Influences of Shielding of Multi-crossing Tunnels on Ground Displacement

Thayanan Boonyarak

Chief of Engineering Division, Seafco Public Company Limited, 144 Prayasuren Road, Bangchan, Klongsamwah, Bangkok, Thailand. (Former PhD candidate at the Hong Kong University of Science and Technology)

Charles W. W. Ng

Chair Professor, Department of Civil and Environmental Engineering, The Hong Kong University of Science and Technology, Clear Water Bay, Kowloon, Hong Kong.

David Mašín

Associate Professor, Faculty of Science, Charles University in Prague, Albertov 6, 128 43 Prague 2, Czech Republic.

ABSTRACT: Tunnel excavation could damage adjacent structures, both above the ground surface and embedded underground, due to the associated ground movements and stress changes. To estimate ground movement caused by tunneling, simplified methods using Gaussian curve or two-dimensional numerical analysis are commonly used. However, when there is a presence of existing underground structures such as tunnels, the simplified methods may not be applicable to predict the ground displacement. A series of three-dimensional centrifuge tests in dry sand and numerical back-analyses were carried out to improve the fundamental understanding of ground movement caused by multi-tunnel interaction. The major objective of this paper is to study the shielding effects on the interaction of multi-crossing tunnels. The diameter of each tunnel is equivalent to 6 m in the prototype scale. The existing tunnel was instrumented and modeled as "wished-in-place". The new tunnel was excavated three-dimensionally in-flight using a device called "Donut", which simulated the effects of volume loss and weight loss. To further analyze the ground displacement and stress distribution in the ground, three-dimensional finite element analyses using an advanced hypoplasticity constitutive model with small strain stiffness were conducted. Unlike any conventional elastic-perfectly plastic constitutive model, the hypoplasticity model can simulate soil stiffness dependency on state, strain and stress path. The impact of the new tunnel excavation on the existing tunnels and ground displacement is reported. Interpretation of measured and back-analyzed data of crossing multi-tunnel interaction is provided. Influences of shielding on movement of soil, stiffness mobilization and stress redistribution are discussed and explained.

1. INTRODUCTION

The effects of ground movement and change of stress caused by tunnel excavation are important in urban areas as more and more closed-space tunnels have been constructed. When a new tunnel is excavated adjacent to existing tunnels, the existing tunnels may be deformed excessively and cracks may be induced on tunnel linings as reported by Cooper et al. (2002).

To evaluate the effect of ground settlement caused by tunnel excavation, a Gaussian distribution curve is commonly adopted (Peck 1969, Mair & Taylor 1997). However, when new tunnel is constructed beneath existing underground structures such as tunnels or pipelines, a simplified method may not be able to estimate ground movement. The presence of the existing tunnel on ground movement and stress change in soil is defined as "shielding". In addition, the impact of new tunnel construction on ground movement can be very complex if there is more than one existing tunnel.

Ng et al. (2013) studied the interaction of perpendicularly crossing-tunnel interaction using centrifuge tests and numerical back-analysis. In their study, the effects of volume and weight losses on tunnel-

tunnel interaction were investigated separately. Different from their study, this paper evaluate the effects of shielding provided by existing tunnels on ground movement and stress change due to excavation of a new tunnel. Interpretation of multi-crossing tunnel interaction from two centrifuge tests is reported. The tunnel excavation was carried out three-dimensionally in-flight by adopting a device that simulates the effect of volume and weight losses (Ng et al. 2013). To improve further understanding of stiffness mobilization and stress redistribution in each centrifuge test, three-dimensional numerical back-analysis was conducted using an advanced hypoplasticity model with small strain stiffness considered.

2. THREE-DIMENSIONAL CENTRIFUGE MODELING

2.1 TYPICAL MODEL SETUP

A plan view of a centrifuge model package to investigate the effects of shielding on multi-crossing tunnel interaction is shown in Figure 1a. The outer diameter (D) of each tunnel is 100 mm, which is equivalent to 6 m in prototype scale (at 60g). The excavation of new tunnel was simulated three-dimensionally in six stages. The tests were conducted in dry Toyoura sand at the geotechnical centrifuge located at the Hong Kong University of Science and Technology.

