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**Effects of construction sequence and cover depth on
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Abstract: An appropriate construction sequence of crossing tunnels can help minimize the adverse impact on the tunnel that is constructed first (considered as the existing tunnel). However, the influence of construction sequence on crossing-tunnel interaction is complex. Two pairs of three-dimensional centrifuge tests were carried out to investigate the effects of construction sequence on crossing-tunnel interaction. In the first pair of tests, the new tunnel was excavated beneath the existing tunnel in a reference test, while in the other test the new tunnel advanced above the existing tunnel. To study the effects of cover depth on the construction sequence, the depths of the existing and new tunnels were increased in the second pair of tests. An advanced hypoplasticity constitutive model with small strain stiffness was adopted to back-analyze the tests. The existing tunnel was found to be vertically compressed when the new tunnel was excavated underneath but vertically elongated when the new tunnel advanced above. This is because the reduction of stress acting on the existing tunnel in the horizontal direction was larger than that in the vertical direction when the new tunnel was constructed beneath. On the other hand, the decrease in vertical stress on the existing tunnel was larger than the horizontal stress reduction when the new tunnel was excavated above. This behavior was observed in both pairs of tests, irrespective of the cover depths of tunnels. As the cover depths of the existing and new tunnels increased, settlement of the existing tunnel due to the new tunnel construction beneath decreased. This is because with the larger cover depths of tunnels, the increase in the mobilized shear stiffness of soil dominated the increase in stress relief caused by the tunnel excavation.

Keywords: Three-dimensional centrifuge modeling; hypoplasticity constitutive model; construction sequence; cover depth; crossing-tunnel interaction

47 Introduction

48 When two tunnels are excavated across each other, a suitable construction sequence
49 should be adopted to minimize the adverse impact on the tunnel that is constructed first
50 (considered as the existing tunnel). This adverse impact on the existing tunnel is caused by
51 ground displacement and a change in stress induced by the construction of the second tunnel
52 (i.e., the new tunnel). It is thus important to improve the understanding of the effects of a new
53 tunnel excavation on an existing tunnel for different construction sequences.

54 Large settlement of the existing tunnels, tunnel distortion and cracks in the tunnel lining
55 have been recorded in case histories of new tunnel excavations underneath existing tunnels
56 (Cooper et al., 2002; Mohamad et al., 2010; Li and Yuan, 2012). Liu et al. (2011) reported a
57 case history of a new cut-and-cover tunnel constructed above existing tunnels. Factors that
58 influenced heave of the existing tunnels such as the tunnel construction technique and skew
59 angle were investigated by numerical analysis using an elastic-perfectly plastic constitutive
60 model. Although field monitoring data have been reported, variation in ground conditions,
61 tunneling technique, geometry of tunnels and volume loss among each case history caused
62 difficulty in data interpretation.

63 Three-dimensional numerical analyses of perpendicularly crossing tunnels constructed
64 in rock according to different sequences were conducted by Liu et al. (2009). Compressive
65 failure of the concrete lining at the crown of the existing tunnel was induced by the
66 construction of the new tunnel above. In contrast, tensile strain caused cracks to appear in the
67 lining of the existing tunnel at the springlines when the new tunnel was excavated
68 underneath. Even though the effects of tunnel construction sequence on tunnel-tunnel
69 interaction were investigated in their study, the results may not apply to soil as the stiffness
70 dependency on stress, strain and path was not modeled. In addition, as the cover depth of the

existing tunnel varies from one construction sequence to another, the influence of cover depth on the interaction of crossing tunnels should also be further examined.

Marshall et al. (2012) carried out centrifuge tests of a single tunnel excavation in sand in plane strain conditions. They reported that the settlement trough width increased with increasing cover depth-to-diameter ratio (C/D) of the tunnel. As the tunnel excavation was simulated in two dimensions, their results may not apply to crossing tunnels as stress transfer in the longitudinal direction of the tunnel was not simulated.

Crossing-tunnel interaction was studied by Ng et al. (2013) using centrifuge testing and numerical back-analysis. They suggested that stress transfer in the longitudinal direction of the new tunnel caused the existing tunnel above to compress in the vertical direction. However, in their study, the effects of construction sequence and cover depth on the interaction of crossing tunnels were not investigated.

The major objectives of this study are to investigate the effects of construction sequence and C/D on the interaction of perpendicularly crossing tunnels. Two pairs of centrifuge tests and a numerical back-analysis were carried out. Three-dimensional excavation of a new tunnel across an existing tunnel was simulated in-flight. A hypoplasticity constitutive model with small strain stiffness was adopted in the numerical back-analysis. The results are interpreted and discussed.

Centrifuge modeling of crossing-tunnel interaction

Typical setup of a model package

Figure 1a shows the typical plan view of a centrifuge model package. Both existing and new tunnels had an outer diameter (D) equivalent to 6 m in the prototype scale at a centrifugal acceleration of 60 times the earth's gravity (g). The existing tunnel was modeled as "wished-in-place" as the tunnel was empty and both ends were closed to prevent soil from

moving into the tunnel. Reference axes, “X” and “Y”, denote longitudinal and transverse directions of the existing tunnel, respectively. In each test, the new tunnel was excavated either above or beneath the existing tunnel in six excavation stages.

Test program

A total of four tests are reported in this study and the cross-sectional views of the four setups are shown in Figure 1b. All the tests were carried out in a geotechnical centrifuge at the Hong Kong University of Science and Technology (Ng et al., 2001, 2002). In Test E2N3, which is considered as a reference test, the new tunnel was excavated beneath the existing tunnel. The identity of each test denotes the approximate C/Ds of the two tunnels in that test. For the reference test, the C/Ds of the existing and new tunnels were 2 (E2) and 3.5 (N3), respectively. The pillar depth-to-diameter ratio (P/D) was 0.5 in all tests, where the pillar depth is the clear vertical distance between the two tunnels. To study the impact of construction sequence on crossing-tunnel interaction, the new tunnel was constructed above the existing tunnel in Test E3N2. In addition, results from Tests E3N5 and E5N3, in which the existing and new tunnels had larger C/Ds than those in Tests E2N3 and E3N2, were also analyzed to investigate the effects of cover depth on the interaction of perpendicularly crossing tunnels. Note that some of measured results from Test E2N3 have been reported Ng et al. (2013).

Simulation of tunnel excavation in-flight

Tunnel excavation was carried out by simulating the effects of volume and weight losses in-flight using a “Donut” (Ng et al., 2013) as shown in Figure 2a. A “Donut” consists of an outer rubber membrane, an aluminum alloy tube serving as the tunnel lining and an inner rubber bag. During model preparation, the outer rubber membrane and the inner rubber

bag were filled with a heavy fluid (ZnCl_2) having the same density as the soil in each test. The outer rubber membrane simulates the effects of volume loss equivalent to 2%, by regulating the outflow of the heavy fluid inside it. To drain out the heavy fluid from the outer rubber membrane, a valve connected to a drainage tube is controlled in-flight. The inner rubber bag mimics the effects of weight loss inside the tunnel. To simulate the effects of weight loss, the heavy fluid inside the inner rubber bag is also drained away using a valve. The outflow of heavy fluid was collected in a reservoir (see Fig. 1a).

Figure 2b shows a new tunnel and the typical setup of crossing tunnels in each test. Tunnel excavation was carried out in six stages using “Donuts” in each test. The length of excavation in each stage was $0.6D$ or equivalent to 3.6 m in the prototype scale. Each end of the new tunnel was closed to prevent soil movement into the tunnel. In each excavation stage, soil movement towards the tunnel face was not allowed. The lining of each tunnel consisted of an aluminum alloy tube with an outer diameter of 100 mm and thickness of 3 mm. This model tunnel simulated a prototype tunnel that is 6 m in diameter. In order to convert the thickness of the model tunnel lining into the equivalent thickness of a tunnel lining made of concrete, the compressive strength of concrete (f'_c) was assumed to be 50 MPa. Thus, Young’s modulus (E_c) was estimated to be 33 GPa (ACI, 2011). According to Taylor (1995), the flexural stiffness per unit width of tunnel lining and flexural stiffness in the longitudinal direction of the tunnel are scaled by factors of $1/N^3$ and $1/N^4$, respectively, where N is an integer by which gravitational acceleration is to be multiplied. Thus, the equivalent thicknesses of the tunnel lining in the transverse and longitudinal directions of each tunnel are 230 and 420 mm, respectively, in the prototype scale.

146 **Model preparation**

147 All four tests were carried out in dry Toyoura sand. The mean particle size (D_{50}),
148 maximum void ratio ($e_{\max}$), minimum void ratio ($e_{\min}$), specific gravity (G_s) and friction angle
149 at the critical state (ϕ_c) of the sand are 0.17 mm, 0.977, 0.597, 2.64, and 30° , respectively
150 (Ishihara, 1993). According to Garnier et al. (2007), the tunnel diameter and mean soil
151 particle size ratio (D/D_{50}) should be larger than 175 in order to minimize the effects of
152 particle size on tunnel-soil interaction. In this study, this ratio was 588.

153 To prepare the soil sample in each test, a dry pluviation technique was adopted with a
154 drop height and rate of pluviation of 500 mm and 100 kg/hour, respectively. The density
155 achieved in Tests E2N3, E3N2, E3N5 and E5N3 were 1529 kg/m^3 (relative density $D_r =$
156 64%), 1534 kg/m^3 ($D_r = 66\%$), 1529 kg/m^3 ($D_r = 64\%$) and 1541 kg/m^3 ($D_r = 68\%$),
157 respectively.

159 **Instruments on the existing tunnel**

160 Typical instruments installed on the existing tunnel are shown in Figure 3a. Strain
161 gages were attached along the crown and invert in the longitudinal direction of the existing
162 tunnel with typical spacing of 50 mm. The strain gages were connected in a full Wheatstone
163 bridge in order to compensate for changes in temperature.

164 The transverse cross-sectional view at the center of the existing tunnel is shown in
165 Figure 3b. Eight sets of strain gages were installed at an evenly spacing of 45° . Each set of
166 strain gages, connected in a full Wheatstone bridge, were attached to the outer and inner
167 surfaces of the tunnel lining. A total of four potentiometers were used to measure tunnel
168 deformation. A potentiometer is a resistor which relates a change in electrical resistance to
169 the displacement of a measuring rod (Todd, 1975). The accuracy of a potentiometer was
170 estimated to be equivalent to $\pm 1 \text{ mm}$ in the prototype scale. The potentiometers were fixed to

a plate which was in turn screwed to a frame that was connected to the tunnel lining at two places at distances of 3D and 5D from the center of the existing tunnel (see Fig. 3a).

