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Effects of Tunnelling on Existing Support Systems of Intersecting Tunnels in the Sydney Region

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Abstract

Full three-dimensional (3D) elasto-plastic finite element analyses are conducted to investigate the effects of tunnelling on existing tunnel support systems, i.e. shotcrete lining and rockbolts, of orthogonally intersecting tunnels in the Sydney region with a high horizontal regional stress regime. It is found that the zone of influence extends about 2.5 tunnel diameters (2.5D) from the intersection. During driving of the new tunnel, the existing support system in the zone of influence is affected according to the progression of the new tunnel face to/from the existing tunnel and the effects are not significant when the new tunnel face is located 4.5D from the intersection. The approach of the new tunnel face towards the existing tunnel causes the existing shotcrete lining on the approach side of the existing tunnel to undergo tensile cracking, while neither further compressive failures nor further tensile cracking is observed when the new tunnel is driven away from the existing tunnel. It is suggested that temporary reinforcement should be installed inside the zone of influence during approach and local thickening of the shotcrete lining of the existing tunnel is necessary in locations immediately adjacent to the intersection.

1 Introduction

Largely because of the need to develop more efficient and environmentally friendly infrastructure, the number of constructed tunnels has been gradually increasing during past decades. For example, most major cities in the world now have underground metro systems, which have required the construction of numerous tunnels. Moreover, according to studies from the International Tunnelling Association, many more tunnels will be required in the coming decades for the purpose of transport, public utilities, city centre revitalisation, storage, etc. In this case, it is highly likely that some new tunnels may need to be constructed so that they intersect with existing tunnels.

When two tunnels intersect, the zone of intersection necessarily has a 3D structural configuration. Moreover, the ground behaviour around the tunnel intersection will be different from that around a single tunnel because the secondary states of stress induced in the ground by each tunnel excavation will interfere (or interact) with each other. High stress concentrations and the occurrence and expansion of any unstable zones can be expected in the zone of intersection. For example, in the Lane Cove Tunnel Project conducted in Sydney, the MC5B/MCAA intersection between the ventilation tunnel (MC5B) and the Pacific Highway Exit Ramp tunnel (MCAA) partly contributed to the ground collapse that occurred in November 2005. This collapse caused the roadway above the area to subside and damaged a building in close proximity to the area of the subsidence (WorkCover, 2005). Clearly, in the design and construction of any tunnel intersection, it is important to understand the size of the zone of influence of the intersection and the magnitude of the stress interaction of the two tunnels, in order to examine the stability of the ground in the vicinity of the intersection.

Some studies have already been presented on the problem of tunnel intersection. Rigorous research of the available literature has indicated that no suitable complete closed-form solution is available for the stress regime generated by the tunnel intersection. The previous approaches generally fall into two main categories: experimental stress-analysis studies involving 3D photoelasticity and numerical stress analyses employing boundary element methods. Gercek (1986) summarised these early studies and concluded that intersections are generally structurally weaker regions of underground openings and the main reason for the potential instability is the interaction of the 3D stresses induced around the intersection. Therefore, in the design and support of tunnel intersections, the 3D nature of the problem should be given due consideration. The ground behaviour around a tunnel intersection was analysed by Tsuchiyama et al. (1988) using a 3D linear elastic finite element analysis and it was concluded that the ground response in the zone of influence around the main tunnel generated by excavation of an access tunnel extended to approximately one and three tunnel diameters, respectively, on the obtuse and acute angle sides of the access tunnel. Simple 3D finite element calculations of a representative junction in the English Channel Tunnel Project were carried out by Pottler (1992) to determine the necessity of an increased thickness of the permanent lining in the vicinity of the intersection compared to the regular lining cross-section. A 3D finite element model was built by Swoboda et al. (1998) for the intersection between the main tunnel and one of the escape tunnels in the Schönberg Tunnel Project in Austria, in order to study the tunnel stability in jointed rock. The 3D Fast Lagrangian Analysis of Continua (FLAC3D) program was used respectively by Hsiao et al. (2005) and by Sjoberg et al. (2006) to model the behaviours of the tunnel intersection areas in the Hsuehshan Tunnel in Taiwan and the Citybanan Tunnel in Stockholm.

As indicated above, finite difference, boundary element and finite element techniques are the main methods used to investigate ground response in the vicinity of tunnel intersections. However, in many respects finite difference and boundary element techniques are inferior to the finite element method for tunnel modelling (Augarde, 1997). Moreover, most of the 3D finite element analyses mentioned above ignored the tunnelling process and only modelled the stress concentrations around the intersections in an ideal linear elastic medium. Some of these studies dealt with only simple configurations and unlined tunnels. In this paper, the behaviour of the support system for an existing tunnel is examined when a new tunnel is excavated to intersect orthogonally with the existing tunnel. This study has been conducted using 3D finite element analysis incorporating elasto-plastic models to represent the constitutive behaviour of the ground and the existing tunnel support system.

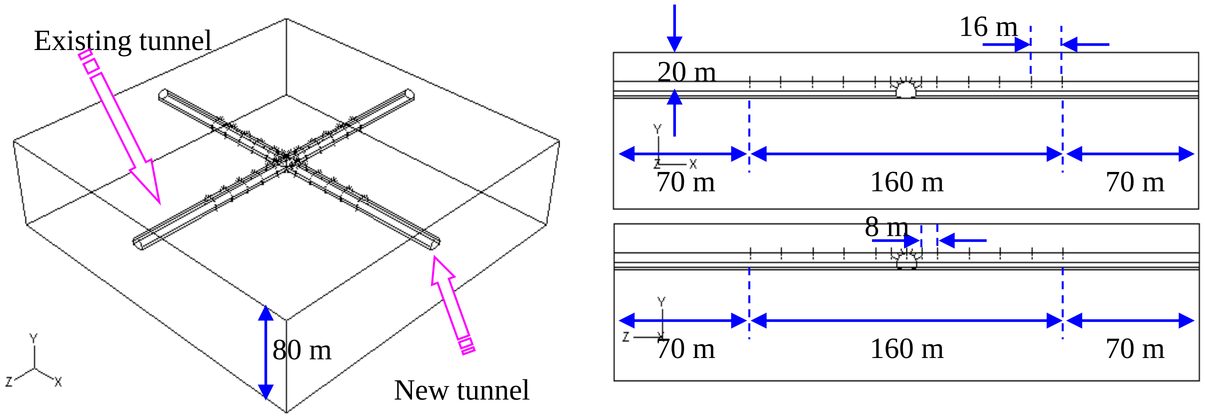
2 3D modelling of the tunnelling process

The geometrical model and the finite element model for the orthogonally intersecting tunnels considered in this study are shown in Figures 1a and 1b, respectively. The modelled region has the dimensions of 30*D* in length (Z-direction), 30*D* in width (X-direction) and 8*D* in height (Y-direction), where $D = 10$ m is the characteristic diameter of the polycentric tunnel depicted in Figure 1 c). It is obvious that the tunnel intersection presents the most issues with respect to mechanical stability and so a higher density mesh is used around the intersection.