2.2 EXCAVATION OF TUNNEL IN-FLIGHT

To simulate tunnel excavation in-flight, a novel device "Donut" was adopted as shown in Figure 1b (Ng et al. 2013). There are three major components in the "Donut". An outer rubber membrane simulates effects of volume loss, equivalent to 2%. Tunnel lining is made of aluminum alloy equivalent to concrete thickness of 420 mm in prototype scale in the longitudinal direction of each tunnel. An inner rubber bag models effects of weight loss (i.e., soil removal inside the new tunnel).

During model preparation, heavy fluid ($ZnCl_2$) is filled into each outer rubber membrane and inner rubber bag. The heavy fluid has the same density as the sand in each test (1532-1535 kg/m^3, equivalent to relative density of 65-66%). To carry out tunnel excavation in-flight, heavy fluid inside each "Donut" is drained away using a valve from stage one to six (refer to Figure 1a).

2.3 TEST PROGRAM

Total two centrifuge tests are reported in this paper. Figure 1c shows elevation view of Test E2N5. The cover depth-to-diameter ratios (C/Ds) of the existing and new tunnels were 2 and 5, respectively. The pillar depth-to-diameter ratio (P/D) was 2, where pillar depth is the vertical clear distance between tunnels. The cover depth of the existing tunnel, cover depth of the new tunnel and pillar depth were equivalent to 12 m, 30 m and 12 m, respectively.

Different from Test E2N5, two existing tunnels were located above the new one in Test E2,3N5 as shown in Figure 1d. The shielding effects of the lower existing tunnel on the upper existing tunnel were studied by comparing the results in this test with those from Test E2N5.

2.4 INSTRUMENTATION

Several types of instruments were adopted to monitor the response of ground and existing tunnels due to the excavation of the new tunnel.

Figures 1c and 1d show extension rods placed along the ground surface and the crown of the existing tunnel to measure settlement, using linear variable differential transformers (LVDTs). For brevity of the paper, results from other types of instrument which are strain gages in the longitudinal and transverse directions of the existing tunnel and potentiometers are not reported.

2.5 MODEL PREPARATION AND TEST PROCEDURES

Dry pluviation technique or sand raining was adopted to prepare Toyoura sand in each test. A drop height of 500 mm and a pouring rate of about 100 kg per hour were used to control the density of sand. When the pluviated sand reached the desired height, each tunnel and instrument were placed on sand.

After the completion of model preparation, the centrifuge was gradually spun up to centripetal acceleration of 60g. The initial readings of each transducer were taken at this stage. Subsequently, tunnel excavation using "Donut" (refer to Figure 1b) was carried out in-flight from stage one to six. Finally, the centrifuge was spun down.