Figure 3c shows extension rods for linear variable differential transformers (LVDTs) placed along the crown of the existing tunnel to measure vertical displacement. Each extension rod consisted of an inner and an outer aluminum alloy tube. One end of the inner tube was placed on the crown of the existing tunnel while the other end was connected to an LVDT. The outer tube encased the inner one in order to minimize the friction from the surrounding soil.

Test procedure

Each model package was prepared following the procedures discussed above. Afterwards, the centrifuge was gradually spun up to reach an acceleration of 60 times the earth's gravity. Initial readings were taken from all transducers once they had become stable. Then, tunnel excavation using "Donuts" was carried out in six consecutive stages. In each excavation stage, sufficient time was provided for the readings to become stable. The centrifuge was spun down after the completion of tunnel excavation.

Three-dimensional numerical analysis

To back-analyze the results from the centrifuge tests, a three-dimensional finite element software PLAXIS 3D 2013 (Brinkgreve et al., 2013) was adopted.

Finite element mesh and boundary conditions

Figure 4a shows the three-dimensional finite element mesh of a new tunnel excavation above an existing tunnel (Test E3N2). The dimensions of the mesh and the geometry of the tunnels were identical to those in the centrifuge tests. Owing to symmetry, only half of the

entire mesh was required. Thus, a plane of symmetry was defined at the centerline of the new tunnel ($X/D = 0$). Roller and pin supports were applied to the four vertical sides and the base of the mesh, respectively. Soil was modeled using a 10-node tetrahedral element and the tunnel lining using a 6-node plate element.

Some details of crossing tunnels are shown in Figure 4b. At the plane of symmetry ($X/D = 0$), additional constraints were imposed on the tunnel lining. By imposing these constraints, rotation of the lining around the longitudinal and vertical axes of the new tunnel was not allowed (ϕ_y and $\phi_z = 0$), as was translational movement in the transverse direction of the new tunnel ($u_x = 0$).

Constitutive model and model parameters

A hypoplastic constitutive model (Gudehus et al., 2008) was adopted to represent Toyoura sand. Hypoplasticity models have been developed to simulate the response of coarse-grained soils (Kolymbas, 1991; Gudehus, 1996; von Wolffersdorff, 1996; Wu et al., 1996; Gudehus and Mašín, 2009). Like many other advanced soil models, the hypoplasticity constitutive model is able to capture soil behavior related to stress state and history. More importantly, it is capable of modelling strain-dependency and path-dependency of soil stiffness at small strains (Niemunis and Herle, 1997).

Herle and Gudehus (1999) reported some model parameters (ϕ_c , h_s , n , e_{d0} , e_{c0} and e_{i0}) based on their calibration results in Toyoura sand. To determine exponents α , β and small strain stiffness parameters (m_R , m_T , R , β_r and χ), test results obtained from local strain measurement and bender elements (Yamashita et al., 2000, 2009) were numerically fitted using a “soil test” (a utility available in PLAXIS). The model parameters are summarized in Table 1.

The tunnel lining was modeled using a linear elastic material. Young's modulus, density and Poisson's ratio of the tunnel lining were assumed to be 69 GPa, 2700 kg/m³ and 0.33, respectively.

Numerical modeling procedures

Drained effective stress analysis was performed in all numerical runs as the tests were carried out in dry sand. The procedures of numerical back-analysis followed that in the centrifuge tests. In the initial stage, the unit weight of soil and void ratio under the 1g conditions were specified. The linings of the existing and new tunnels were modeled as "wished-in-place" as they were activated in the initial stage. In addition, the soil inside the existing tunnel and some of the soil inside the new tunnel were removed or deactivated (see Fig. 4a). To initialize stress in soil, the at-rest earth pressure coefficient (K_0) of 0.5 (using the empirical equation, $K_0 = 1 - \sin \phi'$) was adopted. Rise in centrifugal acceleration was simulated by increasing gravity by 60 times ($\Sigma M_{\text{weight}} = 60$). Then, excavation of the new tunnel was carried out by modeling the effects of volume and weight losses. For effects of volume loss, which were equivalent to 2%, a "surface contraction" (a utility in PLAXIS) was applied on the surface of the new tunnel. The surface contraction technique simulated a uniform radial contraction of the tunnel lining in each section. At the common nodes or interface of each section, the magnitude of contraction was half of that in the middle of the section. The contraction gradually increased from the interface to the middle part of each section. Another half of the contraction at the interface of the section was simulated in the following excavation stage. By doing so, there is no double contraction simulated during the numerical modelling procedures. To model the effects of weight loss, the soil elements inside the new tunnel were deactivated. The face of the new tunnel was supported using rollers. These rollers were applied simultaneously with soil removal (deactivation of soil elements) at

each stage. This means that end conditions simulated by numerical and centrifuge modeling are consistent. The tunnel excavation in the subsequent five stages followed the procedures described above.

Interpretation of results

Measured and computed results reported in this study are in the prototype scale unless otherwise stated.

Vertical displacement and gradient of the existing tunnel

Figure 5 shows vertical displacement of the existing tunnel at the end of excavation. The vertical displacement is normalized by the diameter of the new tunnel to observe the general trend along with those in the case histories. The positive and negative signs denote heave and settlement of the existing tunnel, respectively. By expecting symmetrical tunnel settlements, the extension rods in Tests E2N3 and E3N5 were only placed on one half of the existing tunnel (i.e., from $X/D = 0$ to 4). In Test E5N3, there was no measurement of vertical displacement of the existing tunnel.

In Test E2N3, where the new tunnel was excavated beneath the existing tunnel, the maximum measured tunnel settlement was $0.3\%D$ (18 mm). On the other hand, heave of the existing tunnel occurred due to the new tunnel construction above in Test E3N2. At offset distances from $1D$ to $6D$ from the centerline of the new tunnel, settlement of the existing tunnel occurred. This is because additional stress induced due to stress redistribution at these distances. It is evident that stress redistribution took place in soil to maintain equilibrium. Detailed explanations of stress re-distribution are given in the section under “Mobilized vertical stress of soil acting on the existing tunnel”. The magnitude of the measured vertical displacement of the existing tunnel caused by the new tunnel excavation beneath (Test E2N3)

was much larger than that when the new tunnel advanced above (Test E3N2). This is because the reduction in vertical stress acting on the existing tunnel was larger in the former test than in the latter. In addition, the stiffness of the soil around the existing tunnel was smaller in Test E2N3 than Test E3N2 (to be discussed in the section under “Mobilized soil stiffness due to tunnel excavation”).

The computed results were consistent with the measured ones in both tests with minor discrepancies due likely to the use of model parameters adopted from the literature and empirical relationships. For Test E3N2, the computed heave of the existing tunnel was higher than the measured one. One possible reason was that an idealized uniform radial contraction was applied on the tunnel lining in the numerical simulation. However, in the centrifuge test, ground movement around the new tunnel may not be uniform.

The behavior of the existing tunnel in Tests E3N5 and E5N3 was similar to that in Tests, E2N3 and E3N2 even though the C/Ds of the existing and new tunnels in the former two tests were 1.5 larger than those in the latter two tests. In the tests where the new tunnel was excavated underneath the existing tunnel, tunnel settlement decreased as the C/Ds of tunnels increased (i.e., settlement in Test E3N5 was smaller than that in Test E2N3). This is because the stiffness of soil around the existing tunnel was larger in the deeper tunnel test than in the shallower tunnel test, even though the effects of stress relief on the existing tunnel were larger in the former case than in the latter. However, when the new tunnel advanced above the existing tunnel, heave of the existing tunnel increased with increasing C/D (i.e., heave in Test E5N3 was larger than that in Test E3N2). In these two tests, the effects of stress relief, which increased with depth, dominated the rise in the stiffness of soil around the existing tunnel. Further explanations are given later.

Allowable tunnel displacements less than 15 mm and 20 mm were recommended by LTA (2000) and BD (2009), respectively. Settlement of the existing tunnel measured in Tests

E2N3 and E3N5 exceeded the limit suggested by LTA (2000) while heave of the existing tunnel in Tests E3N2 and E5N3 was still within the allowable limits.

Table 2 summarizes the maximum induced tunnel gradient in each test. Similar to the tunnel vertical displacement, the gradient of the existing tunnel induced by the new tunnel excavation beneath (Tests E2N3 and E3N5) was significantly larger than that due to the new tunnel advancement above (Tests E3N2 and E5N3). The induced tunnel gradient in each test did not exceed the recommended limit of 1:1000 (LTA, 2000; BD, 2009).

Induced strain and shear stress in the longitudinal direction of the existing tunnel

Induced strain along the invert in the longitudinal direction of the existing tunnel at the end of tunnel excavation is shown in Figure 6. The positive and negative signs denote induced tensile and compressive strain at the invert in the tunnel lining, respectively.

In Test E2N3, the maximum measured settlement of the existing tunnel occurred at the location directly above the new tunnel (refer to Figure 5). At this location, measured tensile strain was induced in the tunnel lining at the invert, i.e., sagging zone as shown in Figure 6. At a distance more than $2.5D$ away from the centerline of the new tunnel, compressive strain and tensile strain was induced at the invert and at the crown, respectively. This location is considered as a hogging zone. In Test E3N2, however, compressive strain was induced at the center of the existing tunnel while the induced tensile strain was observed at some distance away. The magnitude of maximum induced strain in Test E2N3 was much larger than that in Test E3N2. This induced strain in the two tests was consistent with the tunnel vertical displacement (refer to Fig. 5). The computed results agreed reasonably well with the measured ones. The possible reason for the minor discrepancies is as mentioned above.

The direction and shape of the induced strain in the tunnel lining in Tests E3N5 and E5N3 were similar to those in Tests E2N3 and E3N2, respectively, even though the C/Ds of

the existing and new tunnels were larger in the latter two tests than in the former two. The computed maximum induced tensile strain and compressive strain in Test E3N5 were slightly larger than those in Test E2N3. Although settlement of the existing tunnel was larger at a shallower depth (i.e., in Test E2N3) than at a greater depth (i.e., in Test E3N5) (see Fig. 5), the induced strain was nevertheless slightly larger in the latter case. Similarly, for the new tunnel excavation above the existing tunnel, the induced maximum compressive strain at a greater depth (i.e., in Test E5N3) was larger than that at a shallower depth (i.e., in Test E3N2). This is because of the greater vertical stress reduction of soil around the existing tunnel in Tests E3N5 and E5N3 (with the larger C/Ds) than in Tests E2N3 and E3N2 (with the smaller C/Ds). Further details are given later.