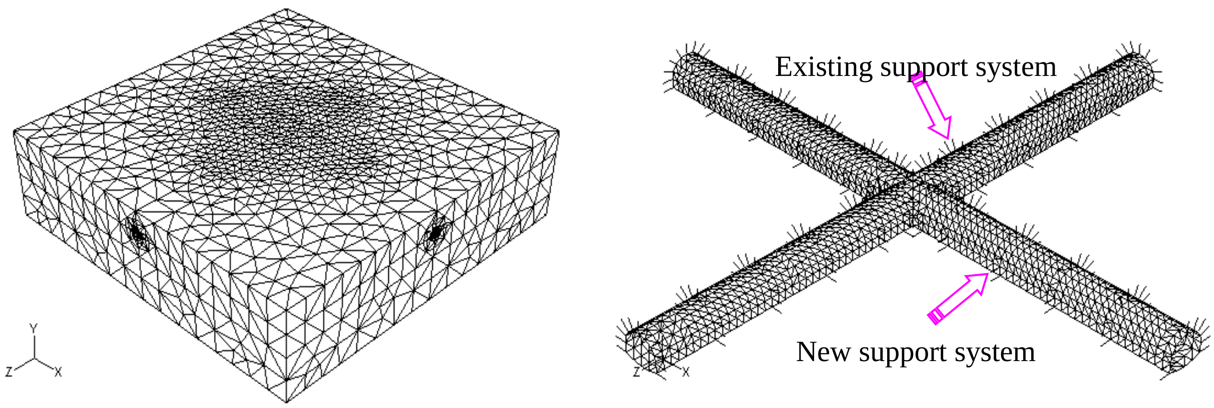
In this study, a tunnel excavated parallel to the Z-axis and supported using a shotcrete lining and rockbolts will be simulated first in a step-by-step procedure. This will then be defined as the existing tunnel. Subsequently, excavation of a tunnel aligned parallel to the X-axis, i.e. intersecting orthogonally with the existing tunnel, will be simulated. Our purpose is to investigate how the driving of the second or new tunnel affects the support system of the existing tunnel using full 3D finite element modelling.

In Figure 1b, the element type chosen to represent the ground is the ten-node tetrahedral element. The most important reason for this choice, rather than the hexahedral element, is that the tetrahedral element can mesh almost any geometry regardless of complexity, which is especially important when tunnels intersect with each other at an oblique angle. Of course, the use of the tetrahedral elements also contributes to reducing the number of elements required in the mesh to achieve refinement and causes less distortion. Moreover, according to Augarde and Burd (2001), the ten-node tetrahedral element has the important advantage for geomechanics modelling of being of sufficiently high order to allow accurate modelling of incompressible material response, as would occur, for example, in a problem involving undrained soil behaviour.

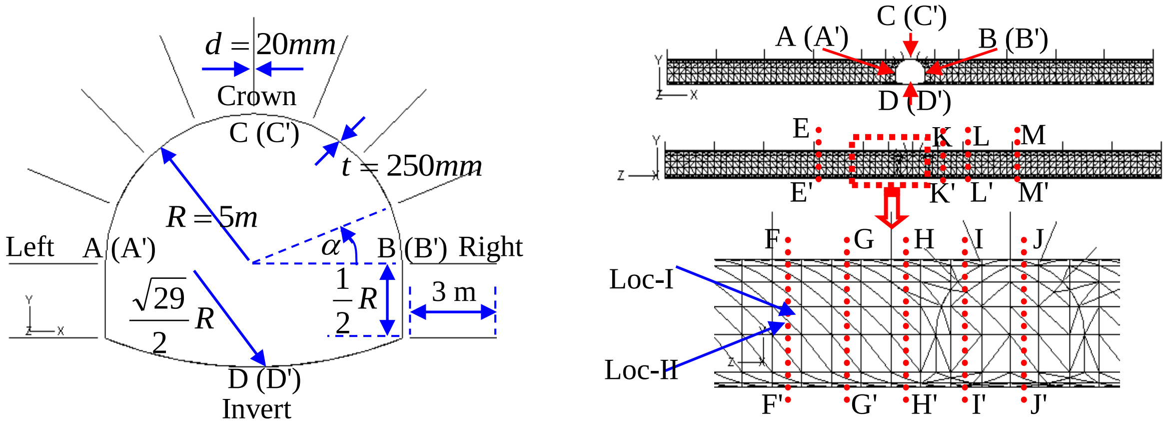
The tunnel lining is modelled using shell elements based on curved thin shell theory. This element is formulated in terms of three displacements and two rotations at each node. Additional fictitious stiffness terms associated with the drilling degree-of-freedom are included to avoid the possibility of a singular stiffness matrix for the case where elements connected to any node lie in the same plane. A reduced integration scheme is used to avoid the possibility of membrane locking. Augarde and Burd (2001) conducted a series of 3D linear elastic finite element analyses to investigate the performance of the shell elements described above, the Phaal and Calladine faceted shell element, and continuum elements. Comparisons between the predictions obtained for these element types and those obtained from analytical solutions indicated that the six-node shell element based on the curved thin shell theory was suitable to model the young shotcrete and the tunnel lining. After excavation, the shell elements were tied to the surface of the tetrahedral elements at the tunnel walls to model interaction between the tunnel lining and surrounding ground. The rockbolts were modelled using three-node beam elements, which were embedded in the ten-node tetrahedral elements to model the interaction between the rockbolts and the surrounding ground.



(a) Geometrical model for intersecting tunnels and their support systems



(b) Finite element model for intersecting tunnels and their support systems



(c) Polycentric tunnel in the XY plane

(d) Support systems in the XY and YZ planes

Figure 1 Three-dimensional (3D) geometrical and finite element models

It is well known that tetrahedral continuum elements and triangular shell elements give very different solution accuracy depending on the mesh patterns used in the model (Benzley et al., 1995; Entrekin, 1999; Lee et al., 2007). Hence, different meshes were used in this study to evaluate the results: a number of different elements were used in these meshes such as four-node linear, ten-node quadratic and ten-node modified (six degrees-of-freedom at each node) tetrahedral elements for modelling the surrounding ground; three-node linear and six-node quadratic triangular elements for modelling the tunnel lining and two-node and three-node beam elements for modelling rockbolts. The influence of these meshes on the results obtained will be addressed in detail in the discussion section. The finite element model presented in Figure 1b includes 841 three-node beam elements, 5068 six-node triangular shell elements and 75,503 ten-node tetrahedral continuum elements for modelling the rockbolts, tunnel lining and surrounding ground, respectively.

According to Pells' (2002) study of the metropolitan area of the Sydney region, most tunnels are constructed in the Hawkesbury sandstone and its inherent weaknesses make the substance properties of the rock mass at the tunnel scale very low. In this study, the behaviour of the rock mass is modelled by an elasto-plastic constitutive relation based on the Mohr–Coulomb criterion with a non-associated flow rule. The behaviour of the shotcrete lining and rockbolts is assumed to be governed by elastic perfectly-plastic relationships using Mises yield surfaces. 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 rockbolts is considered to be negligible and has been ignored. The model parameters selected for this study are summarised in Table 1.

Table 1 Physical-mechanical properties of rock mass, shotcrete lining and rockbolts

Material	E (GPa)	v	c (MPa)	φ (°)	ψ (°)	Model
Rock mass	0.2	0.3	0.5	38	19	Mohr–Coulomb plasticity model
Shotcrete lining	25	0.2	Perfect plasticity model with yield stress = 20 MPa			
Rockbolts	200	0.3	Perfect plasticity model with yield stress = 400 MPa			

According to Pells' (2002) study, the regional stresses in the metropolitan area of the Sydney region can be approximately represented using the following equations:

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

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

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

Where:

σ_x = σ_{NS} is the horizontal regional stress parallel to the axis of the existing tunnel.

σ_y = σ_V is the vertical stress calculated according to the self-weight and the depth H.

σ_z = σ_{WE} is the horizontal regional stress parallel to the axis of the new tunnel.

The external vertical boundaries of the finite element model 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 previous papers (Liu et al., 2008a; 2008b) is used to model the tunnel construction process, where ABAQUS is implemented to solve the non-linear finite element equations and TUNNEL3D, developed using Visual C++, FORTRAN and OpenGL with the ambition of being a virtual reality system in tunnel engineering, is used to modify the input data step by step, control ABAQUS to perform the finite element calculation, analyse the results, as well as retrieve and graphically display the analysis results. TUNNEL3D significantly simplifies the usually complicated series of steps in the full 3D finite element modelling. During simulation of the driving of the new tunnel, the un-installation (removal) of the existing support system is needed around the intersection and TUNNEL3D is capable of dealing with such issues.