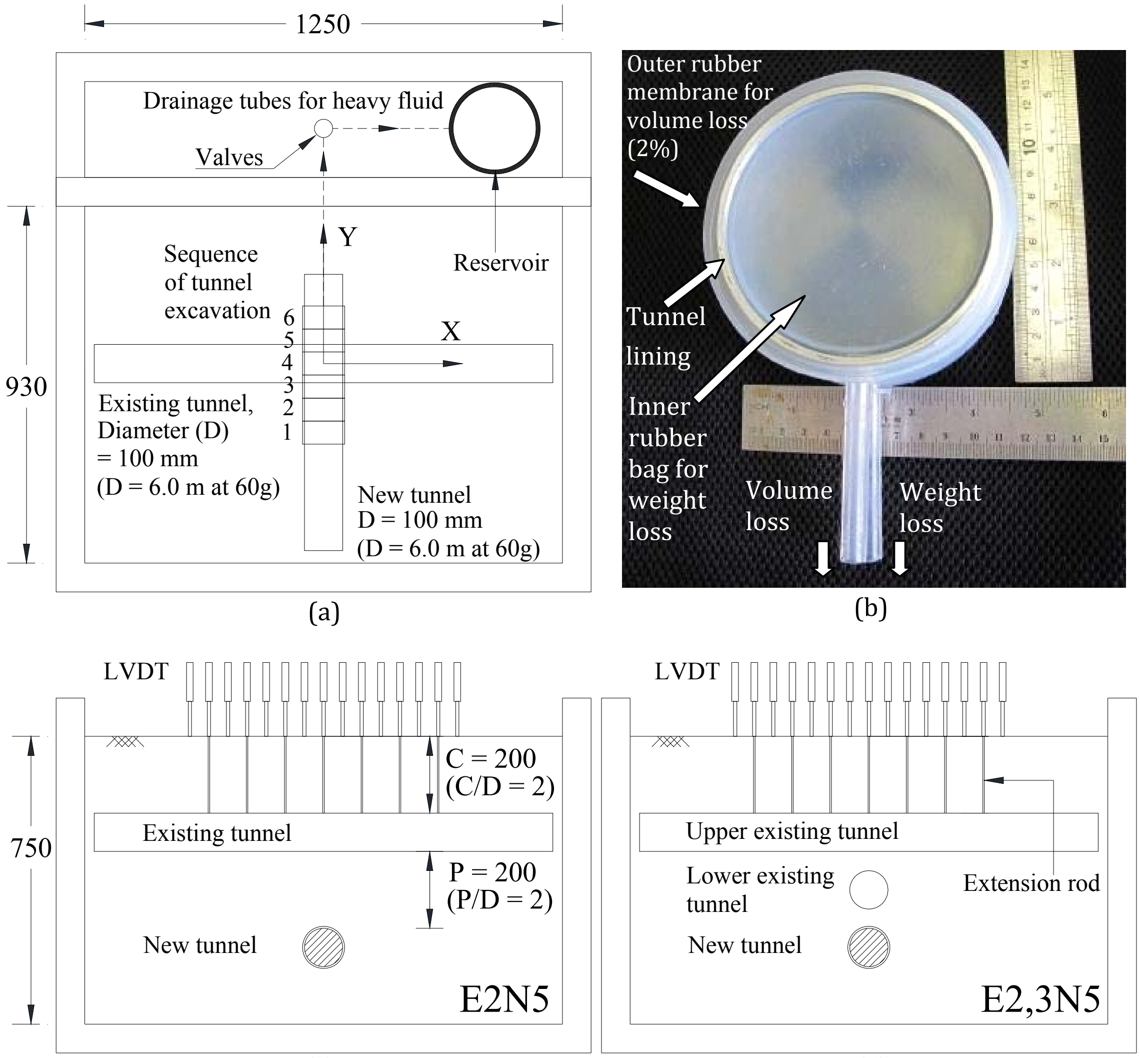

Figure 1: Centrifuge model setup: (a) Plan; (b) "Donut" for simulating tunnel excavation (modified from Ng et al. 2013); (c) Test E2N5; (d) Test E2,3N5

3. THREE-DIMENSIONAL NUMERICAL BACK-ANALYSIS

To improve the understanding of stiffness mobilization and stress transfer in the centrifuge test, finite element back-analysis was carried out three-dimensionally using PLAXIS 3D. In addition to back-analysis, a parametric study numerical run "N5" was carried out to investigate ground movement due to single tunnel excavation without the presence of any existing tunnel.

3.1 MESH AND BOUNDARY CONDITIONS

Figure 2a shows the finite element mesh of Test E2,3N5. The dimension of the mesh and geometry of tunnels were identical to that in the centrifuge test. To reduce computation time, only half of the model was required for the analysis due to symmetry. The soil and tunnel lining was simulated using a 10-node tetrahedral element and a six-node elastic plate element, respectively.

The existing tunnels and the lining of the new tunnel were modeled as "wished-in-place" as the lining of both tunnels was activated and some soil elements inside the tunnel were deactivated in the initial stage. At the plane of symmetry (X/D = 0), translational movement in the "X" direction and rotation around the "Y" and "Z" axes of the tunnel lining was constrained.

