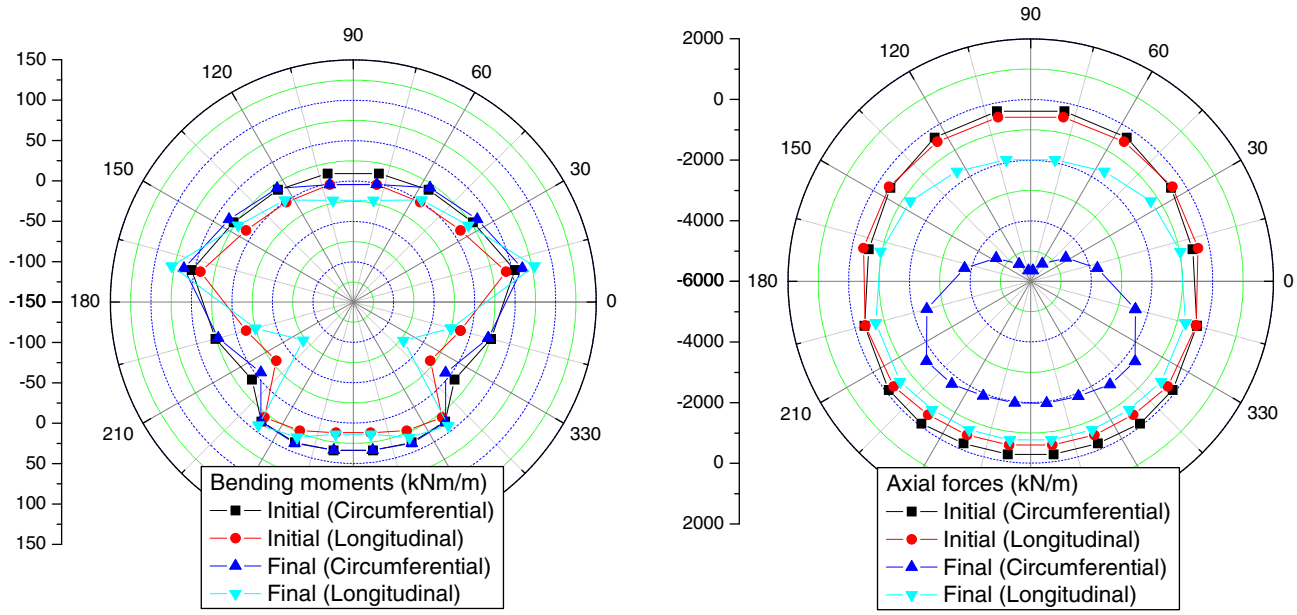


b) During driving of deep tunnel

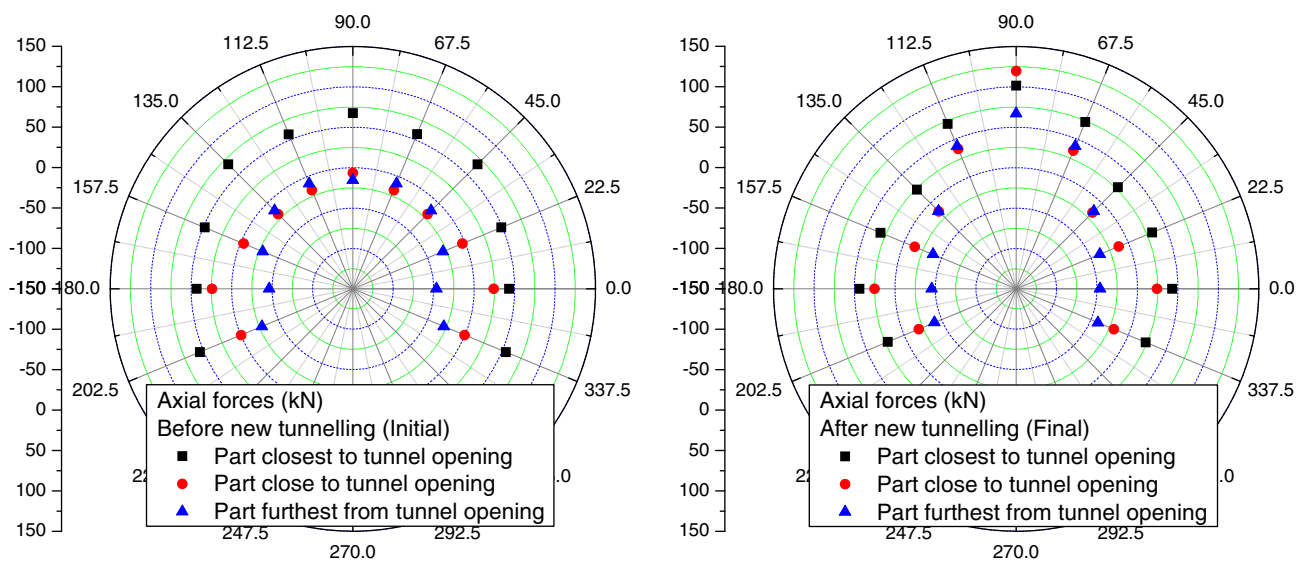
Fig. 11. Developments of settlements/heave in vertical direction at surface (AA′ and BB′ in Fig. 1a) during tunnelling.