Since the numerical procedure was described in detail in previous papers, only a brief description is given here.

The tunnel construction process is modelled using a step-by-step approach. Elements in front of the tunnel face are removed to simulate the tunnel excavation. In this study, the excavation length increment for the existing tunnel is 2 m and that for the new tunnel is 4 m. The reason why different lengths are used is to increase the accuracy of results for the existing support system and to improve solution efficiency. 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 2 m for the existing tunnel and 4 m for the new tunnel. 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' (2002) study, rockbolts are normally used as support in combination with shotcrete lining in underground construction in the metropolitan area of the Sydney region. Thus, the beam elements prestressed with a tensile stress of approximately 100 MPa (≈ 30 kN) are also reactivated behind the tunnel face in every four/eight steps in order to simulate the support provided by the rockbolts, as shown in Figures 1(a) and 1(b). Thus, the entire analysis is divided into 120 analysis steps. Steps 1–82 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 83–120 are implemented to model the driving process of the new tunnel so that the effect of subsequent tunnelling on the existing support system can be quantified.

3 Effects of tunnelling on existing support systems

During modelling several locations, as marked in Figure 1(d), are monitored to quantify the effect of tunnelling on the existing support system. High stresses and stress concentrations are expected for the existing support system around the tunnel intersection during the driving of the new tunnel, which may damage the existing support system and then cause significant instability and subsidence around the tunnel intersection. In this section, the effects of tunnelling on the existing support system is analysed in terms of the induced deformations and stress variations.

3.1 Effects of tunnelling on deformation of existing support systems

The effect of tunnelling on existing support systems is examined here with respect to deformations, especially how the support system of the existing tunnel is influenced by driving the new orthogonally intersecting tunnel. The progression of the working face is one of the important issues whenever ground behaviour is discussed. However, as indicated previously, most existing 3D finite element analyses for the tunnel intersection problem have difficulty in modelling face progression. In this study, full 3D finite element modelling taking into account face progression is performed. The effects of tunnelling on the deformation of the existing support system will be analysed in terms of driving the new tunnel to and from the existing tunnel.

The predicted variations of horizontal and vertical displacements at the left spring line (AA' in Figure 1(d)), the right spring line (BB'), the crown (CC') and the invert (DD') of the existing tunnel during driving of the new tunnel to/from the existing tunnel are shown in Figure 2. It can be seen from Figure 2 that with respect to both horizontal and vertical displacements at the left and right spring lines and at the crown and the invert, the influence of driving of the new orthogonally intersecting tunnel extends about 50 m (5*D*) along the existing tunnel, with the zone of significant influence extending approximately 25 m (2.5*D*) from the existing tunnel intersection, although it seems that the zone of influence with respect to the variation of the vertical displacement is a little smaller than that with respect to the variation of the horizontal displacement.

In the following, the effect of face progression is investigated. During the driving of the new tunnel towards the existing tunnel, the spring lines, the crown and the invert of the existing tunnel move slightly towards the new excavation but are little affected if the driving face is more than 44 m (4.4*D*) away from the intersection. As the working face progresses to be 8 m from the intersection, it is obvious from Figure 2a that the horizontal displacement at the left spring line of the existing tunnel (i.e. the side closest to the new excavation) is significantly influenced by the excavation of the new tunnel (about 30 mm movement towards the new excavation). However, the effect on the horizontal displacement at the right spring line (i.e. the side opposite to the new excavation) is not large except near the point of the intersection where a slight disturbance (about 7 mm of movement) is observed (Figure 2(b)). The magnitudes of the incremental horizontal displacements at the crown and invert are between those predicted at the left and right sides (Figures 2(c) and 2(d)). Moreover, it is observed that the vertical displacements at the spring lines are little affected since the two tunnels intersect with each other at the same level.

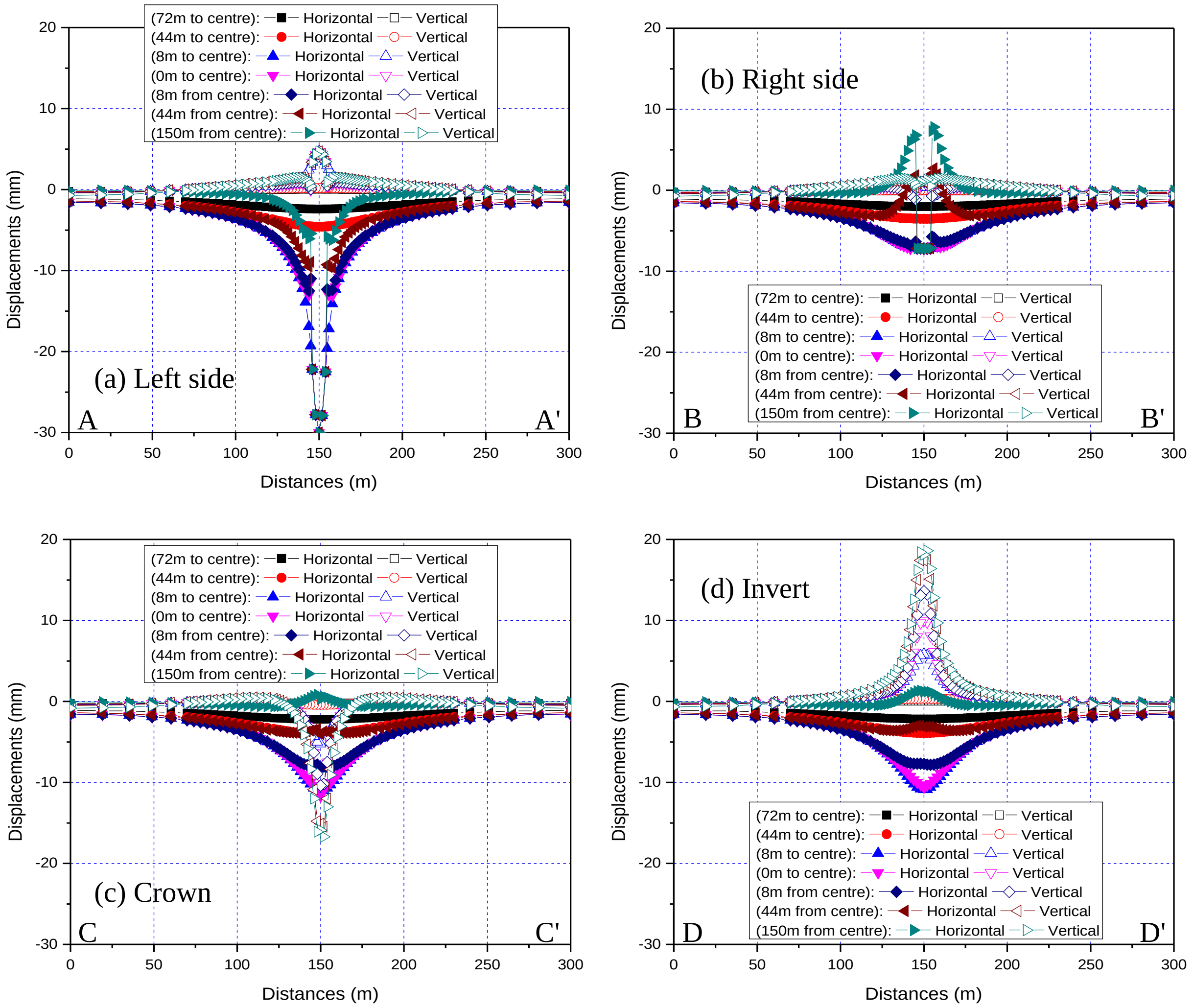


Figure 2 Effects of tunnelling on deformation fields of the existing support system

As the working face approaches the intersection area, the existing shotcrete lining and rockbolts which are located inside the projected new tunnel (refer to Figure 1(d)), are uninstalled (removed). At the same time, four rings of rockbolts (refer to Figure 1) are installed around the intersection to reinforce the existing and new tunnels. The removal of the existing shotcrete lining affects five monitoring locations at each of the left and right spring lines, which makes the displacements in Figure 2(a) and 2(b) at those locations unchanged in the subsequent analysis.