3.2 CONSTITUTIVE MODEL AND MATERIAL PARAMETERS

Toyoura sand was modeled using an advance hypoplasticity with small strain stiffness (von Wolffersdorff 1996, Niemunis & Herle 1997). A critical state friction angle (ϕ_c) and parameters controlling void ratios (h_s, n, e_{d0}, e_{c0} and e_{i0}) were adopted according to Herle & Gudehus (1999). Exponents α and β and small strain stiffness parameters (m_R, m_T, R, β_r and χ) were calibrated numerically by curve fitting the triaxial test results with local strain measurement and bender element reported by Yamashita et al. (2000 & 2009). The material parameters are summarized in Table 1.

Linear elastic material was used to model the tunnel lining having a Young's modulus, density and Poisson's ratio of 69 GPa, 2700 kg/m^3 and 0.33, respectively.

3.3 PROCEDURES OF NUMERICAL MODELING

The procedures of numerical back-analysis followed those in the centrifuge test. Initial stress condition was established in 1g with $K_0 = 0.5$. To simulate increase in centripetal acceleration, gravity was raised by 60 times using $\Sigma M_{weight} = 60$. Subsequently, tunnel excavation in each stage was carried out from stage one to six. To model tunnel excavation in each stage, the effects of volume loss (equaling 2%) were simulated by using a "surface contraction" (refer to Figure 2b). This surface contraction is defined as strain normal to the lining of the new tunnel. The effects of weight loss (i.e., soil removal inside the new tunnel) were modeled simultaneously with the effects of volume loss by deactivating the relevant soil elements.

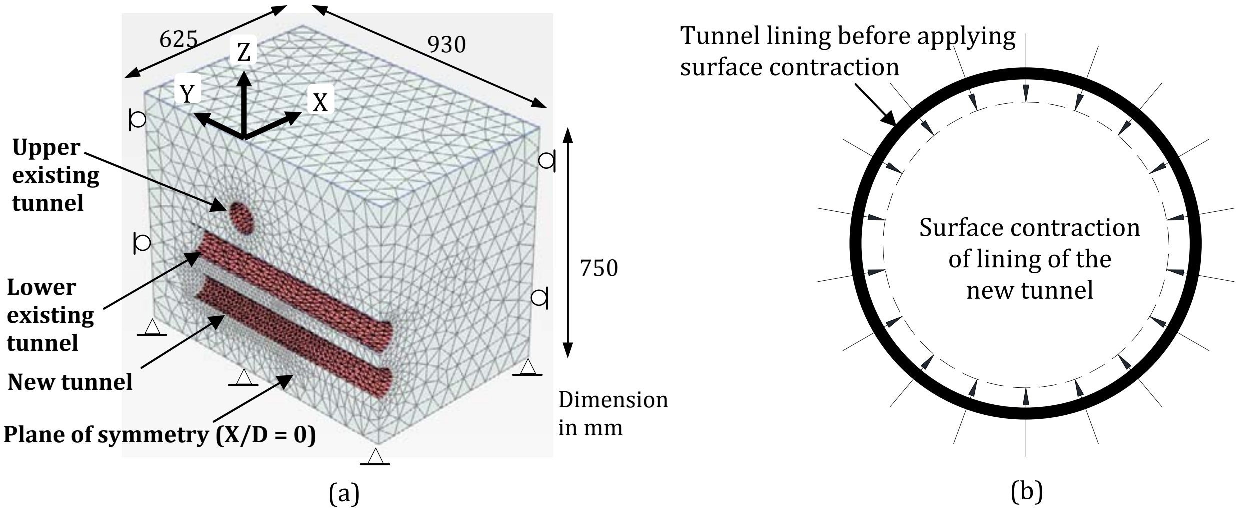

Figure 2: (a) Finite element mesh of Test E2,3N5; (b) diagram of surface contraction adopted for simulating of the effects of volume loss

Table 1: Summary of material parameters adopted in finite element analyses

Basic model parameters (Controlling large strain behavior)		Small strain stiffness parameters (Controlling small strain behavior)	
ϕ_c	30°	m_R	8
h_s	2.60 GPa	m_T	4
n	0.27	R	0.00003
e_{d0}	0.61	β_r	0.2
e_{c0}	0.98	χ	1.0
e_{i0}	1.10		
α	0.50		
β	3.00		

4. INTERPRETATION OF RESULTS

Measured and computed results in this paper are in prototype unless stated otherwise.