According to ACI (2001), the cracking tensile strain of unreinforced concrete is $150 \mu\epsilon$. The maximum tensile strain induced in the existing tunnel by the construction of a new tunnel underneath (Tests E2N3 and E3N5) exceeded the cracking tensile strain.

Table 3 gives the maximum shear stress induced on the tunnel lining in each test. The values were deduced from the slope of the strain induced in the longitudinal direction of the existing tunnel using beam theory. The allowable shear stress of concrete is estimated to be 660 kPa if the concrete compressive strength (f'_c) is 50 MPa and a reduction factor of 0.55 is adopted (ACI, 2011). The maximum shear stress induced on the existing tunnel by the new tunnel advancement beneath (Tests E2N3 and E3N5) exceeded the allowable shear stress. Judging from the maximum induced tensile strain and the maximum induced shear stress observed, cracks may appear in the lining of the existing tunnel due to the new tunnel construction beneath.

Mobilized soil stiffness due to tunnel excavation

Figure 7 shows the mobilized secant shear modulus of soil along the crown and invert of the existing tunnel before and after the new tunnel excavation. The mobilized secant shear modulus (G_m) was obtained from the computed deviatoric stress (q) and the computed deviatoric strain (ϵ_s) according to the equation $G_m = q/3\epsilon_s$.

The mobilized shear modulus of soil along the crown of the existing tunnel is shown in Figure 7a. In Test E2N3, G_m was about 11 MPa before the tunnel excavation. After the tunnel excavation, G_m only changed slightly as the influence of the new tunnel excavation on soil stiffness was minimized by the presence of the existing tunnel. The increase in G_m after tunnel excavation within a distance of 1D from the centerline of the new tunnel is mainly because of increased confining stress resulting from stress redistribution in soil. This increase of stress at the crown is discussed later in the section “Incremental normal stress acting on the existing tunnel”.

G_m before tunnel excavation was larger in Test E3N2 than in Test E2N3 as the G_m increased with confining pressure (which in turn increased with depth). However, after the construction of the new tunnel above (Test E3N2), G_m reduced sharply at the center of the existing tunnel ($X/D = 0$). As expected, G_m before tunnel excavation was larger in Test E3N5 (with the larger C/D s) than in Test E2N3 (with the smaller C/D s). After the excavation of the new tunnel underneath, there was a slight reduction in G_m along the crown. In Test E5N3, G_m before and after tunnel excavation were larger than those in Test E3N2.

In Figure 7b the mobilized shear modulus of soil along the invert of the existing tunnel is illustrated. To compare the vertical displacement of the existing tunnel across the tests, the mobilized shear modulus of soil on the side of the existing tunnel facing the new tunnel is considered. It can be seen that before tunnel excavation, G_m at the crown was larger than that at the invert of the existing tunnel in every test. This is because of the presence of the existing

tunnel, causing reduction in G_m at the invert of the existing tunnel. At any location further away from the existing tunnel, G_m increased with depth.

G_m both before and after tunnel excavation along the crown in Test E3N2 (refer to Fig. 7a) were larger than those along the invert in Test E2N3 (see Fig. 7b). As a result, the magnitude of tunnel heave in Test E3N2 was significantly smaller than the magnitude of tunnel settlement in Test E2N3 (see Fig. 5). G_m along the invert of the existing tunnel in Test E3N5 (with the larger C/Ds) was larger than that in Test E2N3 (with the smaller C/Ds). This resulted in a smaller settlement of the existing tunnel in Test E3N5 than in Test E2N3. However, heave of the existing tunnel in Test E5N3 was larger than that in Test E3N2 as stress relief was a dominating factor (to be discussed later).

Deformation of the existing tunnel

Figure 8 shows the deformation of the existing tunnel in terms of the change in the normalized tunnel radius ($\Delta R/R_0$, where ΔR is the change in the tunnel radius and R_0 is the undeformed tunnel radius, which is 3m). Note that the change in tunnel radius is a relative displacement between the tunnel lining and the centroid of the existing tunnel. The deformations at the center of the existing tunnel ($X/D = 0$) when the excavated section of the new tunnel reached the centerline of the existing tunnel ($Y/D = 0.3$) and at the end of tunnel excavation ($Y/D = 1.5$) are reported. The positive and negative signs denote an increase and a decrease in the normalized tunnel radius, respectively.

Figure 8a shows a decrease in radius at the left springline and invert but an increase in radius at the crown and right springline of the existing tunnel as measured in Test E2N3. The combined results of changes in radius at the crown, springlines and invert revealed that the existing tunnel was elongated horizontally. This is because the reduction in stress caused by the new tunnel excavation, not only resulted in a reduction in vertical stress but it also caused

a significant stress relief at each springline of the existing tunnel. By comparing the locations of stress relief on the existing tunnel, it is found that stress reduction in the horizontal direction was larger than that in the vertical direction. This means that the existing tunnel elongated horizontally. Further explanation of stress changes is given in the section under “Incremental normal stress acting on the existing tunnel”. In Test E3N2, however, the tunnel radius at the crown and left springline increased while the tunnel radius at the right springline and invert decreased according to measurements taken. The combined result was elongation in the vertical direction and compression in the horizontal direction. This is because the reduction in stress in the horizontal direction was larger than that in the vertical direction in Test E2N3, while stress decrease was larger in the vertical direction than in the horizontal direction in Test E3N2. Further explanation is given later. The computed results were in an agreement with the measured ones with only minor discrepancy, the reason for which has been mentioned in the section “Vertical displacement and gradient of the existing tunnel”. Another possible reason for the discrepancy between the measured and computed results is that numerical simulation of volume loss may not represent a perfect match to that in the centrifuge tests. An idealized uniform volume contraction was imposed in the numerical simulation. However, this might not be achieved in centrifuge model tests.

The existing tunnel experienced vertical compression in Test E3N5 as it did in Test E2N3, but vertical elongation in Test E5N3 as it did in E3N2. This shows that an increase in the C/Ds of the existing and new tunnels does not affect how the existing tunnel would deform.

Tunnel deformation at the end of the new tunnel excavation is shown in Figure 8b. As measured in Test E2N3, the reduction in tunnel diameter in the vertical direction and the increase in tunnel diameter in the horizontal direction were larger than those when $Y/D = 0.3$ (refer to Fig. 8a). This suggests that the vertical compression of the existing tunnel at the end

of tunnel excavation was larger than that when the excavated section of the new tunnel was directly underneath the existing tunnel. Similarly, the elongation of the existing tunnel in the vertical direction in Test E3N2 was larger at the completion of the new tunnel than when the excavated section of the new tunnel was directly above the existing tunnel ($Y/D = 0.3$).

According to BTS (2000), the maximum difference between the maximum and minimum diameters of the tunnel should be within 2% [i.e., $(D_{\max} - D_{\min})/D_0 \leq 2\%$, where $D_{\max}$, $D_{\min}$ and D_0 are the maximum, minimum and undeformed tunnel diameters, respectively]. The deformation of the existing tunnel induced by the new tunnel excavation did not exceed this allowable limit in any of the tests. However, given that the at-rest earth pressure coefficient (K_0) was estimated to be 0.5, the existing tunnel was vertically compressed due to overburden pressure even before the new tunnel excavation. The construction of the new tunnel beneath only served to compress the existing tunnel even more in Tests E2N3 and E3N5.

Although the shape of model tunnel deformed into an oval shape as the centripetal acceleration increased to 60g, the deviation ($\Delta D/D_0$) of it from a circular shape (like the one in the field) is less than 0.6%. A separate numerical simulation using a circular model tunnel (i.e., not oval shape at 60g) was carried out. It is found that the general trends of induced strain in the tunnel lining in the transverse direction and incremental normal stress acting on the existing tunnel are very similar for both oval and circular tunnel cases.

Induced strain in the transverse direction of the existing tunnel

Induced strain measured at the outer surface at the center of the existing tunnel ($X/D = 0$) in the transverse direction is shown in Figure 9. The measurements were taken when the excavated section reached the existing tunnel ($Y/D = 0.3$) and at the end of the new tunnel

444 excavation ($Y/D = 1.5$). The positive and negative signs denote induced tensile and induced
445 compressive strain, respectively.

446 Figure 9a shows that in Test E2N3, compressive strain was induced at the left shoulder,
447 right knee and invert while the crown, right shoulder, springlines and left knee experienced
448 tensile strain. In Test E3N2, tensile strain was induced at the crown, right shoulder, knees and
449 invert while the left shoulder and springlines experienced compressive strain. In Test E3N5,
450 compressive strain was induced at the crown, knees and invert while tensile strain was
451 observed at the shoulders and springlines. In Test E5N3, tensile strain was observed at the
452 crown and invert, but compressive strain occurred at the shoulders, springlines and knees.
453 The induced strains in all four tests were consistent with the deformation of the existing
454 tunnel (refer to Fig. 8a).

455 The induced strain at the end of the new tunnel excavation is shown in Figure 9b. In the
456 tests where the new tunnel was excavated beneath the existing tunnel (Tests E2N3 and
457 E3N5), a larger induced tensile strain at the springlines and a larger compressive strain at the
458 crown and invert than when $Y/D = 0.3$ (see Fig. 9a) were observed. In Tests E3N2 and E5N3,
459 induced compressive strain at the springlines of the existing tunnel was larger at the end of
460 tunnel excavation than when the new tunnel was directly above the existing tunnel ($Y/D =$
461 0.3). The induced strains confirm that vertical compression in Test E2N3 was larger at the
462 end of tunnel excavation than when the excavated section of the new tunnel reached the
463 existing tunnel (refer to Figs. 8a and 8b). Similarly, the vertical elongation of the existing
464 tunnel in Test E3N2 was larger at the completion of the new tunnel than at $Y/D = 0.3$.

465 The induced tensile strain in the transverse direction of the existing tunnel did not
466 exceed the cracking tensile strain of unreinforced concrete of $150 \mu\epsilon$ (ACI, 2001). However,
467 in Tests E2N3 and E3N5, tensile strain was present at both springlines due to overburden
468 pressure of soil even before tunnel excavation, as K_0 was estimated to be 0.5. Thus, the

tensile strain induced by the new tunnel excavation may still cause cracks to appear in the lining of the existing tunnel.

Ye et al. (2014) carried out model testing by applying a point load on tunnel linings with and without radial (transverse) joints. They reported that the tunnel linings with joints deformed larger but induced smaller transverse bending moment than that without joints. By adopting their finding, it is possible that the deformation of the existing tunnel in this study (without joints) may be underestimated that in the field. On the other hand, induced strains in the transverse direction of the existing tunnel in this study should be larger than that of jointed tunnel linings and hence more conservative.

