Effects of twin tunnel excavation on an existing horseshoe-shaped tunnel with considering settlement joint

<table>
<thead>
<tr>
<th>Journal:</th>
<th>Canadian Geotechnical Journal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manuscript ID</td>
<td>cgj-2015-0389.R3</td>
</tr>
<tr>
<td>Manuscript Type:</td>
<td>Note</td>
</tr>
<tr>
<td>Date Submitted by the Author:</td>
<td>18-Nov-2016</td>
</tr>
<tr>
<td>Complete List of Authors:</td>
<td>Jiang, Biao; Hunan University, School of Civil Engineering; Changsha Metro Group Co., Ltd Chen, Li’ang; Hong Kong University of Science and Technology, CIVIL Yang, Junsheng; School of Civil Engineering, Central South University Wang, Shuying; School of Civil Engineering, Central South University Ng, C.W.W.; Hong Kong University of Science and Technology</td>
</tr>
<tr>
<td>Keyword:</td>
<td>twin tunnel excavation, settlement joint, three-dimensional centrifuge modelling, three-dimensional numerical modelling, horseshoe</td>
</tr>
</tbody>
</table>

https://mc06.manuscriptcentral.com/cgj-pubs
Effects of twin tunnel excavation on an existing horseshoe-shaped tunnel with considering settlement joint

Biao Jiang\textsuperscript{1,4}, Li’ang Chen\textsuperscript{2}, J.S.Yang\textsuperscript{3}, Shuying Wang\textsuperscript{3} and C.W.W.Ng\textsuperscript{2}

\textsuperscript{1}School of Civil Engineering, Hunan University, Changsha, Hunan, P. R. China;

\textsuperscript{2}Department of Civil and Environmental Engineering, The Hong Kong University of Science and Technology, Clear Water Bay, Kowloon, HKSAR

\textsuperscript{3}School of Civil Engineering, Central South University, Changsha, Hunan, P. R. China

\textsuperscript{4}Changsha Metro Group Co., Ltd. Changsha, Hunan, P. R. China

Corresponding author: Li’ang Chen (e-mail: keithclawolf@163.com)
Abstract: In urban cities, the construction of a new tunnel would cause ground disturbance and affect any adjacent existing tunnel. Tunnel-tunnel interaction problems have not yet been comprehensively studied. In particular, the response of an existing horseshoe-shaped tunnel to the construction of two perpendicularly undercrossing tunnels remains unclear. In this study, three-dimensional centrifuge tests and three-dimensional numerical analysis were designed and conducted in dry sand to investigate the effects of twin-tunnel undercrossing on an existing horseshoe-shaped tunnel considering the influence of settlement joint. It is found that the adverse effects caused by later excavated tunnel were smaller than those by former one. For the existing tunnel without a settlement joint, the settlement at the invert and crown along the longitudinal direction showed sagging and hogging curves, respectively, owing to significant vertical elongation of the middle section. The presence of a settlement joint resulted in an increase in the settlement of the ground surface and the existing tunnel by over 100%. In both cases, larger bending strains were induced in the transverse direction than in the longitudinal direction, particularly around the corners.

Key words: Twin-tunnel excavation, horseshoe, settlement joint, three-dimensional centrifuge modelling, three-dimensional numerical modelling
Introduction

In recent years, it has become increasingly common for one subway line to advance across another because of congested underground space in cosmopolitan cities. Tunnel excavation leads to soil movement and stress changes in the ground, which can cause existing tunnels to sink and crack (Cooper et al. 2002; Mohamad et al. 2010). It is thus essential for designers and engineers to fully understand the tunnel-soil-tunnel interaction mechanism and estimate any potential damage to existing tunnels before proceeding to build new ones close by.