4.1 SUBSURFACE AND EXISTING TUNNEL SETTLEMENT IN THE LONGITUDINAL DIRECTION OF NEW TUNNEL

Figure 3 shows comparison between settlement of ground located above the new tunnel and the existing tunnel. The settlement is normalized by diameter of the new tunnel. The abscissa denotes location of the advancing tunnel face normalized with tunnel diameter (Y/D).

In Test E2N5, settlement of the existing tunnel (Point A) and soil (Point B) was similar when the tunnel face approached to the existing tunnel (i.e. Y/D = -1.5 to -0.3). When the new tunnel advanced further away from the existing tunnel (i.e. Y/D = 0.3 to 1.5), soil at Point B settled larger compared with that of the existing tunnel (Point A). This is because stiffness of soil (Point B) decreased with reduction of confining stress (to be explained later in the following sections).

In Test E2,3N5, upper existing tunnel settlement (Point C) was almost the same as settlement of the lower existing tunnel (Point D). Only when the location of the tunnel face advanced further away from the existing tunnel (i.e. Y/D = 0.9 and 1.5), smaller settlement at Point D was observed compared with that at Point C. This is because the measurement location on the lower existing tunnel (Point D) is further away from excavated section. The presence of the lower existing tunnel or shielding effects resulted in smaller settlement of the upper existing tunnel in Test E2,3N5 (Point C) than that in Test E2N5 (Point A). By comparing the recommended serviceability limit of 20 mm (BD 2009), settlements of the existing tunnel in both tests were still within the allowable value.

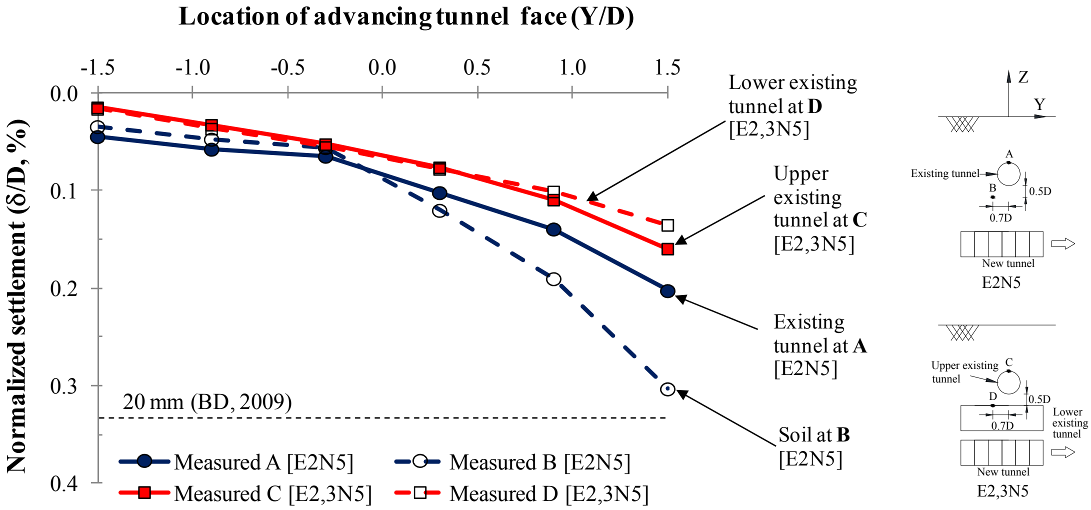

Figure 3: Measured settlement of soil and the existing tunnels with reference to location of advancing tunnel face

4.2 GROUND SURFACE AND EXISTING TUNNEL SETTLEMENT IN THE TRANSVERSE DIRECTION OF NEW TUNNEL

Ground surface settlement at the end of tunnel excavation is shown in Figure 4a. Maximum measured surface settlement in Test E2N5 was 0.20%D or 12 mm. In Test E2,3N5, where two existing tunnels were located above the new tunnel, the maximum ground surface settlement decreased to 0.14%D or 9 mm or reduced by 28% compared with those in Test E2N5. From back-analyzed results, there are minor discrepancies, but the overall trend of numerical modeling reasonably captured major aspects of the measured results. These discrepancies may be because some material parameters were adopted from the literature and empirical relationships.