Incremental normal stress acting on the existing tunnel

Figure 10 shows the incremental normal stress at the center of the existing tunnel ($X/D = 0$). The positive and negative signs denote an increase and a decrease in normal stress, respectively, compared with when the centrifugal acceleration reached 60g.

The incremental normal stress when the excavated section reached the centerline of the existing tunnel ($Y/D = 0.3$) is shown in Figure 10a. In Test E2N3, a significant reduction in normal stress occurred at the left knee and invert as the excavated section of the new tunnel approached the existing tunnel. A decrease in stress was also observed at the springlines and right knee while an increase in stress took place at the crown and shoulders. The stress increase was due to a redistribution of stress to maintain equilibrium. In Test E3N2, normal stress reduced at the crown, shoulders, springlines and left knee but increased slightly at the invert of the existing tunnel. Even with the larger C/D s of the existing and new tunnels in Tests E3N5 and E5N3, normal stress exhibited trends similar to those in Tests E3N5 and E5N3, respectively, differing only in magnitude. The reduction in normal stress at the springlines, knees and invert in Test E3N5 was larger than that in Test E2N3. Similarly, large

stress reductions at the crown, left shoulder, left springline and left knee in Tests E5N3 than E3N2 were observed. Further explanations are given later.

The results at the end of tunnel excavation ($Y/D = 1.5$) are shown in Figure 10b. Comparing these results with those obtained when $Y/D = 0.3$, there was a significant reduction in normal stress at the right knee in Tests E2N3 and E3N5. Similarly, the decrease in normal stress in Tests E3N2 and E5N3 at the right shoulder at the end of tunnel excavation was much larger than that when $Y/D = 0.3$.

For the new tunnel excavation beneath the existing tunnel, the reduction in normal stress at the invert and knees of the existing tunnel increased with increasing C/Ds of the existing and new tunnels. Stress reduction at the invert in Test E3N5 was greater than that in Test E2N3. By using the beam theory, computed tensile strain in the lining at the invert of the existing tunnel in the former was larger than that in the latter case (see Fig. 6). Similarly, computed compressive strain at the crown in the former was also greater than that in the latter case.

For the case when the new tunnel was excavated underneath the existing tunnel, the reduction in vertical stress at the invert was greater with an increasing C/D of existing tunnel from 2 to 3.5. The reduction in vertical stress in Tests E2N3 and E3N5 were 147 kPa and 195 kPa, respectively. As a result of stress reduction, the decrease in stiffness was larger with an increasing C/D of tunnel (refer to Fig. 7b). For Tests E2N3 and E3N5, a decrease in mobilized shear modulus due to tunneling was 7.0 MPa and 9.4 MPa, respectively. The effects of reduction in stress and stiffness in Test E2N3 were greater than that in Test E3N5, resulting in larger tunnel settlement in the former than the latter test (see Fig. 5).

The normal stress reduction at the crown and shoulders of the existing tunnel when the new tunnel was constructed above was larger at a greater depth (E5N3) than at a shallower

depth (E3N2). As a result, larger induced strain in the longitudinal direction of the existing tunnel was observed in Test E5N3 than in Test E3N2 (refer to Fig 6).

In the tests where the new tunnel was constructed above the existing tunnel, the decreases in vertical stress at the crown of the existing tunnel in Tests E3N2 and E5N3 were 161 kPa and 202 kPa, respectively. The decreases in stress resulted in stiffness reduction at the crown in Tests E3N2 and E5N3 by 13.0 MPa and 16.1 MPa, respectively (see Fig. 7a). The effects of decrease in stress and stiffness in Test E3N2 was less than that in Test E5N3, resulting in smaller tunnel heave in the former test than the latter (refer to Fig. 5).

The difference between incremental stress in the horizontal and vertical directions is one of the factors influencing the deformation and induced strain in the transverse direction of the existing tunnel. At the end of tunnel excavation in Tests E2N3 and E3N5, the reduction in stress in the horizontal direction was larger than that in the vertical direction. As a result, the existing tunnel was compressed vertically in these two tests (refer to Figs. 8b and 9b). On the other hand, the decrease in stress in the vertical direction was larger than that in the horizontal direction in Tests E3N2 and E5N3, resulting in the vertical elongation of the existing tunnel.

BD (2009) suggested that the change in stress acting on a tunnel lining should be within 20 kPa. In Tests E2N3 and E3N5, the reduction in normal stress at the springlines, knees and invert as well as the increase in normal stress at the crown all exceeded the recommended limit. For the tests where the new tunnel advanced above the existing tunnel (E3N2 and E5N3), the decrease in normal stress at the crown, shoulders, springlines and knees all exceeded the allowable stress. This suggests that a structural analysis of the existing tunnel considering changes in stress should be undertaken. In addition, as the maximum change in normal stress (e.g., stress reduction at the invert in Tests E2N3 and E3N5 as well as at the crown in Tests E3N2 and E5N3) occurred during the advancement of the new tunnel but not

at the end of tunnel excavation, analysis of crossing-tunnel interaction should be carried out three-dimensionally.

Mobilized vertical stress of soil acting on the existing tunnel

The ratio of vertical stress after tunnel excavation ($\sigma'_{v,after}$) to that before construction of the new tunnel (σ'_{v0}) is referred to as the mobilized vertical stress ratio and is shown in Figure 11. The ratio is considered for soil on the side of the existing tunnel facing the new tunnel (along the invert in Tests E2N3 and E3N5 and along the crown in Tests E3N2 and E5N3).

The minimum mobilized vertical stress ratio of 0.33 was found in Test E2N3 at the center of the existing tunnel ($X/D = 0$). This ratio increased with distance from the center of the existing tunnel. In Test E3N2, the reduction in the mobilized vertical stress ratio was smaller than that in Test E2N3 within the distance of $1D$ from the center of the existing tunnel ($X/D \leq 1$). This smaller reduction in the mobilized vertical stress ratio and larger soil stiffness (refer to Fig. 7) resulted in significantly smaller vertical displacement and strain induced in the lining of the existing tunnel in Test E3N2 than in Test E2N3 (see Figs. 5 and 6).

Despite the larger C/D s of the existing and new tunnels in Test E3N5, the mobilized vertical stress ratio in this test behaved in a similar manner to that in E2N3. The minimum vertical stress ratio in both tests was about 0.35. Similarly, the mobilized vertical stress ratio in Test E5N3 was almost identical to that in Test E3N2 with a minimum stress ratio of about 0.45. Thus, the reduction in stress acting on the existing tunnel in the tests with the larger C/D s was greater than that in the tests with the smaller C/D s (refer to Fig. 10b) as the confining pressure increased with depth. This suggests that the mobilized vertical stress ratio

is influenced by the effects of construction sequence but is independent of the effects of cover depth on the crossing-tunnel interaction.

Summary and conclusions

Two pairs of centrifuge tests were carried out to examine crossing-tunnel interaction. The effects of construction sequence on crossing-tunnel interaction were investigated in the first pair of tests, where a new tunnel was excavated beneath an existing tunnel in a reference test, while the new tunnel advanced above in the other test. The second pair of tests were conducted with larger cover depth-to-tunnel diameter ratios (C/D_s) than in the first pair to study the effects of cover depth on the construction sequence in crossing tunnels. In order to improve the understanding of the interaction of crossing tunnels, a hypoplasticity constitutive model with small strain stiffness was adopted to back-analyze the tests. It should be pointed out that like any numerical simulation, centrifuge modelling is very useful for comparative study among tests with almost identical geometrical and ground conditions. The direct use of any numerical simulation and centrifuge test results to any design, however, should be cautious. Based on the interpretation of measured and computed results, the following conclusions may be drawn:

- (a) The settlement of the existing tunnel when the new tunnel was excavated beneath was significantly larger than the heave of the existing tunnel when the new tunnel was constructed above. This is because the reduction in vertical stress acting on the existing tunnel was larger in the former case than in the latter. In addition, the mobilized shear stiffness of soil around the existing tunnel was smaller in the former tests than in the latter, regardless of the C/D_s of the crossing tunnels.
- (b) When the new tunnel was excavated beneath, the existing tunnel was compressed vertically. However, the existing tunnel was elongated in the vertical direction due to the

new tunnel construction above. This is because the reduction in stress acting on the existing tunnel in the horizontal direction was larger than that in the vertical direction when the new tunnel was excavated underneath. On the other hand, the decrease in stress acting on the existing tunnel in the vertical direction was larger than that in the horizontal direction due to the new tunnel advancement above, irrespective of the C/Ds of tunnels.

- (c) The differential vertical displacement of the existing tunnel induced by the new tunnel excavated underneath was larger than when the new tunnel was constructed above it. As a result, induced tensile strain at the invert and shear stress in the lining of the exiting tunnel in the former was larger than that in the latter tunnel excavation sequence. By using the beam theory, computed compressive strain at the crown in the former was also greater than that in the latter. The tensile strain and shear stress in the existing tunnel induced by the new tunnel construction beneath exceeded the cracking tensile strain (ACI, 2001) and allowable shear stress (ACI, 2011), respectively.

- (d) Settlement of the existing tunnel due to the new tunnel excavation underneath in the test with the larger C/Ds was less than that in the test with the smaller C/Ds. This is because the mobilized shear stiffness of soil around the existing tunnel increased with depth, even though the reduction in vertical stress acting on the existing tunnel in the test with the larger C/Ds was greater than that in the test with the smaller C/Ds. On the other hand, heave of the existing tunnel caused by the new tunnel construction above increased with increasing C/Ds of the existing and new tunnels due to a larger relief of stress acting on the existing tunnel at a greater depth than at a shallower depth. In addition, the increase in stress relief dominated the increase in the mobilized shear stiffness of soil.

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Effects of construction sequence and cover depth on crossing-tunnel interaction

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Abstract: An appropriate construction sequence of crossing tunnels can help minimize the adverse impact on the tunnel that is constructed first (considered as the existing tunnel). However, the influence of construction sequence on crossing-tunnel interaction is complex. Two pairs of three-dimensional centrifuge tests were carried out to investigate the effects of construction sequence on crossing-tunnel interaction. In the first pair of tests, the new tunnel was excavated beneath the existing tunnel in a reference test, while in the other test the new tunnel advanced above the existing tunnel. To study the effects of cover depth on the construction sequence, the depths of the existing and new tunnels were increased in the second pair of tests. An advanced hypoplasticity constitutive model with small strain stiffness was adopted to back-analyze the tests. The existing tunnel was found to be vertically compressed when the new tunnel was excavated underneath but vertically elongated when the new tunnel advanced above. This is because the reduction of stress acting on the existing tunnel in the horizontal direction was larger than that in the vertical direction when the new tunnel was constructed beneath. On the other hand, the decrease in vertical stress on the existing tunnel was larger than the horizontal stress reduction when the new tunnel was excavated above. This behavior was observed in both pairs of tests, irrespective of the cover depths of tunnels. As the cover depths of the existing and new tunnels increased, settlement of the existing tunnel due to the new tunnel construction beneath decreased. This is because with the larger cover depths of tunnels, the increase in the mobilized shear stiffness of soil dominated the increase in stress relief caused by the tunnel excavation.