Kim et al. (1998) tested reduced-scale models under the 1g condition to investigate the effect of constructing a new circular tunnel on existing circular tunnels in clay. They found that the new tunnel construction redistributed the stress acting on the lining of the existing tunnel. These stress changes further induced significant lining deformation and internal force. Ng et al. (2013a, 2015) and Boonyarak and Ng (2015) conducted systematic studies to investigate interactions between perpendicularly crossing tunnels. They carried out a series of centrifuge tests to determine the effects of volume loss, weight loss, pillar depth, tunnel shielding, cover depth (distance from tunnel crown to ground surface) and construction sequence on tunnel-tunnel interactions. Zhang and Huang (2014) conducted three-dimensional finite element numerical simulations to investigate soil disturbance and the deformation of existing parallel twin circular tunnels induced by above-crossing and down-crossing circular tunnels. Deformation of the existing tunnels was mostly caused by the down-crossing tunnel. In all of these studies, the existing tunnels and new tunnels shared similar geometries (i.e., circular in shape and ordinary in size). However, it is not unusual to come across horseshoe-shaped tunnels (Kovári 2003; Shin et al. 2005; Liu et al. 2009). In fact, stress redistribution and deformation may be more severe in horseshoe-shaped tunnels than in circular tunnels (González-Nicieza et al. 2008).
Chen et al. (2011) and Li and Yuan (2012) both carried out field studies to investigate twin-tunnel excavation effects. Their results on the effects of twin-tunnel excavation are inconsistent, however, perhaps due to the lack of complete and reliable information.

Recently, the longitudinal and circumferential joints of tunnels have attracted much attention (Huang et al. 2012; Li et al. 2014; Ye et al. 2014; Shi et al. 2015). But few studies have focused on settlement joints (i.e., a completely disconnected gap filled with flexible adhesive material), even though such joints can significantly reduce the longitudinal flexural rigidity of tunnels. The interaction between circular and non-circular jointed tunnels has rarely been reported in the literature and the influence of joints on the interaction is not well understood.

In this paper, three-dimensional centrifuge model tests were conducted to investigate the response of an existing horseshoe-shaped tunnel to the excavation of perpendicularly undercrossing twin tunnels in dry sand. Three-dimensional numerical back-analyses were carried out to verify and further interpret the centrifuge results. Special attention was paid to the tunnel response induced by each new tunnel. The influence of a settlement joint was also investigated. To design the tunnel shapes and sizes for centrifuge model tests, the actual geometry of the existing Liuyang River tunnel and the newly proposed Changsha Metro line 3 twin circular tunnels are chosen and equivalized. As shown in Figure 1, the new twin tunnels of Metro line 3 are designed to be built close to the bottom of the existing Liuyang River tunnel with a cross-angle of 90°.

**Three-dimensional centrifuge modelling**

**Test program**

Two three-dimensional centrifuge tests were carried out at the Geotechnical Centrifuge Facility of the Hong Kong University of Science and Technology. The tests were performed
at a centripetal acceleration of 60g. The dimensions of the model box used in this test were 1250 mm (length) × 930 mm (width) × 850 mm (depth). The designed soil depth was 800 mm. This soil model is equivalent to a soil sample in prototype measuring 75 m long, 55.8 m wide and 48 m deep.

Test C2 was the reference test and adopted real field conditions. Tests C2 and J2 shared identical geometries (e.g., positions and dimensions of tunnels). The only difference was that the existing tunnel was continuous in Test C2 (hence the prefix ‘C’) and jointed in Test J2 (hence the prefix ‘J’). The numeral ‘2’ in C2 and J2 denotes the ratio of pillar depth (clear vertical distance between the existing tunnel and new tunnels) to the diameter of the new tunnels.

**Centrifuge model package**

As mentioned earlier, Test J2 was configured in the same way as Test C2 except for how the settlement joint was simulated. Thus for simplicity, this section will only describe the model package of Test C2. The modelling of the settlement joint will be described later. Figure 2a shows the centrifuge model of Test C2 in plan view. The two new tunnels were symmetrically placed on either side of the ‘Y’ axis. They were excavated individually in six steps along the ‘Y’ direction, following the sequence of L1, L2…L6 and R1, R2…R6. The length of each advancing section was 60 mm in model scale, corresponding to 3.6 m in prototype scale.

Figure 2b shows the elevation view of the centrifuge model. The cover depths of the existing tunnel and new tunnels were 73 and 470 mm (4.25 and 28.2 m in prototype), respectively.