To assist in the interpretation of ground surface settlements caused by single tunnel excavation in sand, a simplified Gaussian curve (Peck 1969) calculated considering volume loss of 2% and K of 0.35 (an average from 0.25 to 0.45 suggested by Mair & Taylor 1997) is shown. The maximum settlement from the Gaussian curve was 65% larger than the measured results in Test E2N5. In addition, the shape of surface settlement trough from Gaussian curve is narrower than those measured with the presence of existing tunnel. The computed ground surface settlement from the greenfield condition (N5) shows that the maximum surface settlement was similar to that from Gaussian curve. Ground surface settlements obtained from Gaussian curve and N5 were larger than in Test E2,3N5 up to 128%. It suggests that the effects of shielding provided by single existing tunnel (Test E2N5) and twin existing tunnels (Test E2,3N5) significantly reduce ground surface settlement caused by new tunnel excavation. Thus, calculating surface settlement caused by new tunnel excavation should consider the presence of any existing tunnel for reasonable estimation.

Settlement of the existing tunnel due to new tunnel excavation is shown in Figure 4b (refer to upper existing tunnel in Test E2,3N5). Similar to ground surface settlement, maximum measured tunnel settlement in Test E2,3N5 was 25% smaller than those in Test E2N5, due to the presence of the lower existing tunnel caused. Further away from the centerline of the new tunnel, the difference in settlement of the existing tunnel narrowed between the two tests. This is because the lower existing tunnel shielded reduction in stiffness and vertical stress of soil at the invert of the upper existing tunnel. Further explanation is given later in the following sections. The computed subsurface settlement in greenfield case (N5) also suggests that the presence of the existing tunnel significantly decrease subsurface settlement.

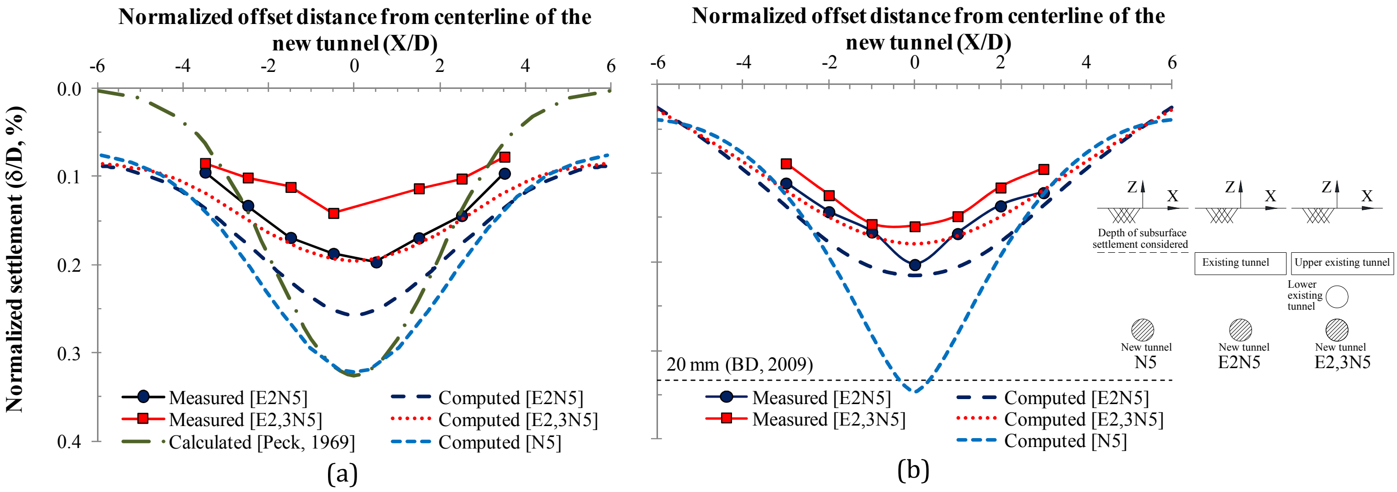

Figure 4: Settlement of (a) ground surface; (b) existing tunnel (upper existing tunnel for Test E2,3N5)