Keywords: Three-dimensional centrifuge modeling; hypoplasticity constitutive model; construction sequence; cover depth; crossing-tunnel interaction

47 Introduction

48 When two tunnels are excavated across each other, a suitable construction sequence
49 should be adopted to minimize the adverse impact on the tunnel that is constructed first
50 (considered as the existing tunnel). This adverse impact on the existing tunnel is caused by
51 ground displacement and a change in stress induced by the construction of the second tunnel
52 (i.e., the new tunnel). It is thus important to improve the understanding of the effects of a new
53 tunnel excavation on an existing tunnel for different construction sequences.

54 Large settlement of the existing tunnels, tunnel distortion and cracks in the tunnel lining
55 have been recorded in case histories of new tunnel excavations underneath existing tunnels
56 (Cooper et al., 2002; Mohamad et al., 2010; Li and Yuan, 2012). Liu et al. (2011) reported a
57 case history of a new cut-and-cover tunnel constructed above existing tunnels. Factors that
58 influenced heave of the existing tunnels such as the tunnel construction technique and skew
59 angle were investigated by numerical analysis using an elastic-perfectly plastic constitutive
60 model. Although field monitoring data have been reported, variation in ground conditions,
61 tunneling technique, geometry of tunnels and volume loss among each case history caused
62 difficulty in data interpretation.

63 Three-dimensional numerical analyses of perpendicularly crossing tunnels constructed
64 in rock according to different sequences were conducted by Liu et al. (2009). Compressive
65 failure of the concrete lining at the crown of the existing tunnel was induced by the
66 construction of the new tunnel above. In contrast, tensile strain caused cracks to appear in the
67 lining of the existing tunnel at the springlines when the new tunnel was excavated
68 underneath. Even though the effects of tunnel construction sequence on tunnel-tunnel
69 interaction were investigated in their study, the results may not apply to soil as the stiffness
70 dependency on stress, strain and path was not modeled. In addition, as the cover depth of the

existing tunnel varies from one construction sequence to another, the influence of cover depth on the interaction of crossing tunnels should also be further examined.

Marshall et al. (2012) carried out centrifuge tests of a single tunnel excavation in sand in plane strain conditions. They reported that the settlement trough width increased with increasing cover depth-to-diameter ratio (C/D) of the tunnel. As the tunnel excavation was simulated in two dimensions, their results may not apply to crossing tunnels as stress transfer in the longitudinal direction of the tunnel was not simulated.

Crossing-tunnel interaction was studied by Ng et al. (2013) using centrifuge testing and numerical back-analysis. They suggested that stress transfer in the longitudinal direction of the new tunnel caused the existing tunnel above to compress in the vertical direction. However, in their study, the effects of construction sequence and cover depth on the interaction of crossing tunnels were not investigated.

The major objectives of this study are to investigate the effects of construction sequence and C/D on the interaction of perpendicularly crossing tunnels. Two pairs of centrifuge tests and a numerical back-analysis were carried out. Three-dimensional excavation of a new tunnel across an existing tunnel was simulated in-flight. A hypoplasticity constitutive model with small strain stiffness was adopted in the numerical back-analysis. The results are interpreted and discussed.

Centrifuge modeling of crossing-tunnel interaction

Typical setup of a model package

Figure 1a shows the typical plan view of a centrifuge model package. Both existing and new tunnels had an outer diameter (D) equivalent to 6 m in the prototype scale at a centrifugal acceleration of 60 times the earth's gravity (g). The existing tunnel was modeled as "wished-in-place" as the tunnel was empty and both ends were closed to prevent soil from

moving into the tunnel. Reference axes, “X” and “Y”, denote longitudinal and transverse directions of the existing tunnel, respectively. In each test, the new tunnel was excavated either above or beneath the existing tunnel in six excavation stages.

Test program

A total of four tests are reported in this study and the cross-sectional views of the four setups are shown in Figure 1b. All the tests were carried out in a geotechnical centrifuge at the Hong Kong University of Science and Technology (Ng et al., 2001, 2002). In Test E2N3, which is considered as a reference test, the new tunnel was excavated beneath the existing tunnel. The identity of each test denotes the approximate C/Ds of the two tunnels in that test. For the reference test, the C/Ds of the existing and new tunnels were 2 (E2) and 3.5 (N3), respectively. The pillar depth-to-diameter ratio (P/D) was 0.5 in all tests, where the pillar depth is the clear vertical distance between the two tunnels. To study the impact of construction sequence on crossing-tunnel interaction, the new tunnel was constructed above the existing tunnel in Test E3N2. In addition, results from Tests E3N5 and E5N3, in which the existing and new tunnels had larger C/Ds than those in Tests E2N3 and E3N2, were also analyzed to investigate the effects of cover depth on the interaction of perpendicularly crossing tunnels. Note that some of measured results from Test E2N3 have been reported Ng et al. (2013).

Simulation of tunnel excavation in-flight

Tunnel excavation was carried out by simulating the effects of volume and weight losses in-flight using a “Donut” (Ng et al., 2013) as shown in Figure 2a. A “Donut” consists of an outer rubber membrane, an aluminum alloy tube serving as the tunnel lining and an inner rubber bag. During model preparation, the outer rubber membrane and the inner rubber

bag were filled with a heavy fluid (ZnCl_2) having the same density as the soil in each test. The outer rubber membrane simulates the effects of volume loss equivalent to 2%, by regulating the outflow of the heavy fluid inside it. To drain out the heavy fluid from the outer rubber membrane, a valve connected to a drainage tube is controlled in-flight. The inner rubber bag mimics the effects of weight loss inside the tunnel. To simulate the effects of weight loss, the heavy fluid inside the inner rubber bag is also drained away using a valve. The outflow of heavy fluid was collected in a reservoir (see Fig. 1a).

Figure 2b shows a new tunnel and the typical setup of crossing tunnels in each test. Tunnel excavation was carried out in six stages using “Donuts” in each test. The length of excavation in each stage was $0.6D$ or equivalent to 3.6 m in the prototype scale. Each end of the new tunnel was closed to prevent soil movement into the tunnel. In each excavation stage, soil movement towards the tunnel face was not allowed. The lining of each tunnel consisted of an aluminum alloy tube with an outer diameter of 100 mm and thickness of 3 mm. This model tunnel simulated a prototype tunnel that is 6 m in diameter. In order to convert the thickness of the model tunnel lining into the equivalent thickness of a tunnel lining made of concrete, the compressive strength of concrete (f'_c) was assumed to be 50 MPa. Thus, Young’s modulus (E_c) was estimated to be 33 GPa (ACI, 2011). According to Taylor (1995), the flexural stiffness per unit width of tunnel lining and flexural stiffness in the longitudinal direction of the tunnel are scaled by factors of $1/N^3$ and $1/N^4$, respectively, where N is an integer by which gravitational acceleration is to be multiplied. Thus, the equivalent thicknesses of the tunnel lining in the transverse and longitudinal directions of each tunnel are 230 and 420 mm, respectively, in the prototype scale.

Model preparation

All four tests were carried out in dry Toyoura sand. The mean particle size (D_{50}), maximum void ratio ($e_{\max}$), minimum void ratio ($e_{\min}$), specific gravity (G_s) and friction angle at the critical state (ϕ_c) of the sand are 0.17 mm, 0.977, 0.597, 2.64, and 30° , respectively (Ishihara, 1993). According to Garnier et al. (2007), the tunnel diameter and mean soil particle size ratio (D/D_{50}) should be larger than 175 in order to minimize the effects of particle size on tunnel-soil interaction. In this study, this ratio was 588.

To prepare the soil sample in each test, a dry pluviation technique was adopted with a drop height and rate of pluviation of 500 mm and 100 kg/hour, respectively. The density achieved in Tests E2N3, E3N2, E3N5 and E5N3 were 1529 kg/m^3 (relative density $D_r = 64\%$), 1534 kg/m^3 ($D_r = 66\%$), 1529 kg/m^3 ($D_r = 64\%$) and 1541 kg/m^3 ($D_r = 68\%$), respectively.

Instruments on the existing tunnel

Typical instruments installed on the existing tunnel are shown in Figure 3a. Strain gages were attached along the crown and invert in the longitudinal direction of the existing tunnel with typical spacing of 50 mm. The strain gages were connected in a full Wheatstone bridge in order to compensate for changes in temperature.

The transverse cross-sectional view at the center of the existing tunnel is shown in Figure 3b. Eight sets of strain gages were installed at an evenly spacing of 45° . Each set of strain gages, connected in a full Wheatstone bridge, were attached to the outer and inner surfaces of the tunnel lining. A total of four potentiometers were used to measure tunnel deformation. A potentiometer is a resistor which relates a change in electrical resistance to the displacement of a measuring rod (Todd, 1975). The accuracy of a potentiometer was estimated to be equivalent to $\pm 1 \text{ mm}$ in the prototype scale. The potentiometers were fixed to

a plate which was in turn screwed to a frame that was connected to the tunnel lining at two places at distances of 3D and 5D from the center of the existing tunnel (see Fig. 3a).

Figure 3c shows extension rods for linear variable differential transformers (LVDTs) placed along the crown of the existing tunnel to measure vertical displacement. Each extension rod consisted of an inner and an outer aluminum alloy tube. One end of the inner tube was placed on the crown of the existing tunnel while the other end was connected to an LVDT. The outer tube encased the inner one in order to minimize the friction from the surrounding soil.

Test procedure

Each model package was prepared following the procedures discussed above. Afterwards, the centrifuge was gradually spun up to reach an acceleration of 60 times the earth's gravity. Initial readings were taken from all transducers once they had become stable. Then, tunnel excavation using "Donuts" was carried out in six consecutive stages. In each excavation stage, sufficient time was provided for the readings to become stable. The centrifuge was spun down after the completion of tunnel excavation.

Three-dimensional numerical analysis

To back-analyze the results from the centrifuge tests, a three-dimensional finite element software PLAXIS 3D 2013 (Brinkgreve et al., 2013) was adopted.