bending moments and axial forces and the typical capacity of the concrete depicted in Fig. 6ii reveals that the loads at the sides of the shotcrete lining are at the limit of tensile cracking for concrete. The initial axial forces in the rock bolts shown in Fig. 12ii are also similar to those in Fig. 7; the sections of the rock bolts closest to the tunnel opening are under higher tension although their exact values are different due to the different regional stress regime and the different cover depth. The driving of the new shallow tunnel leads to an increase in the bending moments at the sides of the existing shotcrete lining and an increase in the axial forces at the crown and invert, as shown in Fig. 12i. The largest increase is that of the axial force in the circumferential direction at the crown, since the crown is closest to the shallow tunnel. The relationship between bending moment and axial force in the concrete in Fig. 6ii indicates that in spite of the increases, the loads on the shotcrete lining are still at the limit of both compressive failure and tensile cracking of concrete, except for those at the crown, where the increase in the axial force causes the loads at the crown of the existing shotcrete lining to exceed the limit of compressive failure in the concrete. Thus, local thickening of the existing shotcrete lining is needed at the crown to ensure the stability of the existing tunnel. Besides, due to the driving of the new tunnel crossing over the deep tunnel, the tensile forces in the existing rock bolts closest to the tunnel opening at the sides decrease slightly and those at the crown increase significantly. In the case of shallow tunnel installation followed by deep tunnel driving, the bending moments at the sides and the axial forces at the crown and invert of the existing shotcrete lining of the shallow tunnel are also both observed to increase, but the largest increase is located at the invert (Fig. 5). The load increase at the sides would cause the existing shotcrete lining to undergo further cracking there, but the load increase at the crown and invert would not damage the existing shotcrete lining at that location. The response of the rock bolts of the existing shallow tunnel to the driving of the new deep tunnel is mainly characterised by the compression of the rock bolts near the invert (Fig. 6). Thus, the interaction effects between perpendicularly crossing tunnels also depends on the construction sequence.

In summary, if the shallow tunnel is constructed first and the deep tunnel subsequently, the construction of the shallow tunnel perpendicular to the major principal regional stress in the Sydney region causes tensile cracking at its sides and the subsequent driving of the deep tunnel perpendicularly crossing beneath the



i) Shotcrete lining



ii) Rock bolts

Fig. 12. Effects of tunnelling on bending moments and/or axial forces at the centre (II′ and GG′ in Fig. 2b) of existing support system if deep tunnelling first followed by shallow tunnelling.

shallow tunnel worsens the situation. Thus, on the basis of this analysis, the existing shotcrete lining at the sides of the shallow tunnel should be thickened before the driving of the deep tunnel. If the deep tunnel is constructed first and the shallow tunnel subsequently, the construction of the deep tunnel perpendicular to the middle principal regional stress induces neither tensile cracking nor compressive failure, but the subsequent driving of the shallow tunnel perpendicular to the major principal regional stress causes compressive failure at the crown of the deep tunnel, which indicates that local thickening of the existing shotcrete lining is needed at the crown of the deep tunnel before the driving of the shallow tunnel. Moreover, according to this study, the construction of tunnels perpendicular to the major principal regional stress should be avoided in the region such as Sydney with high horizontal stresses, since their support systems will probably be damaged due to the major principal stress. This has been validated by the field observations in the Northside Storage Tunnel Project in Sydney [20], which concluded that the stress testing carried out during the investigations suggested that the ratio of horizontal to vertical stress was generally in the order of 2–5, with the highest horizontal component in an approximately north–south direction, and failures resulted from these high horizontal stresses in regions where the tunnel orientation was unfavourable (i.e. east–west). At the depth of the shallow tunnel ($H = 15$ m) in this study, the ratio between the major horizontal regional stress and the vertical stress is 5.37 according to the formula in Eqs. (1)–(3).