As the tunnel face moves gradually away from the intersection area, the predicted horizontal displacements at the spring lines, the crown and the invert again indicate movement towards the new excavation face, which causes these points to move back towards their initial locations. Large changes are observed at the right spring line (i.e. the side closest to the new excavation), as shown in Figure 2, as the new tunnel face moves from 8–44 m away from the intersection point. The vertical displacements at the spring line are not obvious but the driving of the new tunnel causes the crown to settle continuously and the invert to rise continuously.

The discussion above indicates that in an area of approximately 2.5*D* around the intersection area, the existing support system is significantly affected by the driving of the new tunnel. Due to the high horizontal regional stress in the Sydney region, after the driving of the existing tunnel, the two spring lines of the existing support system move inward towards the tunnel opening and the crown and invert heave upwards. Since only the incremental displacement is of interest, the displacement caused by driving the existing tunnel is reset to zero before the driving of the new tunnel. Driving the new tunnel causes the shotcrete lining at the two spring lines of the existing tunnel to move towards the new excavation, that at the crown to settle down, and that at the invert to upheave. As the tunnel face progresses towards the intersection area, the displacements at the left spring line of the existing tunnel inside the intersection area increase dramatically. After that, the existing support system located inside the intersection area is un-installed. Thus, the deformations at those locations remain unchanged in the subsequent analysis. As the tunnel face moves away from the intersection, the shotcrete lining firstly moves back to its initial location in the horizontal direction and then that in the right spring line moves outwards in the direction of the new tunnel as it moves away.

3.2 Effects of tunnelling on bending moments and axial forces of existing support systems

The effect of tunnelling on the existing support system will now be quantified by observing the variation of the internal forces, i.e. the bending moments and/or axial forces, of the existing tunnel lining and rockbolts. If the bending moment tends to put the side of the tunnel lining facing towards the tunnel opening into tension and the side facing the surrounding ground into compression, it is regarded as positive. Otherwise, it is negative. Positive and negative values of axial force refer to tension and compression respectively.

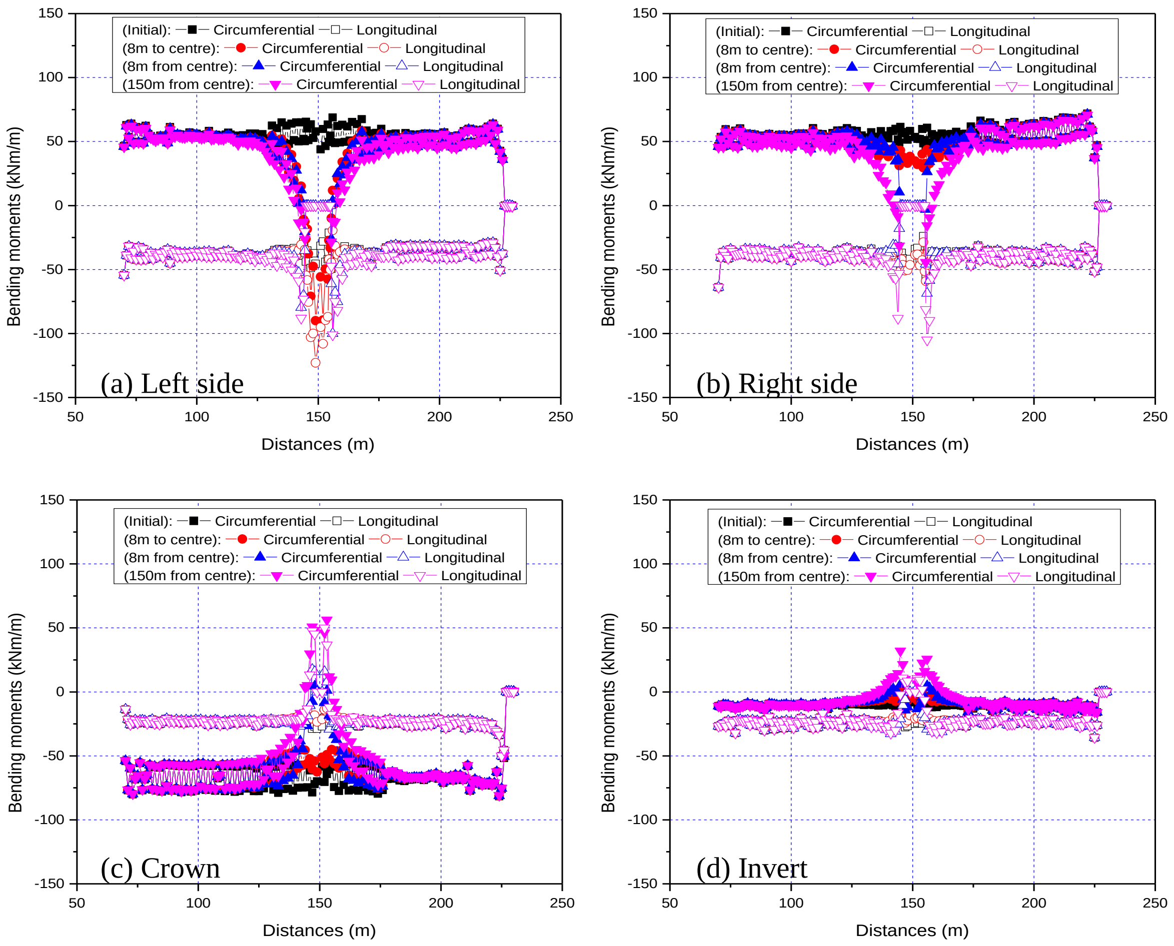


Figure 3 Effects of tunnelling on bending moments of the existing shotcrete lining

Figures 3 and 4 show the variation of the bending moments and axial forces, respectively, monitored at the left spring line (AA' in Figure 1(d)), the right spring line (BB'), the crown (CC') and the invert (DD') of the existing tunnel when the new tunnel is driven to intersect with the existing tunnel. For brevity, only the variations of bending moments and axial forces observed when the new tunnel face is very distant (i.e. initially) and 8 m to, and 8 m and 150 m from the intersection point are shown in Figures 3 and 4. It can be seen that before the driving of the new tunnel, the bending moments and the axial forces at the corresponding locations of the left and right spring lines, the crown and the invert along the axis of the existing tunnel are approximately equal to each other, except those near 230 m. These exceptions are artificial and arise because the driving of the existing tunnel was not modelled step-by-step after the tunnel face progresses to 230 m in order to save calculation time. Moreover, since the model is symmetrical before driving the new tunnel, the internal forces at the left and right spring lines of the existing tunnel are equal to each other, as shown in Figures 3a and 3b for the bending moments, and Figures 4(a) and 4(b) for the axial forces. In the circumferential direction, the positive bending moments at the left and right spring lines mean the tunnel lining there is in tension in the side facing the tunnel opening and the negative bending moments at the crown and invert indicate that the tunnel lining there is in compression in the side facing the tunnel opening. These effects are caused by the high horizontal regional stress in the Sydney region. Comparison of the bending moments indicates that the moments at the crown are bigger than those at the invert, which is reasonable since the ratio between the horizontal regional stress and vertical regional stress decreases as the depth increases.