Finite element mesh and boundary conditions

Figure 4a shows the three-dimensional finite element mesh of a new tunnel excavation above an existing tunnel (Test E3N2). The dimensions of the mesh and the geometry of the tunnels were identical to those in the centrifuge tests. Owing to symmetry, only half of the

entire mesh was required. Thus, a plane of symmetry was defined at the centerline of the new tunnel ($X/D = 0$). Roller and pin supports were applied to the four vertical sides and the base of the mesh, respectively. Soil was modeled using a 10-node tetrahedral element and the tunnel lining using a 6-node plate element.

Some details of crossing tunnels are shown in Figure 4b. At the plane of symmetry ($X/D = 0$), additional constraints were imposed on the tunnel lining. By imposing these constraints, rotation of the lining around the longitudinal and vertical axes of the new tunnel was not allowed (ϕ_y and $\phi_z = 0$), as was translational movement in the transverse direction of the new tunnel ($u_x = 0$).

Constitutive model and model parameters

A hypoplastic constitutive model (Gudehus et al., 2008) was adopted to represent Toyoura sand. Hypoplasticity models have been developed to simulate the response of coarse-grained soils (Kolymbas, 1991; Gudehus, 1996; von Wolffersdorff, 1996; Wu et al., 1996; Gudehus and Mašín, 2009). Like many other advanced soil models, the hypoplasticity constitutive model is able to capture soil behavior related to stress state and history. More importantly, it is capable of modelling strain-dependency and path-dependency of soil stiffness at small strains (Niemunis and Herle, 1997).

Herle and Gudehus (1999) reported some model parameters (ϕ_c , h_s , n , e_{d0} , e_{c0} and e_{i0}) based on their calibration results in Toyoura sand. To determine exponents α , β and small strain stiffness parameters (m_R , m_T , R , β_r and χ), test results obtained from local strain measurement and bender elements (Yamashita et al., 2000, 2009) were numerically fitted using a “soil test” (a utility available in PLAXIS). The model parameters are summarized in Table 1.

The tunnel lining was modeled using a linear elastic material. Young's modulus, density and Poisson's ratio of the tunnel lining were assumed to be 69 GPa, 2700 kg/m³ and 0.33, respectively.

Numerical modeling procedures

Drained effective stress analysis was performed in all numerical runs as the tests were carried out in dry sand. The procedures of numerical back-analysis followed that in the centrifuge tests. In the initial stage, the unit weight of soil and void ratio under the 1g conditions were specified. The linings of the existing and new tunnels were modeled as "wished-in-place" as they were activated in the initial stage. In addition, the soil inside the existing tunnel and some of the soil inside the new tunnel were removed or deactivated (see Fig. 4a). To initialize stress in soil, the at-rest earth pressure coefficient (K_0) of 0.5 (using the empirical equation, $K_0 = 1 - \sin \phi'$) was adopted. Rise in centrifugal acceleration was simulated by increasing gravity by 60 times ($\Sigma M_{\text{weight}} = 60$). Then, excavation of the new tunnel was carried out by modeling the effects of volume and weight losses. For effects of volume loss, which were equivalent to 2%, a "surface contraction" (a utility in PLAXIS) was applied on the surface of the new tunnel. The surface contraction technique simulated a uniform radial contraction of the tunnel lining in each section. At the common nodes or interface of each section, the magnitude of contraction was half of that in the middle of the section. The contraction gradually increased from the interface to the middle part of each section. Another half of the contraction at the interface of the section was simulated in the following excavation stage. By doing so, there is no double contraction simulated during the numerical modelling procedures. To model the effects of weight loss, the soil elements inside the new tunnel were deactivated. The face of the new tunnel was supported using rollers. These rollers were applied simultaneously with soil removal (deactivation of soil elements) at

each stage. This means that end conditions simulated by numerical and centrifuge modeling are consistent. The tunnel excavation in the subsequent five stages followed the procedures described above.

Interpretation of results

Measured and computed results reported in this study are in the prototype scale unless otherwise stated.

Vertical displacement and gradient of the existing tunnel

Figure 5 shows vertical displacement of the existing tunnel at the end of excavation. The vertical displacement is normalized by the diameter of the new tunnel to observe the general trend along with those in the case histories. The positive and negative signs denote heave and settlement of the existing tunnel, respectively. By expecting symmetrical tunnel settlements, the extension rods in Tests E2N3 and E3N5 were only placed on one half of the existing tunnel (i.e., from $X/D = 0$ to 4). In Test E5N3, there was no measurement of vertical displacement of the existing tunnel.

In Test E2N3, where the new tunnel was excavated beneath the existing tunnel, the maximum measured tunnel settlement was $0.3\%D$ (18 mm). On the other hand, heave of the existing tunnel occurred due to the new tunnel construction above in Test E3N2. At offset distances from $1D$ to $6D$ from the centerline of the new tunnel, settlement of the existing tunnel occurred. This is because additional stress induced due to stress redistribution at these distances. It is evident that stress redistribution took place in soil to maintain equilibrium. Detailed explanations of stress re-distribution are given in the section under “Mobilized vertical stress of soil acting on the existing tunnel”. The magnitude of the measured vertical displacement of the existing tunnel caused by the new tunnel excavation beneath (Test E2N3)

was much larger than that when the new tunnel advanced above (Test E3N2). This is because the reduction in vertical stress acting on the existing tunnel was larger in the former test than in the latter. In addition, the stiffness of the soil around the existing tunnel was smaller in Test E2N3 than Test E3N2 (to be discussed in the section under “Mobilized soil stiffness due to tunnel excavation”).

The computed results were consistent with the measured ones in both tests with minor discrepancies due likely to the use of model parameters adopted from the literature and empirical relationships. For Test E3N2, the computed heave of the existing tunnel was higher than the measured one. One possible reason was that an idealized uniform radial contraction was applied on the tunnel lining in the numerical simulation. However, in the centrifuge test, ground movement around the new tunnel may not be uniform.

The behavior of the existing tunnel in Tests E3N5 and E5N3 was similar to that in Tests, E2N3 and E3N2 even though the C/Ds of the existing and new tunnels in the former two tests were 1.5 larger than those in the latter two tests. In the tests where the new tunnel was excavated underneath the existing tunnel, tunnel settlement decreased as the C/Ds of tunnels increased (i.e., settlement in Test E3N5 was smaller than that in Test E2N3). This is because the stiffness of soil around the existing tunnel was larger in the deeper tunnel test than in the shallower tunnel test, even though the effects of stress relief on the existing tunnel were larger in the former case than in the latter. However, when the new tunnel advanced above the existing tunnel, heave of the existing tunnel increased with increasing C/D (i.e., heave in Test E5N3 was larger than that in Test E3N2). In these two tests, the effects of stress relief, which increased with depth, dominated the rise in the stiffness of soil around the existing tunnel. Further explanations are given later.

Allowable tunnel displacements less than 15 mm and 20 mm were recommended by LTA (2000) and BD (2009), respectively. Settlement of the existing tunnel measured in Tests

E2N3 and E3N5 exceeded the limit suggested by LTA (2000) while heave of the existing tunnel in Tests E3N2 and E5N3 was still within the allowable limits.

Table 2 summarizes the maximum induced tunnel gradient in each test. Similar to the tunnel vertical displacement, the gradient of the existing tunnel induced by the new tunnel excavation beneath (Tests E2N3 and E3N5) was significantly larger than that due to the new tunnel advancement above (Tests E3N2 and E5N3). The induced tunnel gradient in each test did not exceed the recommended limit of 1:1000 (LTA, 2000; BD, 2009).

Induced strain and shear stress in the longitudinal direction of the existing tunnel

Induced strain along the invert in the longitudinal direction of the existing tunnel at the end of tunnel excavation is shown in Figure 6. The positive and negative signs denote induced tensile and compressive strain at the invert in the tunnel lining, respectively.

In Test E2N3, the maximum measured settlement of the existing tunnel occurred at the location directly above the new tunnel (refer to Figure 5). At this location, measured tensile strain was induced in the tunnel lining at the invert, i.e., sagging zone as shown in Figure 6. At a distance more than $2.5D$ away from the centerline of the new tunnel, compressive strain and tensile strain was induced at the invert and at the crown, respectively. This location is considered as a hogging zone. In Test E3N2, however, compressive strain was induced at the center of the existing tunnel while the induced tensile strain was observed at some distance away. The magnitude of maximum induced strain in Test E2N3 was much larger than that in Test E3N2. This induced strain in the two tests was consistent with the tunnel vertical displacement (refer to Fig. 5). The computed results agreed reasonably well with the measured ones. The possible reason for the minor discrepancies is as mentioned above.

The direction and shape of the induced strain in the tunnel lining in Tests E3N5 and E5N3 were similar to those in Tests E2N3 and E3N2, respectively, even though the C/Ds of

the existing and new tunnels were larger in the latter two tests than in the former two. The computed maximum induced tensile strain and compressive strain in Test E3N5 were slightly larger than those in Test E2N3. Although settlement of the existing tunnel was larger at a shallower depth (i.e., in Test E2N3) than at a greater depth (i.e., in Test E3N5) (see Fig. 5), the induced strain was nevertheless slightly larger in the latter case. Similarly, for the new tunnel excavation above the existing tunnel, the induced maximum compressive strain at a greater depth (i.e., in Test E5N3) was larger than that at a shallower depth (i.e., in Test E3N2). This is because of the greater vertical stress reduction of soil around the existing tunnel in Tests E3N5 and E5N3 (with the larger C/Ds) than in Tests E2N3 and E3N2 (with the smaller C/Ds). Further details are given later.

According to ACI (2001), the cracking tensile strain of unreinforced concrete is $150 \mu\epsilon$. The maximum tensile strain induced in the existing tunnel by the construction of a new tunnel underneath (Tests E2N3 and E3N5) exceeded the cracking tensile strain.

Table 3 gives the maximum shear stress induced on the tunnel lining in each test. The values were deduced from the slope of the strain induced in the longitudinal direction of the existing tunnel using beam theory. The allowable shear stress of concrete is estimated to be 660 kPa if the concrete compressive strength (f'_c) is 50 MPa and a reduction factor of 0.55 is adopted (ACI, 2011). The maximum shear stress induced on the existing tunnel by the new tunnel advancement beneath (Tests E2N3 and E3N5) exceeded the allowable shear stress. Judging from the maximum induced tensile strain and the maximum induced shear stress observed, cracks may appear in the lining of the existing tunnel due to the new tunnel construction beneath.