The models of the existing tunnel and new tunnels were made of aluminum alloy. The dimensions and shapes of the existing tunnel and new tunnels are shown in Figure 2c.
Soil property and model preparation

Toyoura sand was used in this study for ease of model preparation and data interpretation. The soil model was prepared using the pluvial deposition method. With a constant rate of pluviation, the average density of the sand sample in the tests was approximately 1521 kg/m$^3$, corresponding to an average relative density ($D_r$) of 62%.

Instrumentation

Eleven sets of LVDTs were used to measure crown settlement of the existing tunnel at every 100 mm. In addition, 10 sets of LVDTs were used to measure the ground surface settlement. The locations of all LVDTs are illustrated in Figure 2b. To measure the bending strain of the existing tunnel along the longitudinal direction, 15 sets of strain gauges were mounted on the external surface of the tunnel at a spacing of 50-100 mm.

The middle section of the existing tunnel, where X=0, was chosen as the monitoring section. Eight sets of strain gauges were used to measure bending strain of the tunnel lining along the transverse direction. Moreover, four potentiometers were installed to measure the radial deformation of the existing tunnel.

Test procedure

The centrifuge test procedure adopted in this study is similar to those reported by Ng et al. (2013a, 2015) and Boonyarak and Ng (2014). Tunnel advancement was modelled using the so-called “donut” method (Ng et al. 2013a), which could simultaneously simulate volume loss and weight loss during tunnel excavation. A volume loss of 1.5% was assumed in the test based on Mair and Taylor’s (1997) report that EPB shield tunnel excavations typically induce volume losses of 1 to 2%.
Three-dimensional numerical back-analysis

Finite element mesh and boundary conditions

The finite element program, Plaxis 3D 2013 (Brinkgreve et al. 2013), was adopted in this study for numerical back-analysis. The model geometries and tunnel positions in the finite element mesh were identical to those in the centrifuge test. A 10-node tetrahedral was adopted to model the soil. Roller support and pin support were applied to the vertical and horizontal boundaries of the mesh, respectively. The tunnel lining was modelled using 6-node elastic plate elements. Tie constraint was applied between the tunnel lining and surrounding soil.

Constitutive model and model parameters

In order to capture the stress dependency and history dependency of Toyoura sand, an advanced constitutive model called the ‘hypoplastic’ model was adopted in this study (Kolymbas 1991; von Wolffersdorff 1996; Gudehus and Mašín 2009). Since the hypoplastic constitutive model has performed well in dealing with tunnel-tunnel interaction problems in previous studies (Boonyarak and Ng 2014; Ng et al. 2015), the same model parameters were used in this study. The values of model parameters in this study are summarized in Table 1. The coefficient of at-rest earth pressure ($K_0$) was assumed to be 0.5.

The model tunnels were modelled as a linear elastic material with Young’s modulus of 69 GPa and Poisson's ratio of 0.33.

Numerical modelling procedures

The adopted numerical modelling procedure basically followed the centrifuge test procedures. First, the deformation of soil and tunnel was initialized at 1g. Then, the spin-up
process was achieved by increasing the unit weights of the soil and tunnel by 60 times. The weight loss and volume loss were simulated by deactivating the soil mass inside the tunnels and applying surface contraction to the tunnel lining, respectively.

**Modelling of the settlement joint**

The function of a settlement joint is to prevent the tunnel lining from cracking from excessive differential settlement. In this test, the stiffness of the settlement joint was zero, which means almost no connection exists between the two sides of the settlement joint. To achieve this, the existing model tunnel was cut into two halves. The settlement joint was designed at the middle of the existing tunnel (right above the center of the twin-tunnel) in order to study the most critical condition.

Figure 3a shows the modelling of the settlement joint in the centrifuge test. For ease of installation, the two halves of the existing tunnel were temporarily connected together by thin plates and screws. The screws were completely loosened after the existing tunnel was installed in the model box. The two halves were spaced 5 mm apart (i.e., the length of the settlement joint is corresponding to 300 mm in prototype) to prevent contact. Tape was used to cover the gap to prevent sand intrusion.