4.3 SHEAR MODULUS OF SOIL ALONG THE INVERT OF THE EXISTING TUNNEL

Figure 5 shows the normalized shear modulus (G_{before} / G_{after}) along the invert of the existing tunnel in order to explain the difference in tunnel settlement between the two tests. The shear modulus was considered before tunnel excavation (once the centrifugal acceleration had reached 60g, G_{before}) and after tunnel completion (G_{after}). Secant shear moduli (G_{before} and G_{after}) were calculated from deviatoric stress and deviatoric strain ($G = q/3\varepsilon_s$) at the end of each stage.

In Test E2N5, the normalized stiffness was significantly reduced at the location directly above the new tunnel (X/D = 0). As for three-tunnel interaction, there was almost no change in normalized shear modulus at the centerline of the new tunnel in Test E2,3N5. The minimum stiffness in Test E2,3N5 occurred at an offset distance of 0.5D, which is the offset distance of the springline of the lower existing tunnel, due to the shielding effects.

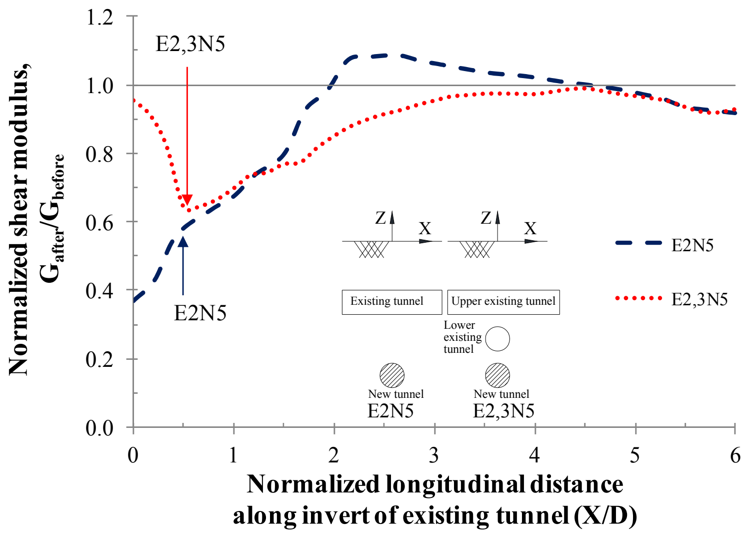

Figure 5: Computed normalized shear modulus of soil located directly underneath the invert of the existing tunnel at the end of tunnel excavation

Larger reduction in shear modulus in Test E2N5 than those in Test E2,3N5 above the new tunnel, caused larger tunnel settlement (refer to Figure 4b). In addition, larger reduction in vertical stress in the former also caused larger tunnel settlement than the latter (to be discussed in the next section).

4.4 INCREMENTAL VERTICAL STRESS ACTING ON THE INVERT OF THE EXISTING TUNNEL

Computed incremental vertical stress along the invert of the existing tunnel at the end of tunnel excavation is shown in Figure 6. The positive and negative signs denote increases and decreases in stress, respectively, compared with that prior to tunnel advancement.

In Test E2N5, maximum reduction in vertical stress of 115 kPa occurred directly above the new tunnel. Due to the shielding effects of the lower existing tunnel in Test E2,3N5, the maximum reduction in stress on the upper existing tunnel was significantly smaller than that in Test E2N5. As a result, the soil stiffness in the former was smaller than the latter (see Figure 5). At a distance away from the centerline of the new tunnel, stress increase was observed in every case as a result of stress redistribution.