Mobilized soil stiffness due to tunnel excavation

Figure 7 shows the mobilized secant shear modulus of soil along the crown and invert of the existing tunnel before and after the new tunnel excavation. The mobilized secant shear modulus (G_m) was obtained from the computed deviatoric stress (q) and the computed deviatoric strain (ϵ_s) according to the equation $G_m = q/3\epsilon_s$.

The mobilized shear modulus of soil along the crown of the existing tunnel is shown in Figure 7a. In Test E2N3, G_m was about 11 MPa before the tunnel excavation. After the tunnel excavation, G_m only changed slightly as the influence of the new tunnel excavation on soil stiffness was minimized by the presence of the existing tunnel. The increase in G_m after tunnel excavation within a distance of 1D from the centerline of the new tunnel is mainly because of increased confining stress resulting from stress redistribution in soil. This increase of stress at the crown is discussed later in the section “Incremental normal stress acting on the existing tunnel”.

G_m before tunnel excavation was larger in Test E3N2 than in Test E2N3 as the G_m increased with confining pressure (which in turn increased with depth). However, after the construction of the new tunnel above (Test E3N2), G_m reduced sharply at the center of the existing tunnel ($X/D = 0$). As expected, G_m before tunnel excavation was larger in Test E3N5 (with the larger C/D s) than in Test E2N3 (with the smaller C/D s). After the excavation of the new tunnel underneath, there was a slight reduction in G_m along the crown. In Test E5N3, G_m before and after tunnel excavation were larger than those in Test E3N2.

In Figure 7b the mobilized shear modulus of soil along the invert of the existing tunnel is illustrated. To compare the vertical displacement of the existing tunnel across the tests, the mobilized shear modulus of soil on the side of the existing tunnel facing the new tunnel is considered. It can be seen that before tunnel excavation, G_m at the crown was larger than that at the invert of the existing tunnel in every test. This is because of the presence of the existing

tunnel, causing reduction in G_m at the invert of the existing tunnel. At any location further away from the existing tunnel, G_m increased with depth.

G_m both before and after tunnel excavation along the crown in Test E3N2 (refer to Fig. 7a) were larger than those along the invert in Test E2N3 (see Fig. 7b). As a result, the magnitude of tunnel heave in Test E3N2 was significantly smaller than the magnitude of tunnel settlement in Test E2N3 (see Fig. 5). G_m along the invert of the existing tunnel in Test E3N5 (with the larger C/Ds) was larger than that in Test E2N3 (with the smaller C/Ds). This resulted in a smaller settlement of the existing tunnel in Test E3N5 than in Test E2N3. However, heave of the existing tunnel in Test E5N3 was larger than that in Test E3N2 as stress relief was a dominating factor (to be discussed later).

Deformation of the existing tunnel

Figure 8 shows the deformation of the existing tunnel in terms of the change in the normalized tunnel radius ($\Delta R/R_0$, where ΔR is the change in the tunnel radius and R_0 is the undeformed tunnel radius, which is 3m). Note that the change in tunnel radius is a relative displacement between the tunnel lining and the centroid of the existing tunnel. The deformations at the center of the existing tunnel ($X/D = 0$) when the excavated section of the new tunnel reached the centerline of the existing tunnel ($Y/D = 0.3$) and at the end of tunnel excavation ($Y/D = 1.5$) are reported. The positive and negative signs denote an increase and a decrease in the normalized tunnel radius, respectively.

Figure 8a shows a decrease in radius at the left springline and invert but an increase in radius at the crown and right springline of the existing tunnel as measured in Test E2N3. The combined results of changes in radius at the crown, springlines and invert revealed that the existing tunnel was elongated horizontally. This is because the reduction in stress caused by the new tunnel excavation, not only resulted in a reduction in vertical stress but it also caused

a significant stress relief at each springline of the existing tunnel. By comparing the locations of stress relief on the existing tunnel, it is found that stress reduction in the horizontal direction was larger than that in the vertical direction. This means that the existing tunnel elongated horizontally. Further explanation of stress changes is given in the section under “Incremental normal stress acting on the existing tunnel”. In Test E3N2, however, the tunnel radius at the crown and left springline increased while the tunnel radius at the right springline and invert decreased according to measurements taken. The combined result was elongation in the vertical direction and compression in the horizontal direction. This is because the reduction in stress in the horizontal direction was larger than that in the vertical direction in Test E2N3, while stress decrease was larger in the vertical direction than in the horizontal direction in Test E3N2. Further explanation is given later. The computed results were in an agreement with the measured ones with only minor discrepancy, the reason for which has been mentioned in the section “Vertical displacement and gradient of the existing tunnel”. Another possible reason for the discrepancy between the measured and computed results is that numerical simulation of volume loss may not represent a perfect match to that in the centrifuge tests. An idealized uniform volume contraction was imposed in the numerical simulation. However, this might not be achieved in centrifuge model tests.

The existing tunnel experienced vertical compression in Test E3N5 as it did in Test E2N3, but vertical elongation in Test E5N3 as it did in E3N2. This shows that an increase in the C/Ds of the existing and new tunnels does not affect how the existing tunnel would deform.

Tunnel deformation at the end of the new tunnel excavation is shown in Figure 8b. As measured in Test E2N3, the reduction in tunnel diameter in the vertical direction and the increase in tunnel diameter in the horizontal direction were larger than those when $Y/D = 0.3$ (refer to Fig. 8a). This suggests that the vertical compression of the existing tunnel at the end

of tunnel excavation was larger than that when the excavated section of the new tunnel was directly underneath the existing tunnel. Similarly, the elongation of the existing tunnel in the vertical direction in Test E3N2 was larger at the completion of the new tunnel than when the excavated section of the new tunnel was directly above the existing tunnel ($Y/D = 0.3$).

According to BTS (2000), the maximum difference between the maximum and minimum diameters of the tunnel should be within 2% [i.e., $(D_{\max} - D_{\min})/D_0 \leq 2\%$, where $D_{\max}$, $D_{\min}$ and D_0 are the maximum, minimum and undeformed tunnel diameters, respectively]. The deformation of the existing tunnel induced by the new tunnel excavation did not exceed this allowable limit in any of the tests. However, given that the at-rest earth pressure coefficient (K_0) was estimated to be 0.5, the existing tunnel was vertically compressed due to overburden pressure even before the new tunnel excavation. The construction of the new tunnel beneath only served to compress the existing tunnel even more in Tests E2N3 and E3N5.

Although the shape of model tunnel deformed into an oval shape as the centripetal acceleration increased to 60g, the deviation ($\Delta D/D_0$) of it from a circular shape (like the one in the field) is less than 0.6%. A separate numerical simulation using a circular model tunnel (i.e., not oval shape at 60g) was carried out. It is found that the general trends of induced strain in the tunnel lining in the transverse direction and incremental normal stress acting on the existing tunnel are very similar for both oval and circular tunnel cases.

Induced strain in the transverse direction of the existing tunnel

Induced strain measured at the outer surface at the center of the existing tunnel ($X/D = 0$) in the transverse direction is shown in Figure 9. The measurements were taken when the excavated section reached the existing tunnel ($Y/D = 0.3$) and at the end of the new tunnel

excavation ($Y/D = 1.5$). The positive and negative signs denote induced tensile and induced compressive strain, respectively.

Figure 9a shows that in Test E2N3, compressive strain was induced at the left shoulder, right knee and invert while the crown, right shoulder, springlines and left knee experienced tensile strain. In Test E3N2, tensile strain was induced at the crown, right shoulder, knees and invert while the left shoulder and springlines experienced compressive strain. In Test E3N5, compressive strain was induced at the crown, knees and invert while tensile strain was observed at the shoulders and springlines. In Test E5N3, tensile strain was observed at the crown and invert, but compressive strain occurred at the shoulders, springlines and knees. The induced strains in all four tests were consistent with the deformation of the existing tunnel (refer to Fig. 8a).

The induced strain at the end of the new tunnel excavation is shown in Figure 9b. In the tests where the new tunnel was excavated beneath the existing tunnel (Tests E2N3 and E3N5), a larger induced tensile strain at the springlines and a larger compressive strain at the crown and invert than when $Y/D = 0.3$ (see Fig. 9a) were observed. In Tests E3N2 and E5N3, induced compressive strain at the springlines of the existing tunnel was larger at the end of tunnel excavation than when the new tunnel was directly above the existing tunnel ($Y/D = 0.3$). The induced strains confirm that vertical compression in Test E2N3 was larger at the end of tunnel excavation than when the excavated section of the new tunnel reached the existing tunnel (refer to Figs. 8a and 8b). Similarly, the vertical elongation of the existing tunnel in Test E3N2 was larger at the completion of the new tunnel than at $Y/D = 0.3$.

The induced tensile strain in the transverse direction of the existing tunnel did not exceed the cracking tensile strain of unreinforced concrete of $150 \mu\epsilon$ (ACI, 2001). However, in Tests E2N3 and E3N5, tensile strain was present at both springlines due to overburden pressure of soil even before tunnel excavation, as K_0 was estimated to be 0.5. Thus, the

tensile strain induced by the new tunnel excavation may still cause cracks to appear in the lining of the existing tunnel.

Ye et al. (2014) carried out model testing by applying a point load on tunnel linings with and without radial (transverse) joints. They reported that the tunnel linings with joints deformed larger but induced smaller transverse bending moment than that without joints. By adopting their finding, it is possible that the deformation of the existing tunnel in this study (without joints) may be underestimated that in the field. On the other hand, induced strains in the transverse direction of the existing tunnel in this study should be larger than that of jointed tunnel linings and hence more conservative.

Incremental normal stress acting on the existing tunnel

Figure 10 shows the incremental normal stress at the center of the existing tunnel ($X/D = 0$). The positive and negative signs denote an increase and a decrease in normal stress, respectively, compared with when the centrifugal acceleration reached 60g.

The incremental normal stress when the excavated section reached the centerline of the existing tunnel ($Y/D = 0.3$) is shown in Figure 10a. In Test E2N3, a significant reduction in normal stress occurred at the left knee and invert as the excavated section of the new tunnel approached the existing tunnel. A decrease in stress was also observed at the springlines and right knee while an increase in stress took place at the crown and shoulders. The stress increase was due to a redistribution of stress to maintain equilibrium. In Test E3N2, normal stress reduced at the crown, shoulders, springlines and left knee but increased slightly at the invert of the existing tunnel. Even with the larger C/D s of the existing and new tunnels in Tests E3N5 and E5N3, normal stress exhibited trends similar to those in Tests E3N5 and E5N3, respectively, differing only in magnitude. The reduction in normal stress at the springlines, knees and invert in Test E3N5 was larger than that in Test E2N3. Similarly, large

stress reductions at the crown, left shoulder, left springline and left knee in Tests E5N3 than E3N2 were observed. Further explanations are given later.