Figure 3b illustrates the numerical modelling of the existing tunnel in Test J2. The dark-colored object is the existing tunnel, whose material type is identical to that in the numerical model of Test C2. The light-colored band at the middle of existing tunnel simulates the tape used to cover the gap between the two halves of the existing tunnel.

**Interpretation of results**

All results presented in this study are in prototype scale unless stated otherwise. Only the results related directly to new tunnel excavation are presented.
Attention will initially be paid to Test C2 only to investigate the effect of twin-tunnel excavation on an existing horseshoe-shaped tunnel. The results labeled with “1st only” in the figures and tables represent the effects due to the first tunnel excavation only, and they are obtained by comparing responses (e.g., displacement, strain, etc.) before and after the first tunnel excavation. Similarly, the results labeled with “2nd only” represent the responses due to the second tunnel excavation only. The results labeled with “total” are the cumulative responses after the twin-tunnel excavation. Results of Test C2 and Test J2 will be compared later to study the influence of a settlement joint on multi-tunnel interaction.

**Settlement of the existing tunnel invert induced by the first and second tunnel excavations (Test C2)**

Figure 4 compares the tunnel invert settlements induced by the first new tunnel (on the left) and the second new tunnel (on the right). Instead of the crown settlement, the invert settlement of the existing tunnel was examined because that was where the maximum settlement occurred due to vertical elongation. The distance from the midline of the existing tunnel to the monitored location was normalized by the depth of the new tunnel’s centroid (Z, Z=520mm). The discrepancy between computed and measured invert settlement was small (i.e., less than 5%). For the first tunnel, the computed maximum invert settlement was about 13.6 mm and occurred directly above the axis. The maximum incremental invert settlement (12.9 mm) induced by the second tunnel was 5% less than that induced by the first tunnel. Moreover, the location where the maximum settlement resulting from the second tunnel excavation was shifted slightly away from the axis of the second tunnel to that of the first tunnel, which is in agreement with the findings of Addenbrooke and Potts (2001). As a result, the maximum invert settlement after the twin-tunnel excavation was located slightly to the left of the midline of the existing tunnel.
For comparison purpose, the greenfield condition (i.e., in the absence of an existing tunnel) was also computed. The computed subsurface settlement at the invert level of the existing tunnel is shown in Figure 4. As expected, the greenfield subsurface settlement shows a similar trend to the invert settlement of the existing tunnel. The computed invert settlement of the existing tunnel was approximately 20% smaller than the greenfield subsurface settlement. This is because the existing tunnel stiffened the ground and hence led to smaller settlements. The maximum greenfield subsurface settlements induced by the first new tunnel and the second new tunnel were 17.3 and 16.5 mm, respectively.

Longitudinal bending strain of the existing tunnel induced by the first and second tunnels (Test C2)

Figure 5 illustrates the bending strains at the invert along the longitudinal direction induced by the first and second tunnel excavations. The positive and negative signs denote tensile and compressive strains at the invert, respectively, corresponding to sagging and hogging bending moments along the longitudinal direction. The computed and measured results were slightly different, due possibly to the fact that measured longitudinal strain was deduced by assuming that the tunnel was a beam and using beam theory for calculation, which may not be completely accurate. However, the general trend of computed longitudinal strain and the maximum value fit the measured results very well.

Sagging moment was induced near the midline of the existing tunnel while hogging moment was induced at the two ends, consistent with the invert settlement curve. The total measured maximum longitudinal strain after twin-tunnel excavation was 37.6 με at the midline of the existing tunnel and reduced to -9 and -5 με at the left and right ends of the existing tunnel, respectively (see Fig. 5). Note that the ultimate tensile strain (cracking limit) of unreinforced concrete is 150 με (ACI 2001). The induced longitudinal bending strains in
this study were small compared with the cracking limit, since the existing tunnel was very stiff (governed by tunnel size and thickness) compared with a normal-sized tunnel (e.g. the new tunnels in this test).