The displacements of the support system (II′ in Fig. 2b) around the existing deep tunnel opening before the driving of

the shallow tunnel are shown in Fig. 13. It can be seen that the sides of the existing support system move inward, similar to that shown in Fig. 8 when the shallow tunnel is installed first and the deep tunnel subsequently, although the movements are much smaller in this case due to the relatively low regional stress regime in the Z direction Eq. (3). The crown has moved down marginally, but the invert has heaved substantially, which is different from the deformed shape shown in Fig. 8 in which both the crown and invert have heaved. The comparison between Figs. 8 and 13 further reveals that the crown heave in Fig. 8 is a consequence of the high horizontal regional stress used in this study. The axis of the shallow tunnel is perpendicular to the direction of the maximum horizontal stress Eq. (1), which is much higher than the vertical stress Eq. (2) and this causes heave at the crown of the shallow tunnel (Fig. 8). The axis of the deep tunnel is perpendicular to the direction of the minimum horizontal stress Eq. (3), which is still larger than the vertical stress Eq. (2) but significantly smaller than the maximum horizontal stress Eq. (1). Thus, the crown of the deep tunnel only settles slightly (Fig. 13). Moreover, this phenomenon is observed in tunnelling practice in the Sydney region; the excavation of the access tunnel to Opera House Carpark cavern with a span of 10 m induced only approximately 0.5 mm of settlement at the crown due to the relatively high horizontal stress compared with the vertical stress [14]. Similar phenomena were also found in the field observations in the Northside Storage Tunnel Project in Sydney [20]. However, rigorous searches of available ground surface measurements in tunnelling practice indicates few of them concluded that the ground surface heaved, although almost all of them commented that due to the relatively high horizontal regional stresses, very small settlements were induced in spite of the large-span tunnel excavation. We believe that there are at least two reasons to explain the difference between the numerical simulations and field observations: (1) almost all of the ground surface measurements are made on at locations where there are sensitive structures (however, in the numerical simulation, the surcharge of the surface structures, which may contribute to the settlements, is not considered) and, (2) in the tunnelling practice, few tunnels are placed perpendicular to the major horizontal regional stress. However, in the numerical simulation, the extreme conditions are considered.

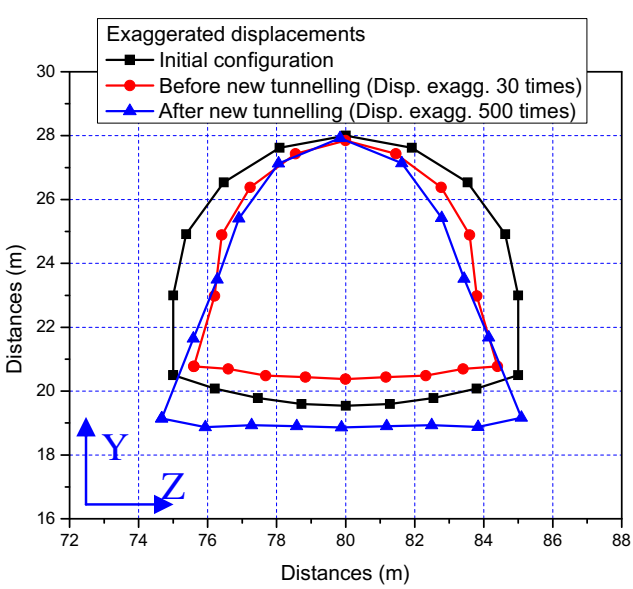


Fig. 13. Effects of tunnelling on deformations at the centre (II′ in Fig. 2b) of existing support system if deep tunnelling first followed by shallow tunnelling.