The results at the end of tunnel excavation ($Y/D = 1.5$) are shown in Figure 10b. Comparing these results with those obtained when $Y/D = 0.3$, there was a significant reduction in normal stress at the right knee in Tests E2N3 and E3N5. Similarly, the decrease in normal stress in Tests E3N2 and E5N3 at the right shoulder at the end of tunnel excavation was much larger than that when $Y/D = 0.3$.

For the new tunnel excavation beneath the existing tunnel, the reduction in normal stress at the invert and knees of the existing tunnel increased with increasing C/D s of the existing and new tunnels. Stress reduction at the invert in Test E3N5 was greater than that in Test E2N3. By using the beam theory, computed tensile strain in the lining at the invert of the existing tunnel in the former was larger than that in the latter case (see Fig. 6). Similarly, computed compressive strain at the crown in the former was also greater than that in the latter case.

For the case when the new tunnel was excavated underneath the existing tunnel, the reduction in vertical stress at the invert was greater with an increasing C/D of existing tunnel from 2 to 3.5. The reduction in vertical stress in Tests E2N3 and E3N5 were 147 kPa and 195 kPa, respectively. As a result of stress reduction, the decrease in stiffness was larger with an increasing C/D of tunnel (refer to Fig. 7b). For Tests E2N3 and E3N5, a decrease in mobilized shear modulus due to tunneling was 7.0 MPa and 9.4 MPa, respectively. The effects of reduction in stress and stiffness in Test E2N3 were greater than that in Test E3N5, resulting in larger tunnel settlement in the former than the latter test (see Fig. 5).

The normal stress reduction at the crown and shoulders of the existing tunnel when the new tunnel was constructed above was larger at a greater depth (E5N3) than at a shallower

518 depth (E3N2). As a result, larger induced strain in the longitudinal direction of the existing
519 tunnel was observed in Test E5N3 than in Test E3N2 (refer to Fig 6).

520 In the tests where the new tunnel was constructed above the existing tunnel, the
521 decreases in vertical stress at the crown of the existing tunnel in Tests E3N2 and E5N3 were
522 161 kPa and 202 kPa, respectively. The decreases in stress resulted in stiffness reduction at
523 the crown in Tests E3N2 and E5N3 by 13.0 MPa and 16.1 MPa, respectively (see Fig. 7a).
524 The effects of decrease in stress and stiffness in Test E3N2 was less than that in Test E5N3,
525 resulting in smaller tunnel heave in the former test than the latter (refer to Fig. 5).

526 The difference between incremental stress in the horizontal and vertical directions is
527 one of the factors influencing the deformation and induced strain in the transverse direction
528 of the existing tunnel. At the end of tunnel excavation in Tests E2N3 and E3N5, the reduction
529 in stress in the horizontal direction was larger than that in the vertical direction. As a result,
530 the existing tunnel was compressed vertically in these two tests (refer to Figs. 8b and 9b). On
531 the other hand, the decrease in stress in the vertical direction was larger than that in the
532 horizontal direction in Tests E3N2 and E5N3, resulting in the vertical elongation of the
533 existing tunnel.

534 BD (2009) suggested that the change in stress acting on a tunnel lining should be within
535 20 kPa. In Tests E2N3 and E3N5, the reduction in normal stress at the springlines, knees and
536 invert as well as the increase in normal stress at the crown all exceeded the recommended
537 limit. For the tests where the new tunnel advanced above the existing tunnel (E3N2 and
538 E5N3), the decrease in normal stress at the crown, shoulders, springlines and knees all
539 exceeded the allowable stress. This suggests that a structural analysis of the existing tunnel
540 considering changes in stress should be undertaken. In addition, as the maximum change in
541 normal stress (e.g., stress reduction at the invert in Tests E2N3 and E3N5 as well as at the
542 crown in Tests E3N2 and E5N3) occurred during the advancement of the new tunnel but not

at the end of tunnel excavation, analysis of crossing-tunnel interaction should be carried out three-dimensionally.

Mobilized vertical stress of soil acting on the existing tunnel

The ratio of vertical stress after tunnel excavation ($\sigma'_{v,after}$) to that before construction of the new tunnel (σ'_{v0}) is referred to as the mobilized vertical stress ratio and is shown in Figure 11. The ratio is considered for soil on the side of the existing tunnel facing the new tunnel (along the invert in Tests E2N3 and E3N5 and along the crown in Tests E3N2 and E5N3).

The minimum mobilized vertical stress ratio of 0.33 was found in Test E2N3 at the center of the existing tunnel ($X/D = 0$). This ratio increased with distance from the center of the existing tunnel. In Test E3N2, the reduction in the mobilized vertical stress ratio was smaller than that in Test E2N3 within the distance of 1D from the center of the existing tunnel ($X/D \leq 1$). This smaller reduction in the mobilized vertical stress ratio and larger soil stiffness (refer to Fig. 7) resulted in significantly smaller vertical displacement and strain induced in the lining of the existing tunnel in Test E3N2 than in Test E2N3 (see Figs. 5 and 6).

Despite the larger C/Ds of the existing and new tunnels in Test E3N5, the mobilized vertical stress ratio in this test behaved in a similar manner to that in E2N3. The minimum vertical stress ratio in both tests was about 0.35. Similarly, the mobilized vertical stress ratio in Test E5N3 was almost identical to that in Test E3N2 with a minimum stress ratio of about 0.45. Thus, the reduction in stress acting on the existing tunnel in the tests with the larger C/Ds was greater than that in the tests with the smaller C/Ds (refer to Fig. 10b) as the confining pressure increased with depth. This suggests that the mobilized vertical stress ratio

is influenced by the effects of construction sequence but is independent of the effects of cover depth on the crossing-tunnel interaction.

Summary and conclusions

Two pairs of centrifuge tests were carried out to examine crossing-tunnel interaction. The effects of construction sequence on crossing-tunnel interaction were investigated in the first pair of tests, where a new tunnel was excavated beneath an existing tunnel in a reference test, while the new tunnel advanced above in the other test. The second pair of tests were conducted with larger cover depth-to-tunnel diameter ratios (C/D_s) than in the first pair to study the effects of cover depth on the construction sequence in crossing tunnels. In order to improve the understanding of the interaction of crossing tunnels, a hypoplasticity constitutive model with small strain stiffness was adopted to back-analyze the tests. It should be pointed out that like any numerical simulation, centrifuge modelling is very useful for comparative study among tests with almost identical geometrical and ground conditions. The direct use of any numerical simulation and centrifuge test results to any design, however, should be cautious. Based on the interpretation of measured and computed results, the following conclusions may be drawn:

- (a) The settlement of the existing tunnel when the new tunnel was excavated beneath was significantly larger than the heave of the existing tunnel when the new tunnel was constructed above. This is because the reduction in vertical stress acting on the existing tunnel was larger in the former case than in the latter. In addition, the mobilized shear stiffness of soil around the existing tunnel was smaller in the former tests than in the latter, regardless of the C/D_s of the crossing tunnels.
- (b) When the new tunnel was excavated beneath, the existing tunnel was compressed vertically. However, the existing tunnel was elongated in the vertical direction due to the

new tunnel construction above. This is because the reduction in stress acting on the existing tunnel in the horizontal direction was larger than that in the vertical direction when the new tunnel was excavated underneath. On the other hand, the decrease in stress acting on the existing tunnel in the vertical direction was larger than that in the horizontal direction due to the new tunnel advancement above, irrespective of the C/Ds of tunnels.

(c) The differential vertical displacement of the existing tunnel induced by the new tunnel excavated underneath was larger than when the new tunnel was constructed above it. As a result, induced tensile strain at the invert and shear stress in the lining of the exiting tunnel in the former was larger than that in the latter tunnel excavation sequence. By using the beam theory, computed compressive strain at the crown in the former was also greater than that in the latter. The tensile strain and shear stress in the existing tunnel induced by the new tunnel construction beneath exceeded the cracking tensile strain (ACI, 2001) and allowable shear stress (ACI, 2011), respectively.

(d) Settlement of the existing tunnel due to the new tunnel excavation underneath in the test with the larger C/Ds was less than that in the test with the smaller C/Ds. This is because the mobilized shear stiffness of soil around the existing tunnel increased with depth, even though the reduction in vertical stress acting on the existing tunnel in the test with the larger C/Ds was greater than that in the test with the smaller C/Ds. On the other hand, heave of the existing tunnel caused by the new tunnel construction above increased with increasing C/Ds of the existing and new tunnels due to a larger relief of stress acting on the existing tunnel at a greater depth than at a shallower depth. In addition, the increase in stress relief dominated the increase in the mobilized shear stiffness of soil.

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Table 1 Summary of material parameters adopted in finite element analyses	
Critical state friction angle ^(a) , ϕ_c	30°
Granulates hardness ^(a) , h_s	2.6 GPa
Exponent $n^{(a)}$, n	0.27
Minimum void ratio at zero pressure ^(a) , e_{do}	0.61
Critical void ratio at zero pressure ^(a) , e_{co}	0.98
Maximum void ratio at zero pressure ^(a) , e_{io}	1.10
Exponent $\alpha^{(b)}$, α	0.5
Exponent $\beta^{(b)}$, β	3
Parameter controlling the initial shear modulus upon 180° strain path reversal and in the initial loading ^(b) , m_R	8
Parameter controlling the initial shear modulus upon 90° strain path reversal ^(b) , m_T	4
The size of the elastic range ^(b) , R	0.00003
Parameter controlling the rate of degradation of stiffness with strain ^(b) , β_r	0.2
Parameter controlling the rate of degradation of stiffness with strain ^(b) , χ	1.0
The coefficient of at-rest earth pressure, K_0	0.5
Note: (a) Adopted from Herle and Gudehus, 1999 (b) Calibrated based on previous studies (Yamashita et al., 2000, 2009)	

Table 2 Summary of maximum induced tunnel gradient

Test ID	Induced tunnel gradient	
	Measured	Computed
E2N3	1:1600	1:1850
E3N2	1:4300	1:3400
E3N5	1:1000	1:1880
E5N3	N/A	1:2800

Note: Allowable induced gradient is 1:1000
(LTA, 2000; BD, 2009)

Table 3 Summary of maximum shear stress induced on tunnel lining

Test ID	Induced shear stress (kPa)	
	Measured	Computed
E2N3	780	750
E3N2	200	500
E3N5	N/A	880
E5N3	N/A	510

Note: Allowable shear stress is 660 kPa (ACI, 2011)

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(b)

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