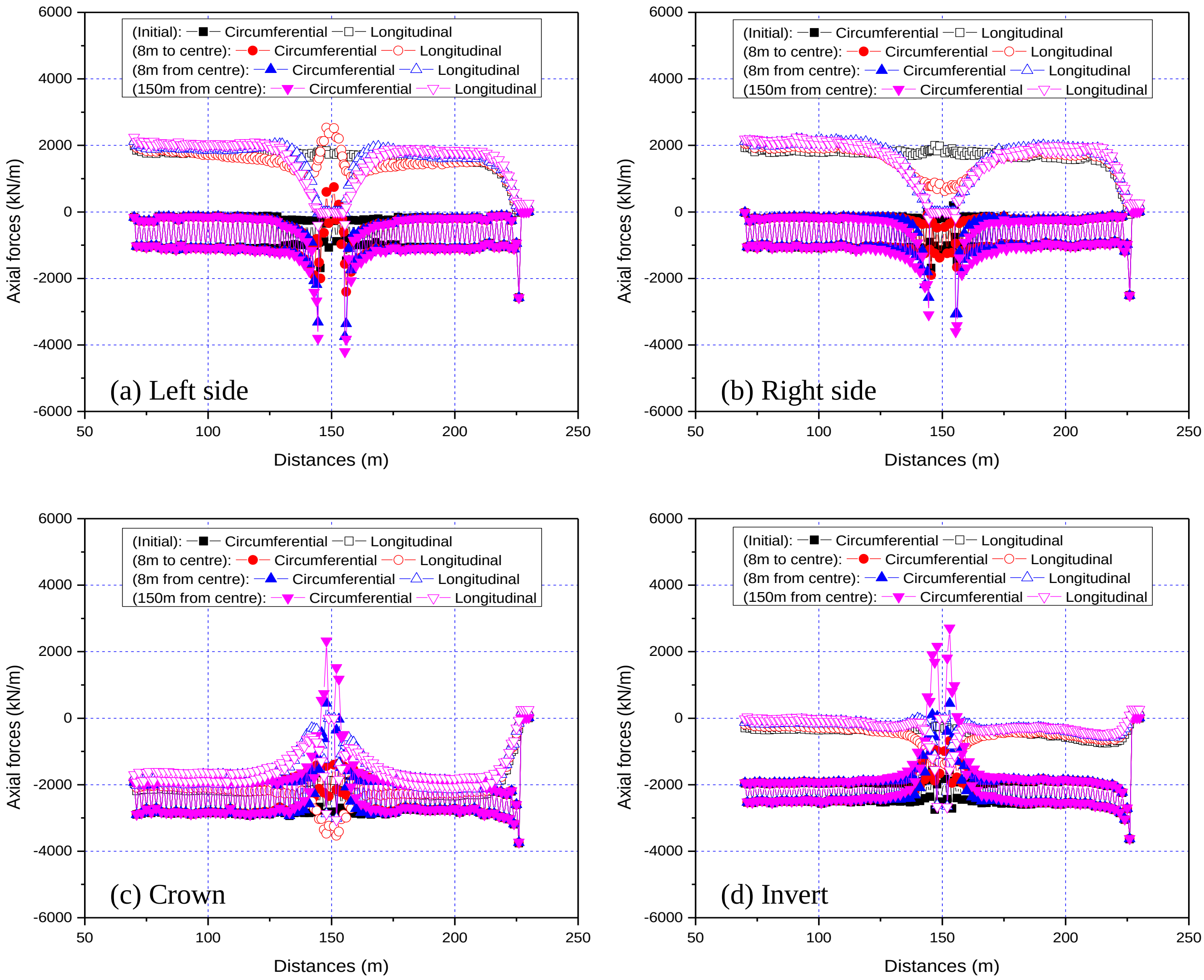


Figure 4 **Effects of tunnelling on axial forces of existing shotcrete lining**

However, it can be seen from Figure 4 that, the axial forces of the existing shotcrete lining, especially in the circumferential direction, show oscillations within a single shotcrete ring of width (2 m) after the step-by-step installation of the existing tunnel. Bonnier et al. (2002) also noted that the bending moments and the normal forces of the tunnel lining have a non-constant, oscillating pattern in their 3D finite element modelling using Plaxis. They argued that the oscillation is caused by the step-by-step installation and is logical since the unsupported tunnel face is arching on the front but not on the back of the tunnel segment. However, it may be, that the oscillation is mainly caused by the combination of tetrahedral and triangular elements used to model the ground and the shotcrete lining, respectively. The oscillations were not observed in previous studies (Liu et al., 2008a; 2008b), where hexahedral elements and quadrilateral elements were used to model the surrounding ground and the shotcrete lining. This issue is discussed in more detail later in this paper.

From Figures 3 and 4, it can be seen that the driving of the new tunnel affects the bending moments and axial forces of the existing support system located in the 25 m zone (2.5*D*) around the tunnel intersection point, which is consistent with the significant influence zone identified for deformations. Beyond this zone, the existing support system remains relatively unchanged in spite of the driving of the new tunnel. Within the zone of influence, the effect is different depending on whether the new tunnel is driven away from or towards the existing tunnel. When the new tunnel is driven towards the existing tunnel, the positive circumferential bending moments of the existing shotcrete lining on the left side of the existing tunnel opening (i.e. the side closest to the new excavation) firstly decrease and then increase to become negative (Figure 3(a)), which indicates that the existing shotcrete lining at this location changes from tension to compression in the side facing the tunnel opening. The longitudinal bending moments on the left side increase significantly when the new tunnel is driven towards the existing tunnel. However, the bending moments at the right spring line (Figure 3(b)), the crown (Figure 3(c)) and the invert (Figure 3(d)) remain almost unchanged (they decrease slightly) until the new tunnel passes through the existing tunnel. The variation of the axial forces in the circumferential and longitudinal directions is similar to that of the bending moments at the corresponding locations (Figure 4). During face advance, the bending moments on the left spring line of the existing shotcrete lining gradually decrease in both circumferential and longitudinal directions. The bending moments on the right spring line, the crown and the invert change significantly during this stage. At the right spring line of the existing shotcrete lining, the bending moment in the circumferential direction decreases and that in the longitudinal direction increases. At the crown and invert, the bending moments in both circumferential and longitudinal directions firstly decrease and then increase, which causes the shotcrete lining to go from compression into tension in the side facing the tunnel opening. The variation of the axial forces in the circumferential and longitudinal directions follows that of bending moments. As the tunnel face moves away from the existing tunnel, the bending moments and the axial forces in the circumferential and longitudinal directions change a little and gradually become stable at those locations.

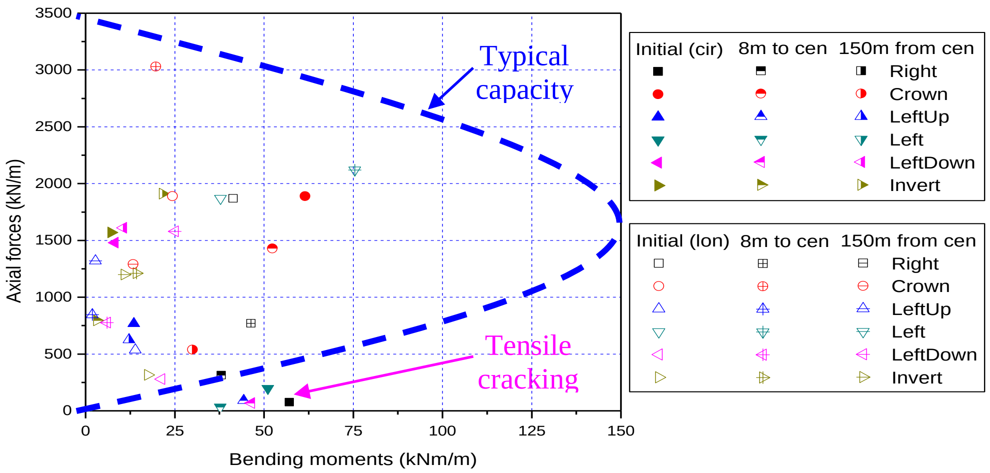


Figure 5 Bending moments and axial forces at II' and comparison with typical lining capacity