The maximum longitudinal strain (measured) induced by the first tunnel excavation was 25.2 $\mu$e at the axis of the first tunnel, and gradually reduced to -8 $\mu$e at the two ends of the existing tunnel. The maximum longitudinal strain (measured) induced by the second tunnel was 11% less than that due to the first tunnel. The results of both invert settlements and longitudinal strains imply that the second excavated tunnel had a (5-11%) smaller influence on the existing tunnel. The reason for this phenomenon is that the excavation of the first new tunnel had redistributed the soil stress field around existing tunnel. The first tunnel excavation caused the soil stiffness to decrease within a small area close to the axis of the excavated tunnel, due to the decrease in confining pressure. However, the stiffness of adjacent soil (e.g., the area between the second new tunnel and the existing tunnel) increased because of the increase in confining pressure, which contributed to reducing the response induced by the second tunnel. In the interest of space, a detailed discussion of the results of stress redistribution will not be made here but can be found in Chen (2016).

**Tunnel settlement due to twin-tunnel excavation (Test C2)**

Figure 6 shows the settlement at the crown and invert of the existing tunnel after both new tunnels were excavated. The computed results are consistent with the measured ones. The settlement curves for the tunnel crown displayed an inverted U shape. The induced minimum settlement of the tunnel crown was 7.8 mm near the midline of the existing tunnel and increased to 12.4 mm at the two ends of the existing tunnel. According to previous studies, however, the curves of ground and existing tunnel settlement induced by excavation underneath should display a U shape (Ng et al. 2013a; Li and Yuan 2012). The unusual
inverted U-shaped curves observed in this study will be explained later.

Unlike the crown settlement, the invert settlement did show a U-shaped curve. The maximum computed invert settlement was 22.7 mm at the midline, which was 2.7 times higher than crown settlement at the same location. The maximum settlement at the invert exceeded the allowable settlement limit of both LTA (2000) and BD (2009), indicating that special attention should be paid to controlling the settlement of the existing tunnel. The vertical deformation of the tunnel in the longitudinal direction can be deduced by subtracting crown settlement from invert settlement. As shown in Figure 6, significant positive vertical deformation (elongation) of 14.3 mm was induced at the midline. With increasing distance from the midline, tunnel vertical deformation dropped to -5 mm (compression) at the two ends of the existing tunnel. This suggests that the middle section (i.e. X/Z=0) might be the most vulnerable to deformation. The slope of the vertical deformation curve was steeper than that of the invert settlement curve, which explains why the slopes of the crown settlement curve and the invert settlement curve had opposite signs. Possible reasons for the large vertical elongation might be the larger than normal size and shallow cover depth of the existing tunnel. In order to verify this hypothesis, the effects of cover depth and existing tunnel size were investigated.

Effects of cover depth and size on the settlement of the existing tunnel

Figure 7 shows the computed crown and invert settlements of the existing tunnel induced by twin-tunnel excavation for three different cases. The legend with “original C/D=0.4” represents the original case, which is test C2. The legend with “C/D=1.9” means that the cover depth ratio of existing tunnel was increased to 1.9. The vertical distance between the existing tunnel and new tunnels was kept to the same value as in test C2. For the third case, the legend with “50% size” means that the size (i.e., width and height) of the
existing tunnel was reduced by 50%. In this “50% size” case, the centroids of the existing tunnel and new tunnels were kept to the same values as in test C2.

Hogging settlement of the crown occurred only in the original case. The crown settlements at the middle of the existing tunnel were 8.2, 11.6 and 17.8 mm for original, “C/D =1.9” and “50% size” cases, respectively. The invert settlements at the middle of the existing tunnel were 22.8, 20.8 and 23.1 mm for the three cases. It is clear that the invert settlement of the existing tunnel is insensitive to changes in the tunnel size and cover depth of the existing tunnel. Therefore, the significant difference in crown settlement of the existing tunnel between the original case and the other two cases is mainly due to the difference in vertical elongation of the existing tunnel. The middle section of the existing tunnel was vertically elongated by 9.2 and 5.3 mm (invert settlement minus crown settlement) in the “C/D =1.9” and “50% size” cases, respectively, which were 37 and 62% less than that in the original case (14.6 mm). This is because the radial deformation of the existing tunnel should be proportional to its diameter. Moreover, a higher confining pressure would help restrict the radial deformation of the existing tunnel.