After the driving of the new tunnel, the incremental displacements of the existing support system around the deep tunnel opening shown in Fig. 13 reveal that the shotcrete lining is distorted in different patterns compared with those (Fig. 8) induced when the shallow tunnel is installed first and the deep tunnel subsequently. In the case of the deep tunnel installation first followed by the shallow tunnel, the incremental displacements of the shotcrete lining of the existing deep tunnel are concentrated mainly at the sides and the invert. The sides of the deep tunnel move inwards and the invert moves downwards. In the case of the shallow tunnel installation first followed by the deep tunnel, the crown and invert move upwards causing an upward movement of the whole cross-section of the shallow tunnel because of the relatively high horizontal stress. Moreover, comparisons between these two cases reveal that the displacement interaction effects seem to be much larger when the deep tunnel is installed beneath the shallow tunnel than those when the shallow tunnel is installed over the deep tunnel. Kim [2] conducted model testing to investigate the interaction between perpendicular tunnels. He concluded that in the case of lower tunnel installation first, the incremental displacement of the shotcrete lining of the lower tunnel were similar to the parallel tunnel case, i.e. the two springlines moved outward and the crown moved downward. Our predicted incremental displacement of the existing shotcrete lining is also somewhat similar to the parallel tunnel case in the high regional stress regime [10], i.e. the sides move inward and the invert moves downward. Thus, it is reasonable to conclude that our predictions are consistent with Kim's model tests [2], although the shotcrete lining actually moves in different directions due to the different stress regimes in our numerical modelling Eqs. (1)–(3) compared with Kim's model test ($K_0 < 1$).

Therefore, the interaction mechanisms between perpendicular tunnels observed when a new tunnel is constructed passing over an existing tunnel are different from those obtained when the new tunnel is installed beneath an existing tunnel, which reveals that the interaction effects between perpendicular tunnels may be minimised by careful choice of the construction sequence.

5. Conclusions

It is generally recognised that the interaction effects between adjacent tunnels are complex, especially those between perpendicularly crossing tunnels, which cannot be investigated using the traditional two-dimensional (2D) plane-strain methods. In this study, the interactions between closely-spaced perpendicularly crossing tunnels in the Sydney region, with a high stress regime, are investigated using a full three-dimensional (3D) finite element analysis coupled with elasto-plastic material models. Special attention is paid to the effect of subsequent tunnelling on the support system, i.e. the shotcrete lining and rock bolts, of the existing tunnel. For this particular study, the following conclusions can be drawn:

(1) The existing support system in the crossing area is affected first at the leading side, then at the invert, after that at the crown, and finally at the far side as the underlying tunnel face advances, but relatively far from the crossing area remains almost unchanged during new tunnelling. Moreover, it is found that the interaction effects between perpendicularly crossing tunnels during tunnelling are larger than those after the installation is finished and thus it is important to investigate the tunnelling process.
(2) Due to the relatively high stress regime in the Sydney region, the existing shotcrete lining at the sides is placed in tension in the side facing towards the tunnel opening and in compression at the crown and invert. The driving of the new tunnel perpendicularly beneath the existing tunnel results in a

significant increase of the bending moments at the sides and the axial forces at the crown and invert. The load increase at the sides of the tunnel causes the existing shotcrete lining to undergo further cracking there, but the increased load at the crown and invert is still within the limits of both compressive failure and tensile cracking of concrete if compared with its typical capacity. Moreover, the driving of the new tunnel causes the tensile forces in the existing rock bolts at the sides of the shallow tunnel to increase, but those at the crown to decrease. Therefore, on the basis of the calculated results, local thickening is needed at the sides of the existing shotcrete lining to ensure that the existing support system remains intact during the driving of the new tunnel.

(3) The existing support system moves further inward at the sides and heaves further at the crown and invert when a new tunnel is driven beneath an existing perpendicularly crossing tunnel, in a relatively high stress regime such as exists in Sydney.

(4) The driving of the new tunnel perpendicularly above the existing tunnel causes compressive failure of the existing shotcrete lining at the crown for the typical capacity of concrete and the significant tensile force increase in the existing rock bolts around the crown. According to these results, the existing shotcrete lining at the crown should be thickened before the driving of the new tunnel, which is different from the interaction effect obtained when a new tunnel is installed beneath the existing tunnel. Thus, the interaction effects between perpendicularly crossing tunnels may be minimised by the careful choice of construction sequences.

However, it must be pointed out that there are several major limitations in this study: (1) the effect of groundwater is ignored, (2) the material model used for the shotcrete lining fails to model its tensile cracking and compressive failure and, (3) weaknesses such as bedding discontinuities, joints and faults existing in the Hawkesbury sandstone are not well modelled. Further studies are needed in these areas.

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