Influence of aspect ratio of basement on three-dimensional tunnel responses due to overlying excavation

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ABSTRACT

For the convenience of shoppers and users, there is an increasing demand for construction of basements in close proximity to existing tunnels. To ensure the safety and serviceability of existing tunnels, attention has been paid to the basement-tunnel interaction. However, most of previous numerical studies simply assumed the complex interaction as a plane strain problem and often they have overlooked effects of stress path and strain dependency on soil stiffness. Based on a dimensional analysis of the basement-tunnel interaction, three-dimensional numerical parametric study is conducted to explore the influence of aspect ratio (i.e., excavation length (L) along longitudinal tunnel direction / excavation width (B) along transverse tunnel direction) on tunnel responses due to basement excavation. Excavation length (L) varying from 2-10 \(H_e\) (i.e., final excavation depth) while excavation width (B) changing from 1-6 \(H_e\) are considered. Centrifuge test results are used to calibrate and verify soil model and soil parameters adopted. Because of larger inward wall movement and stress relief in a longer excavation, induced heave and transverse tensile strain in the tunnel increase with an increase in aspect ratio. When the aspect ratio and normalised excavation width (\(B/H_e\)) are larger than 2, induced tunnel heave at basement centre can exceed the allowable movement limit (i.e., 15 mm). Moreover, the transverse tensile strain of tunnel is larger than the cracking strain limit of unreinforced concrete (i.e., 150 \(\mu\varepsilon\)) when the aspect ratio is larger than 1.3. This implies that basements with a smaller aspect ratio impose less adverse effects on existing tunnel.

Keywords: Basement excavation, tunnel, three-dimensional, aspect ratio, dry sand

1. INTRODUCTION

For the convenience of shoppers and users, there is an increasing demand for construction of new basements in close proximity to existing tunnels. Due to the stress relief, basement excavation may cause adverse effects on existing tunnel.

To ensure the safety and serviceability of existing tunnels, attention has been paid to the basement-tunnel interaction via centrifuge tests (Zheng et al., 2010; Ng et al., 2013b; Huang et al., 2014; Shi, 2015) and numerical analyses (Lo and Ramsay, 1991; Doležalová, 2001; Sharma et al., 2001; Hu et al., 2003; Karki, 2006; Zheng and Wei, 2008; Liu et al., 2011; Huang et al., 2013; Shi et al., 2015; Shi, 2015). Deep excavations impose unsymmetrical stress relief and movement to existing tunnel not only along its transverse but also longitudinal directions. However, previous numerical studies often simply assumed the basement-tunnel interaction as a plane strain problem. Moreover, effects of strain and stress path dependency on soil stiffness were often overlooked in those studies.

At a given rectangular excavation area on plan, the longer side of basement could be parallel or perpendicular to the longitudinal tunnel direction. It is still not clear that which arrangement (i.e., aspect ratio) causes less adverse effects on existing tunnel. As far as the authors are aware, the influence of aspect ratio of basement on the three-dimensional tunnel responses is rarely reported.

In this paper, a dimensional analysis was firstly conducted to identify dimensionless groups affecting the complex interaction. Then, centrifuge test results in dry sand were used to calibrate soil model and soil parameters. Finally, numerical parametric study was conducted to investigate the influence of aspect ratio on the basement-tunnel interaction. Soil behaviour was simulated by a hypoplastic sand model, which can capture the effects of strain and stress path dependency on soil stiffness (Ng et al., 2013a; Wong et al., 2014; Ng et al., 2015).
2. DIMENSIONAL ANALYSIS

2.1 Parameters influencing basement-tunnel interaction mechanism

Fig. 1 shows schematic diagram of basement-tunnel interaction which involves 22 variables. Geometric variables included excavation length (L), excavation width (B), wall penetration depth (Hp), wall thickness (tw) and prop spacing (h). The relative basement-tunnel location included initial tunnel cover depth (C), tunnel diameter (D) and distance between tunnel springline and basement centre (F). Tunnel flexural stiffness (Et/T), tunnel axial stiffness (EtA/T), wall stiffness (Et/Iw) and soil stiffness (Ew) were also considered. Moreover, soil properties including soil density (ρ), void ratio (e), overconsolidated ratio (OCR), excess pore water pressure (u), undrained shear strength (cu), effective frictional angle at the critical state (φ′), dilatation angle (ψ) and effective cohesion (c′) were taken into account. Gravitational acceleration (g) was also considered.

Variables of 
- Young’s modulus (Es) and soil stiffness (T)
- and prop spacing (h)
- were dimensionless.
- Parameters of 
  - B, L, Hp, tw, F, C, D
  - only had dimension of length. Dimension of Es, c′ and u was 
- M/L² while dimensions of Et/Iw, EtA/T and Et/T were
  - M/L²/² and M/L², respectively. Moreover, dimensions of ρ and g were M/L² and L/T², respectively.