**Effect of settlement joint on tunnel response in the longitudinal direction (Tests C2 and J2)**

Figure 8a compares the settlement curves of the ground surface and tunnel crown in Tests J2 and C2 (after twin-tunnel excavation). The computed results match the measured results well, which reflects the high quality of the centrifuge tests. In Test C2, the settlement curve of the ground surface had the same inverted U shape as that of the tunnel crown. This is because with the shallow cover depth of the existing tunnel, the soil above the crown tended to deform in a similar way to the tunnel crown. The maximum ground settlement was 12.4 mm at X/Z= 0.9, and gradually decreased to 9.8 mm at X/Z= -0.1. The ground surface settled
more than the tunnel crown, which means that the soil above the crown of the existing tunnel was compressed due to the new tunnel excavation.

The settlement curves of the ground surface in Test J2 exhibited opposite trends to those in Test C2. The maximum and minimum ground settlements were 19.3 and 6mm, respectively, and occurred near the midline and at the end of the existing tunnel. The opposing trends of ground surface settlement in Test C2 and Test J2 implies that the shielding effect of the existing tunnel was eliminated by the settlement joint. Similar to the ground surface, the settlement curve of the tunnel crown also showed a U shape in Test J2. It also displayed a jump at the middle of the existing tunnel, owing to the presence of the settlement joint. The maximum crown settlement for the left half of the existing tunnel was 19.5 mm at X/Z= -0.005, which was 1.1 mm larger than the maximum settlement for the right half of the existing tunnel. This is as expected because the left new tunnel would have induced a larger impact on the existing tunnel than the right new tunnel as discussed in the previous section, resulting in the left half of the existing tunnel settling more than the right half. The maximum settlement at the crown in Test J2 was approximately 2.5 times that in Test C2.

Figure 8b illustrates the induced longitudinal bending strain in Test C2 and Test J2 after twin-tunnel excavation. In the left half of the existing tunnel, bending moment was expected to be zero near the settlement joint because the settlement joint cannot sustain any bending moment. Maximum sagging moment was induced near the axis of the first new tunnel while maximum hogging moment was induced at a distance of 0.8Z away from the new tunnel axis. The induced maximum positive and negative bending strains were 4.6 and -13.5 µε, respectively. A similar distribution was observed for the right half of the existing tunnel with maximum positive and negative bending strains of 8 and -15.4 µε, respectively. However, in Test C2, sagging moment was observed along most of the existing tunnel (from X/Z= -0.8 to 0.8), which can be attributed to its continuous bending
stiffness. The maximum tensile strain (37.6 \( \mu \varepsilon \)) was 140% larger than that in Test J2.

The longitudinal shear stress can be deduced by differentiating the bending moment, which can be calculated by multiplying flexural rigidity with bending strain. The maximum shear stress was 970 kPa in Test C2 and 260 kPa in Test J2. Assuming that the allowable shear stress for concrete is 660 kPa \( (f'_c = 50 \text{ MPa with a reduction factor of 0.55}) \) based on the code of ACI (2011), the maximum deduced shear stress in Test C2 exceeded the allowable limit, indicating that the tunnel lining may crack due to shear stress.

**Effect of settlement joint on tunnel response in the transverse direction (Tests C2 and J2)**

Figure 9a compares the radial displacement of the existing tunnel in Test C2 and J2. Section A \((X/Z=0)\) was selected as the monitoring section. The positive and negative signs denote radial elongation and radial compression, respectively. The computed results were overestimated by about 20% at the tunnel invert. At the crown, L-springline and R-springline, the computed results matched the measured results well.

In Test C2, the tunnel diameter increased in the vertical direction and correspondingly decreased in the horizontal direction. The measured radial displacements at the crown, invert, L-springline and R-springline were 5, 8.8, -4 and -3.2 mm, respectively. Similarly, the existing tunnel in Test J2 was also elongated vertically and compressed horizontally. The radial displacements at the crown, invert, L-springline and R-springline were 5.2, 8.4, -5 and -4.1 mm, respectively, which were 104%, 95%, 125% and 125% of those in Test C2.