It can be seen from Figures 3 and 4 and the above description that the variation of the internal force on the existing support system around the intersection point is very complicated and this kind of variation caused by the driving of the new tunnel may damage the existing support system. In order to further clarify the effect of tunnelling on the existing support system, the variation of the internal force on the existing shotcrete lining in the five full rings (FF' - JJ' in Figure 1(d)) closest to the intersection point are monitored. For convenience, we will focus on the internal forces observed on the front of the tunnel segment (i.e. Loc-I in Figure 1(d)). Comparison of the internal forces at the five rings indicates that driving the new tunnel strongly affects the existing shotcrete lining near the intersection and the effect decreases rapidly as the new face moves away from the intersection. The monitored bending moments and axial forces at the left and right spring lines, the crown, the invert, the left-upper corner and the left-down corner of the five rings at three critical stages are compared with the typical capacity of a concrete lining (Mashimo et al., 2002) to evaluate the significance of the new tunnel, i.e. if the driving of the new tunnel will damage the existing support system.

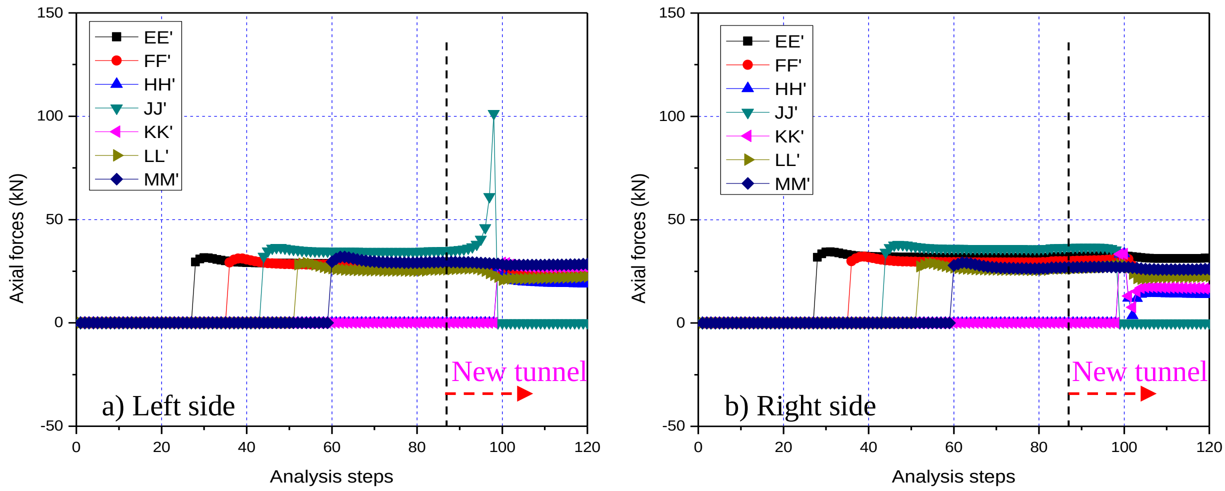


Figure 6 Effects of tunnelling on axial forces of existing rockbolts

For brevity, only the comparisons at the ring II' are depicted in Figure 5. After installation of the existing tunnel, the predicted bending moments and axial forces in the shotcrete lining in the ring are referred to as the initial internal forces. The relationships between the initial bending moments and axial forces in the ring reveal that tensile cracking will occur at the left and right spring lines of a typical shotcrete lining. The reason is the relatively high horizontal regional stress in the Sydney region, which causes the shotcrete lining in the two lateral spring lines to be in tension in the side facing the new tunnel opening. The internal forces at other locations are within the limits of both compressive failure and tensile cracking of a typical concrete lining. Driving the new tunnel towards the existing tunnel causes the circumferential bending moments at the right spring line (the side opposite to the new excavation) of the existing shotcrete lining in the five rings to decrease consistently, although the amount of this reduction decreases with the distance of the ring from the intersection point. At the left spring line (the side close to the new excavation), driving of the new tunnel towards the existing tunnel causes the initially positive circumferential bending moments at the rings II' – JJ' to become negative and those at the rings FF' – HH' to decrease. The changes of the circumferential bending moments at the crown and the invert are not obvious. Driving the new tunnel towards the existing tunnel also causes the bending moments at the upper-left and lower-left of the existing shotcrete lining at the rings II' – JJ' to significantly increase. In the longitudinal direction, the driving of the new tunnel also causes significant increases of the bending moments at the two spring lines, especially the left spring line of the rings II' – JJ', but the effects at other locations are not obvious. The relationship between the bending moment and axial force observed when the new tunnel face is located 8 m from the existing tunnel indicates that the driving of the new tunnel causes the existing shotcrete lining located in the upper-left and lower-left corner of the ring JJ' to start tensile cracking in the circumferential direction, and the crown and the left spring line approach the limit of the compressive failure. At the ring II', as shown in Figure 5, the driving of the new tunnel causes the shotcrete lining located at the upper-left and lower-left corners and the left spring line to undergo tensile cracking. At the rings FF' – HH', driving of the new tunnel causes neither tensile cracking nor compressive failure at this stage. During the tunnel excavation, the existing shotcrete lining located inside the new tunnel is removed and its internal forces are reset to zero. Thus, all of the shotcrete lining at the ring JJ' and that at the left and right spring lines of the ring II' are removed. As the new tunnel is driven away from the existing tunnel, significant increases of bending moments and axial forces are observed at the shotcrete lining located at the ring II’ in the circumferential and longitudinal directions and the increases in the circumferential direction are especially large. At the ring FF' – HH', the variation is not significant. Comparison of the predicted bending moments and axial forces with the typical capacity of concrete lining indicates neither further tensile cracking nor compressive failure is caused during driving of the new tunnel away from the existing tunnel.

The effects of tunnelling on existing rockbolts are depicted in Figure 6 in terms of the variations of axial forces monitored at the rings EE', FF', HH' and JJ' – MM' (Figure 1(d)). It can be seen that before the driving of the new tunnel, the axial forces become stable as the tunnel face advances and those at symmetrical locations have the same values. During driving of the new tunnel, the rockbolts in the rings JJ’ are removed and those in the rings HH' and KK' are installed when the new tunnel face approaches the point of intersection. The removal causes the axial forces of the rockbolts in the rings JJ’ to become zero. The axial forces of the rockbolts at the rings HH' and KK' change as the new tunnel face passes the intersection area and become stable as the tunnel face moves away. During the passing of the new tunnel face through the intersection area, the rockbolts at other locations are little affected since they are relatively far from the intersection area.

4 Discussion

4.1 Zone of influence

The variations of both the predicted deformation and stress fields described in Section 3 indicate that the existing support system is significantly affected in a zone extending a distance of approximately 2.5D from the intersection area when a new tunnel is driven to intersect orthogonally with an existing tunnel in a region with a high horizontal regional stress regime. In this zone, when the new tunnel is excavated to within 4.5D of the intersection, the support system on the approach side of the existing tunnel opening begins being pulled significantly towards the new tunnel excavation. When the new tunnel face is approximately 8 m before the intersection, the existing shotcrete lining within the zone of influence starts tensile cracking on the approach side and compressive failures occur at the crown. Moreover, the existing shotcrete lining closest to the intersection area also undergoes tensile cracking at the lower-left corner, the left spring line and the upper-left corner. However, neither tensile cracking nor compressive failure is observed in the existing shotcrete lining at other locations within the zone of influence. Since the existing support system located inside the intersection area is eventually removed when the new tunnel face passes through the existing tunnel, only temporary support is needed there to be sure that collapse does not occur before the support is removed. However, the existing shotcrete lining immediately adjacent to the intersection area should be thickened or reinforced permanently. At other locations within the zone of influence, no additional support measures are needed. As the face of the new tunnel proceeds through the intersection and reaches a distance of 4.5D from the intersection, the crown of the existing support system settles, the invert heaves and the right spring line is pulled towards the new tunnel. Fortunately, neither further tensile cracking nor further compressive failures are observed during this approach period.