There were four major components 
- {Es, Et, ρ, g} in selected variables. Repeated variables were taken as Es, Et and g in this study. Accordingly, 19 dimensionless groups were obtained:

\[
\left(\frac{\varepsilon_s}{D}, \frac{\delta}{H}\right) = f\left[\frac{L, L, H_p, tw, h, F, H, C}{B, H, H', H', H, B, C, D}\right]
\]

Dimensionless groups affecting the complex interaction can be summarised into excavation geometry (L/B, L/Hp, Hp/Hp), tunnel location with respect to basement (F/B, H/H', C/D), stiffness of retaining system {tw/Hp, h/Hc, Et/Iw(Et/Hc)}, relative tunnel-soil stiffness {Et/T(Et/Hc), EtA/T(Et/Hc)} and soil properties {u, OCR, cc, cg} in the basement-tunnel interaction were rarely reported.

3. CENTRIFUGE MODELLING

Before conducting numerical parametric study, soil model and soil parameters were calibrated by centrifuge test results reported by Ng et al. (2013b).

This test was conducted in dry Toyoura sand with average dry density of 1542 kg/m³ (i.e., relative density of 38%). Both the model basement and tunnel were assumed to be wished-in-place. A basement with a length (L) of 18 m, a width (B) of 18 m and a final excavation depth (H) of 9 m in prototype was excavated directly above a tunnel. The penetration depth of retaining wall was 0.5 times the final excavation depth. Effects of basement excavation were simulated by draining heavy fluid away in-flight. Aluminium sheets with thickness of 12.7 mm (model scale) were used to simulate 0.96 m thick concrete wall in field (i.e., take Young’s modulus of reinforcement concrete as 35 GPa).

An aluminium tube with a diameter (D) of 6 m and a thickness (T) of 0.18 m in prototype was used to simulate the model tunnel. Assuming Young’s modulus of concrete as 35 GPa, this aluminium tube had the equivalent longitudinal and transverse stiffness as 420 mm and 230 mm thick slabs of concrete in prototype, respectively. The cover-to-tunnel diameter ratio (C/D) was 2, giving the distance between the tunnel crown and the formation level of the basement of 3 m (0.5 D).

Excavation induced heave and tensile strain in the tunnel were monitored in this test. More details of the test can be found in a previous study (Ng et al., 2013b).

4. NUMERICAL MODELLING

4.1 Finite element mesh and boundary conditions

Three-dimensional numerical analyses were conducted using the software package ABAQUS (Hibbitt et al., 2008). Fig. 2 shows three-dimensional finite element mesh used to back-analyse the centrifuge test. All the dimensions in model scale were identical to those in the test. Eight-node cubic elements were used
to simulate sand stratum and model wall while four-node shell elements were used to model tunnel. Soil movements normal to the four vertical sides of the mesh were restrained while they were restrained in all directions at the base of mesh. In following numerical parametric study, the distance between the model wall and the outer boundary of mesh was ensured to be larger than $2 H$, (final excavation depth) to minimise boundary effects. Perfect contacts were assumed at the soil-structure interface.

![Diagram of the model tunnel and retaining wall](image)

**Fig. 2 (a) Three-dimensional finite element mesh; (b) Intersection of tunnel and retaining wall**

### 4.2 Constitutive models and model parameters

A user-defined hypoplastic sand model was incorporated in the software (ABAQUS) to describe soil responses. Hypoplastic sand model proposed by Von Wolffersdorff (1996) was used and it required eight material parameters ($\phi', h_s, n, e_d, e_o, e_o$, $\alpha$ and $\beta$ summarised in Table 1). The model can capture state dependent sand responses. Intergranular strain concept requiring another five soil parameters ($m_f, m_b, R, \beta$, and $\kappa$ summarised in Table 1) was then incorporated into the model to capture effects of strain and stress path dependency on soil stiffness (Niemunis and Herle, 1997). Parameters ($\phi', h_s, n, e_d, e_o$, $e_o$) were obtained from Herle and Gudehus (1999), while parameters of $\alpha$ and $\beta$ were calibrated by triaxial test results from Maeda and Miura (1999). Stiffness degradation curve of Toyoura sand (Yamashita et al., 2000) was used to calibrate the five parameters for intergranular strain concept. The coefficient of at-rest earth pressure ($K_0$) of soil was assumed to be 0.5.

The model tunnel and the diaphragm wall were modelled as a linear elastic material with Young’s modulus and Poisson ratio of 70 GPa and 0.2, respectively. The unit weight of aluminium for the model tunnel and retaining wall was 27 kN/m$^3$.