Figure 9b shows the bending strain induced in the existing tunnel in the transverse direction after the twin-tunnel excavation. The positive and negative signs denote tensile and compressive strains at the outer surface, respectively. Computed and measured results shared similar trends except at the tunnel shoulders. Similar trends of the bending strain distribution
were found in Tests C2 and J2. In both tests, positive bending strain was measured at the crown, invert, both shoulders and both knees, whereas negative bending strain was measured at both springlines. This is consistent with the observation that the existing tunnel was vertically elongated and horizontally compressed. The maximum measured bending strain was 76.3 $\mu\varepsilon$ in Test C2 and 12% larger in Test J2 (85.6 $\mu\varepsilon$). In both cases, the maximum bending strain was located at the right corner of the existing tunnel, indicating that the corners might be the most critical parts of the existing tunnel during twin-tunnel undercrossing.

**Conclusions**

Although the design of the tunnel shapes and sizes for centrifuge model tests was based on the actual geometry of the existing Liuyang River tunnel and the newly proposed twin circular tunnels of Changsha Metro line 3, the local complex geological and ground water conditions were not considered for simplicity to focus on the interaction of multiple tunnels in dry sand using the effective stress principle. Thus, the following conclusions drawn may not be directly applicable for engineering practice. The results reported should be treated with great caution.

Based on results from the idealized centrifuge model tests and numerical simulations, the following conclusions may be drawn:

1. Tunnel invert settlement and longitudinal bending strain induced by the second (later) tunnel excavation were 5% and 11% lower than those induced by the first (earlier) tunnel excavation. For an initial design, it may be sufficient to think of the adverse effects caused by the later excavated tunnel as the same as those caused by the earlier one.

2. In the test with a continuous existing tunnel, the tunnel invert showed a U-shaped settlement curve, whereas the tunnel crown showed an inverted U-shaped. The reason for
this difference is that the vertical deformation of the existing tunnel overwhelmed the invert settlement. A possible cause of the large vertical elongation is the large size and shallow cover depth of the existing tunnel.

(3) The presence of a settlement joint substantially reduced the shielding effect of the existing tunnel. The tunnel crown settlement and ground surface settlement were increased by more than 100%, while the maximum longitudinal bending strain and deduced shear stress dropped by more than 60%. This is due to a substantial reduction in the bending stiffness of the jointed existing tunnel.

(4) The bending strain and deformation of the jointed existing tunnel in the transverse direction followed similar trends to those of the continuous tunnel but with slightly larger magnitudes.

(5) The maximum induced longitudinal and transverse bending strains in Test C2 were 37.6 and 76.3 µε, respectively. Cracking was more likely to occur in the transverse direction than in the longitudinal direction. Therefore, strengthening the existing tunnel in the transverse direction would be a good option to meet serviceability requirements. Moreover, the maximum transverse bending strain was found at the corner of the horseshoe-shaped tunnel. Special measures should be considered to strengthen the corners of the horseshoe-shaped tunnel.

Acknowledgements

The authors would like to acknowledge financial support from the Research Grants Council of the HKSAR (General Research Fund project numbers 16207414 and 16208115).
REFERENCES


Boonyarak, T., and Ng, C. W. 2014. Effects of construction sequence and cover depth on crossing-tunnel interaction. Canadian Geotechnical Journal, 52(999), 1-17.


Foundation Engineering (Hamburg, 1997), Balkema (pp. 2353-2385).


Ng, C. W., Shi, J., and Hong, Y. 2013b. Three-dimensional centrifuge modelling of basement excavation effects on an existing tunnel in dry sand. Canadian Geotechnical Journal, 50(8), 874-888.