Gercek (1986) reviewed previous studies on tunnel intersections and concluded that for a given state of in situ stress the extent of the zone of influence around an intersection, depends primarily on the intersection configuration. For tee (T) or cross (+) junctions, the influence of the intersection is negligible beyond about 2 diameters from the centre of intersection for openings in an elastic medium. Tsuchiyama et al. (1988) performed a 3D linear elastic finite element analysis with linear tetrahedral elements to study unlined tunnels intersecting at an angle of 45 degrees and concluded that the zone of influence around the existing tunnel generated by the excavation of the new tunnel extended approximately 1 and 3 tunnel diameters on the obtuse and acute angle sides, respectively. They further concluded that the displacement variations became significant during excavation towards the existing tunnel and reduced as excavation proceeded away from the existing tunnel. Variations in displacement effectively ceased when the new tunnel face passed a section located at a distance of 2 to 3 diameters from the tunnel intersection. According to the Japan Nuclear Cycle

Development Institute (JNC, 1999), the influence zone extends approximately 1 diameter on both sides of the tunnel intersection for an intersection angle of 90 degrees, 2 diameters on the acute angle side and 1 diameter on the obtuse angle side for an intersection angle of 60 degrees, and approximately 4 diameters on the acute angle side and 1 diameter on the obtuse angle side for an intersection angle of 30 degrees. It is further concluded that the intersection area in a soft rock system should be reinforced in the area over distances of 4*D* and 1*D* from the intersection on the acute and obtuse angle sides of the intersection, respectively. Pottler (1992) performed a simple 3D finite element calculation to analyse the tee (T) junctions in the English Channel Tunnel Project and found that the T-junction causes compressive stresses and strains in the permanent lining to increase by a factor of approximately 1.5 as compared with those in the regular cross-section, and in some cases the allowable compressive stresses in the permanent lining were exceeded. He concluded that a thickening of the permanent lining in the junction area over a length of approximately 0.5*D* was inevitable but because tension could be sustained by the tunnel lining reinforcement of the permanent lining was not found necessary. As a result of these calculations, appropriate measures were adopted, i.e. local thickening of the lining but no reinforcement were adopted in a number of junctions in the English Channel Tunnel Project.

Due to the relatively high horizontal regional stress field in the Sydney region, the estimated zone of influence of 2.5*D* is larger than the values cited in the literature. As pointed out by Pottler (1992), the size of the influence zone is affected by the physical properties of the ground and the support system, the excavation method, the angle and other aspects of the intersection. Thus, in design and support practice, full 3D numerical analyses should be conducted to take such factors into consideration.

4.2 Influence of finite element mesh

The intersection of tunnels supported with shotcrete lining and rockbolts presents complex structural components, which makes it difficult to mesh the components using quadrilateral/hexahedral elements. In this case, triangular/tetrahedral elements are often used because of their ability to mesh almost any geometry, regardless of the complexity. However, the formulations of the linear triangular/tetrahedral elements make it necessary to use a very large number of elements to accurately model areas around stress concentrations (Entrekin, 1999). In general, the element edge lengths must all be a fraction of the size of the smallest feature in order to get accurate results (Entrekin, 1999). This density of mesh produces models that are often too cumbersome to be analysed. In these situations, second-order triangular/tetrahedral elements are very useful. Since second-order elements are not restricted to straight-line edges, they can model complex solids more accurately with fewer elements.

However, as pointed out in Section 3.2, the bending moments and axial forces in the shotcrete lining, obtained using even second-order triangular/tetrahedral meshes, show oscillations along the tunnel axis. If the tunnel is constructed for sufficient length, the plane strain condition is eventually achieved, which means that the internal forces at any position around the tunnel should become constant along the tunnel. In order to evaluate how the oscillation caused by the tetrahedral/triangular elements will affect the results predicted in this study, five rings (FF' – JJ' in Figure 1(d)) of the tunnel lining around the intersection were monitored, and the internal forces on the front (Loc-I in Figure 1(d)) and the back (Loc-II) of the tunnel segment in those rings are depicted in Figure 7 in terms of the bending moments and the axial forces in the circumferential and longitudinal directions. It can be seen that after installation of the existing tunnel, the bending moments on the front and the back of the tunnel segment in corresponding locations around those rings are approximately equal to each other in both the circumferential and longitudinal directions. Moreover, the bending moments at the left and right spring lines of those rings are equal since the model is symmetrical about a vertical plane through the tunnel axis before the driving of the new tunnel. These numerical results indicate that the density mesh adopted here can reduce the oscillation caused by inaccuracies in the tetrahedral/triangular elements. After installation of the existing tunnel, the axial forces on the front and the back of the tunnel segment at corresponding locations of those rings are also approximately equal to each other in the circumferential direction, but show some variation in the longitudinal direction. Bonnier et al. (2002) argued that this oscillation is caused by the step-by-step installation, since the unsupported tunnel face arches on the front but not on the back of the tunnel. However, because these oscillations were not observed in previous studies conducted by the authors (Liu et al., 2008a; 2008b) where hexahedral/quadrilateral elements were used, it is thought that this oscillation is mainly caused by the inaccurate tetrahedral/triangular elements, which can not completely redistribute the stress caused by the arching as the tunnel face advances. Although there are variations between the longitudinal axial forces in the front and the back of the tunnel segment, the longitudinal axial forces in the front (and back) of the tunnel segment at corresponding locations of those rings are approximately equal to each other, as shown in Figure 7. The oscillation has been greatly reduced in this study by using the second-order tetrahedral/triangular elements, density meshes and small excavation steps, selected after trial modelling using various finite element meshes with different densities and elements.