The major reason for the larger tunnel settlement in Test E2N5 than those in Test E2,3N5 (refer to Figure 4b) was the reduction in stress and shear modulus of soil at the invert of the existing tunnel in the former were larger than the latter.

The reduction in vertical stress exceeded the recommended limit up to a distance of 1.5D from the centerline of the new tunnel in all tests. Due to stress redistribution along the invert of the existing tunnel, the increase in stress exceeded the allowable limit up to a distance of X/D = 4 in both tests. It suggests that before excavating a new tunnel underneath an existing one, the structural capacity of the existing tunnel should be reviewed up to an offset distance of 4D from the centerline of the new tunnel.

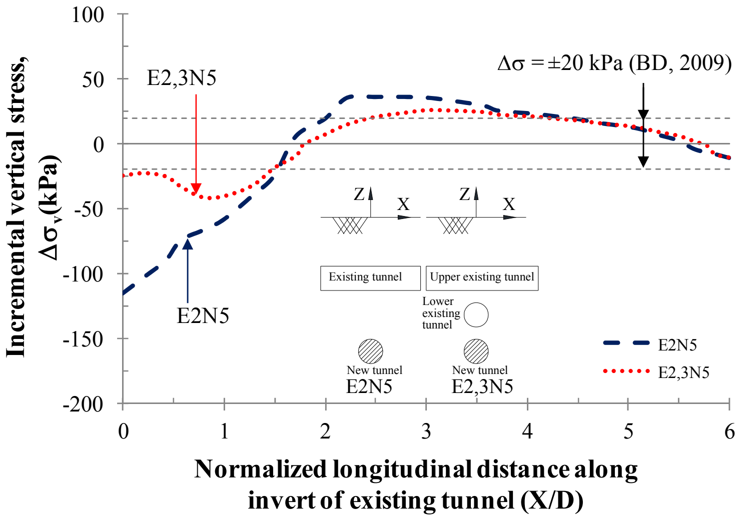

Figure 6: Computed incremental vertical stress acting along the invert of the existing tunnel after completion of new tunnel construction

5. CONCLUSIONS

Based on three-dimensional centrifuge modeling and numerical back-analysis, the following conclusions may be drawn:

(a) In the case of two existing perpendicular tunnels located above a new tunnel, the lower existing tunnel "shielded" the upper existing tunnel from the influence of tunnel excavation, causing 25% smaller measured settlement of the upper existing tunnel than in the case without the shielding effects. This is because the lower existing tunnel reduced the effect of stress reduction at the invert of the upper existing tunnel, resulting in larger mobilized soil stiffness in the case of two existing tunnels than in the case of just one existing tunnel.

(b) Ground surface settlement caused by tunnel excavation in greenfield conditions was up to 128% larger than those influenced by shielding effects. It suggests that the simplified method is not suitable for calculating ground displacement in multi-crossing tunnel interaction. Thus, presence of any existing tunnel should be considered to obtain a reasonable estimation of ground movement induced by new tunnel excavation.

(c) The location of significant reduction in vertical stress and stiffness of soil along the invert of the existing tunnel was from directly above the centerline of the new tunnel up to an offset distance of two times the tunnel diameter. Further away, there is an increase in stress and stiffness due to stress redistribution.

(d) The zone of large vertical stress induced on the invert of the existing tunnel extended up to four times the tunnel diameter from directly above the centerline of the new tunnel. Thus, the structural capacity of the existing tunnel should be reviewed and instruments should be installed up to this distance.

6. ACKNOWLEDGEMENTS

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Title, first name, last name: ***Dr. Thayanan Boonyarak***

Place of work: Seafco Public Company Limited

E-mail address: thayanan@seafco.co.th

Title, first name, last name: ***Chair Professor Charles W. W. Ng***

Place of work: The Hong Kong University of Science and Technology

E-mail address: cecwwng@ust.hk

Title, first name, last name: ***Associate Professor David Mašín***

Place of work: Charles University in Prague

E-mail address: masin@natur.cuni.cz