List of captions

Tables
Table 1 Summary of material parameters adopted in the finite element analysis (Boonyarak and Ng, 2014; Ng et al. 2015)

Figures
Fig. 1 Schematic diagrams showing the intersection of the Liuyang River tunnel and the two tunnels of Changsha Metro line 3
Fig. 2 Schematic diagrams of the centrifuge model package in Test C2: (a) plan view; (b) elevation view; (c) cross section view of tunnels (dimensions in millimetres)
Fig. 3 Simulation of the settlement joint in the: (a) centrifuge model; (b) numerical model
Fig. 4 Invert settlement of the existing tunnel and greenfield subsurface settlement induced by the first and second new tunnel excavations
Fig. 5 Longitudinal bending strain along the invert of the existing tunnel induced by the first and second new tunnel excavations
Fig. 6 Invert and crown settlement of the existing tunnel induced by the twin-tunnel excavations
Fig. 7 Effects of cover depth and size on the settlement of the existing tunnel
Fig. 8 Effect of settlement joint on: (a) ground surface settlement and tunnel crown settlement; (b) longitudinal bending strain of the existing tunnel
Fig. 9(a) Induced radial displacement of the existing tunnel; (b) transverse bending strain of the existing tunnel due to the twin-tunnel excavation
Tables

Table 1 Summary of material parameters adopted in the finite element analysis (Boonyarak and Ng, 2014; Ng et al., 2015)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varphi^*$</td>
<td>Angle of internal shearing resistance at critical state</td>
<td>30°</td>
</tr>
<tr>
<td>$h_s$</td>
<td>Hardness of granulates</td>
<td>2.6 GPa</td>
</tr>
<tr>
<td>$n$</td>
<td>Exponent controlling the shape of limiting void ratio curves</td>
<td>0.27</td>
</tr>
<tr>
<td>$e_{do}$</td>
<td>Minimum void ratio at zero pressure</td>
<td>0.61</td>
</tr>
<tr>
<td>$e_{co}$</td>
<td>Critical void ratio at zero pressure</td>
<td>0.98</td>
</tr>
<tr>
<td>$e_{so}$</td>
<td>Maximum void ratio at zero pressure</td>
<td>1.10</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Exponent controlling the dependency of peak friction angle on relative density</td>
<td>0.5</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Exponent controlling the dependency of soil stiffness on relative density</td>
<td>3</td>
</tr>
<tr>
<td>$m_R$</td>
<td>Parameter controlling initial shear modulus upon 180° strain path reversal</td>
<td>8</td>
</tr>
<tr>
<td>$m_T$</td>
<td>Parameter controlling initial shear modulus upon 90° strain path reversal</td>
<td>4</td>
</tr>
<tr>
<td>$R$</td>
<td>Elastic range</td>
<td>$3 \times 10^{-5}$</td>
</tr>
<tr>
<td>$\beta_r$</td>
<td>Parameter controlling stiffness degradation rate with strain</td>
<td>0.2</td>
</tr>
<tr>
<td>$\chi$</td>
<td>Parameter controlling stiffness degradation rate with strain</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Figures

Fig. 1 Schematic diagrams showing the intersection of the Liuyang River tunnel and the two tunnels of Changsha Metro line 3
Fig. 2 Schematic diagrams of the centrifuge model package in Test C2: (a) plan view; (b) elevation view; (c) cross section view of tunnels (dimensions in millimetres)
Fig. 3 Simulation of the settlement joint in the: (a) centrifuge model; (b) numerical model
Fig. 4 Invert settlement of the existing tunnel and greenfield subsurface settlement induced by the first and second new tunnel excavations
Fig. 5 Longitudinal bending strain along the invert of the existing tunnel induced by the first and second new tunnel excavations
Fig. 6 Invert and crown settlement of the existing tunnel induced by the twin-tunnel excavations
Fig. 7 Effects of cover depth and size on the settlement of the existing tunnel
Fig. 8 Effect of settlement joint on: (a) ground surface settlement and tunnel crown settlement; (b) longitudinal bending strain of the existing tunnel.
Fig. 9 (a) Induced radial displacement of the existing tunnel; (b) transverse bending strain of the existing tunnel due to the twin-tunnel excavation

---

**Change in tunnel radius, ΔR (mm)**

- Measured (C2)
- Computed (C2)
- Measured (J2)
- Computed (J2)

Sign convention:
+ Increase in radius
- Decrease in radius

**Induced bending strain (µε)**

- Measured (C2)
- Computed (C2)
- Measured (J2)
- Computed (J2)

Sign convention:
+ Tensile at outer surface
- Compressive at outer surface