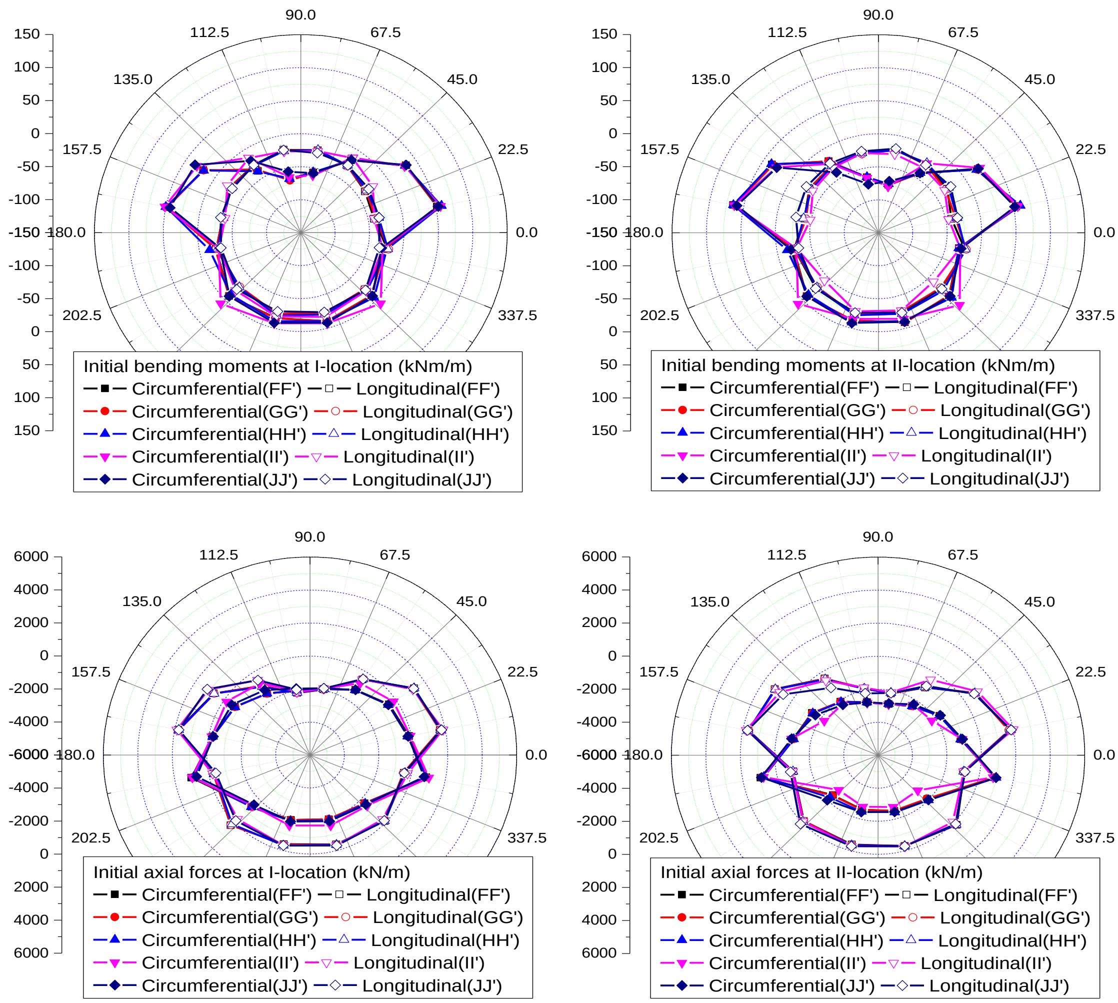


Figure 7 Bending moments and axial forces in the lining after driving the first tunnel

5 Conclusions

The effects on existing support systems, i.e. shotcrete lining and rockbolts, caused by excavation of intersecting tunnels in the Sydney region with a high horizontal regional stress regime have been investigated using full 3D finite element analyses coupled with elasto-plastic material models. These analyses indicate that the stress and displacement fields of the existing support system are significantly affected in a zone extending approximately 2.5*D* from the intersection point. The support systems located beyond this zone appear to be almost unaffected when the new tunnel is driven orthogonally to the existing tunnel. Within the zone of influence, variations of stress and displacements begin to appear in the existing support system as the new tunnel approaches and they become negligible when the new tunnel face passes the section located approximately 4.5*D* away from the tunnel intersection. Specifically, in a region such as Sydney with a high horizontal regional stress regime, the existing shotcrete lining at the tunnel sides is placed in tension in the side facing towards the tunnel opening and in compression at the crown and invert. The pre-stressed existing rockbolts are usually tensioned more in the zone closest to the tunnel opening. Comparisons of the predicted bending moments and axial forces with the typical capacity of a shotcrete lining indicate that tensile cracking may occur at the sides of the existing tunnel. During the approach of the new tunnel face, tensile cracking is induced in the existing shotcrete lining inside the zone of influence on the approach side. It is suggested that temporary reinforcement should be installed inside this zone and local thickening of the shotcrete lining may be necessary in locations immediately adjacent to the intersection. However, neither further tensile cracking nor further compressive failure occurs in the existing shotcrete lining as the new tunnel face progresses away from the intersection. Finally, various aspects were seen to influence the results of this analysis such as the initial stress regime, the finite element mesh, and the length of tunnel excavation.

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References

Augarde, C.E. (1997) Numerical modelling of tunnelling processes for assessment of damage to buildings. Department of Engineering Science, University of Oxford, pp. 39–40.

Augarde, C.E. and Burd, H.J. (2001) Three-dimensional finite element analysis of lined tunnels. International Journal for Numerical and Analytical Methods in Geomechanics 25, pp. 243–262.

Benzley, S.E., Perry, E., Merkley, K., Clark, B. and Sjaardama, G. (1995) A comparison of all hexagonal and all tetrahedral finite element meshes for elastic and elasto-plastic analysis. Proceedings 4th International Meshing Roundtable Albuquerque, New Mexico, Sandia National Laboratories, pp. 179–191.

Bonnier, P.G., Moller, S.C. and Vermeer, P.A. (2002) Bending moments and normal forces in tunnel linings. 5th European Conference Numerical Methods in Geotechnical Engineering (NUMGE 2002), Paris: Presses de l'ENPC/LCPC, pp. 515–522.

Entrekin, A. (1999) Accuracy of MSC/NASTRAN first- and second-order tetrahedral elements in solid modelling for stress analysis. MSC Aerospace Users' Conference, USA, pp. 1–8.

Gercek, H. (1986) Stability considerations for underground excavation intersections. Mining Science and Technology 4, pp. 49–57.

Hsiao, F.Y., Yu, C.W. and Chen, J.C. (2005) Modelling the behaviours of the tunnel intersection areas adjacent to the ventilation shafts in the Hsuehshan tunnel. World Long Tunnel 2005, Taiwan, China, pp. 81–90.

Japan Nuclear Cycle Development Institute (JNC) (1999) Supporting report 2: Repository design and engineering technology. H12: Project to establish the scientific and technical basis for HLW disposal in Japan, Japan.

Lee, P.S., Noh, H.C. and Bathe, K.J. (2007) Insight into 3-node triangular shell finite elements: the effects of element isotropy and mesh patterns. Computers and Structures 85, pp. 404–418.

Liu, H.Y., Small, J.C. and Carter, J.P. (2008a) Full 3D modelling for effects of tunnelling on existing support systems in the Sydney region. Tunnelling and Underground Space Technology 23, pp. 399–420.

Liu, H.Y., Small, J.C. and Carter, J.P. (2008b) Effects of tunnelling on existing support systems of perpendicularly crossing tunnels. Computers and Geotechnics (c0urrently in revision).

Mashimo, H., Isago, N., Yoshinaga, S., Shiroma, H. and Baba, K. (2002) Experimental investigation on load-carrying capacity of concrete tunnel lining. 28th ITA General Assembly and World Tunnel Congress (AITES-ITA DOWNUNDER 2002), pp. 1–10.

Pells, P.J.N. (2002) Developments in the design of tunnels and caverns in the Triassic rocks of the Sydney region. International Journal of Rock Mechanics and Mining Sciences 39(5), pp. 569–587.

Pottler, R. (1992) Three-dimensional modelling of junctions at the channel tunnel project. International Journal for Numerical and Analytical Methods in Geomechanics 16, pp. 683–695.

Sjoberg, J., Perman, F., Leander, M. and Saiang, D. (2006) Three-dimensional analysis of tunnel intersections for a train tunnel under Stockholm. Proceeding 2006 North American Tunnelling Conference, Ozdemir L. (editor), Taylor & Francis Group, London, pp. 39–48.

Swoboda, G., Shen, X.P. and Rosas, L. (1998) Damage model for jointed rock mass and its application to tunnelling. Computers and Geotechnics 22(3/4), pp. 189–203.

Tsuchiyama, S., Hayakawa, M., Shinokawa, T. and Konno, H. (1988) Deformation behaviour of the tunnel under the excavation of crossing tunnel. Proceedings 6th International Conference on Numerical Methods in Geomechanics. Innsbruck, Balkema, Rotterdam, pp. 1591–1596.

WorkCover (2005) Lane Cove Tunnel collapse and subsidence. WorkCover Authority of New South Wales, Australia, November, 2005.