**Table 1 Summary of material parameters adopted for the finite element analysis (Ng et al., 2013b)**

| Parameter                                      | Value |
|------------------------------------------------|-------|
| Angle of internal shearing resistance at critical state, $\phi'$ (°) | 30    |
| Hardness of granulates, $h_s$ (GPa)            | 2.6   |
| Exponent controls the shape of limiting void ratio curves, $n$     | 0.27  |
| Minimum void ratio at zero pressure, $e_{o0}$  | 0.61  |
| Critical void ratio at zero pressure, $e_{o0}$ | 0.98  |
| Maximum void ratio at zero pressure, $e_{o0}$  | 1.10  |
| Exponent controls the dependency of peak friction angle on relative density, $\alpha$ | 0.14  |
| Exponent controls the dependency of soil stiffness on relative density, $\beta$ | 3     |
| Parameter controlling initial shear modulus upon $180^\circ$ strain path reversal, $m_f$ | 8     |
| Parameter controlling initial shear modulus upon $90^\circ$ strain path reversal, $m_b$ | 4     |
| Size of elastic range, $R$                    | $2\times10^3$ |
| Parameter controlling degradation rate of stiffness with strain, $\beta$ | 0.1   |
| Parameter controlling degradation rate of stiffness with strain, $\beta$ | 1.0   |
| Coefficient of at-rest earth pressure, $K_0$  | 0.5   |

$^a$ Obtained from Herle and Gudehus (1999).

$^b$ Calibrated from triaxial test results of Toyoura sand (Maeda and Miura, 1999; Yamashita et al., 2000).

### 4.3 Numerical modelling procedures

a). Established the initial boundary and stress conditions of soil at 1 g (i.e., gravitational acceleration) by assuming the coefficient of at-rest earth pressure of soil ($K_0$) as 0.5. Then, equivalent fluid pressures were applied on the wall and the formation level of basement to simulate the existence of heavy fluid (ZnCl$_2$).

b). Increased the gravitational acceleration (g) to the entire mesh in four stages. At each stage, corresponding lateral and vertical fluid pressures applied on the wall and the formation level of the basement increased as well.

c). Decreased the lateral and vertical fluid pressures applied on the wall and the formation level of basement simultaneously to simulate excavation.

### 5. NUMERICAL BACK-ANALYSIS OF CENTRIFUGE TEST

Fig. 3a shows comparison of the measured and computed heaves at the crown of the tunnel along its longitudinal direction. As expected, heave is induced in the tunnel along its longitudinal direction due to vertical stress relief. As basement excavation proceeds
further, induced tunnel heave increases with the excavation depth. The maximum tunnel heave occurs at the basement centre and gradually decreases with an increase in normalised distance from basement centre. Basement excavation induced influence zone of longitudinal tunnel heave is 1.2 L (basement length) away from basement centre. In general, the computed tunnel heave is in fairly good agreement with the measured one, especially when the excavation depth is 9 m. In following parametric study, only the maximum tunnel heave is presented.

Fig. 3b shows comparison of the measured and computed strains at the outer surface of tunnel lining along its transverse direction. Only strain induced at basement centre is compared. Due to symmetrical stress changes around the tunnel lining, profiles of the tunnel strains are symmetrical. It is found that tensile strain is induced at the outer surface of tunnel crown, shoulder, knee and invert, while compressive strain is observed at the outer surface of tunnel springline. It is demonstrated that the existing tunnel is vertically elongated and horizontally compressed. Because of the largest stress relief at the tunnel crown, the maximum tensile strain is always observed at this location. At a given tensile strain in the tunnel along its transverse direction, the crown is more vulnerable to cracking than other regions. Upon completion of excavation, the computed maximum transverse tensile strain of tunnel at the crown is only 15% larger than the measured one. In following parametric study, only the maximum tensile strain at the tunnel crown is presented.

Fig. 3c shows comparison of the measured and computed strains at the crown of the tunnel along its longitudinal direction. If the basement-tunnel interaction is assumed as a plane strain problem, strain of tunnel along its longitudinal direction cannot be obtained. In terms of longitudinal tensile strain of tunnel, plane strain assumption causes unconservative prediction of tunnel responses. Due to differential heave induced in the tunnel as shown in Fig. 3a, the existing tunnel bends upward, as expected. Along the tunnel crown, tensile and compressive strains are induced at the hogging and sagging regions, respectively. Both the measured and computed strains in the tunnel along its longitudinal direction increase with an increase in excavation depth. This is consistent with variations of the tunnel heave with excavation depth. Induced maximum longitudinal strain of tunnel at the hogging region is about 3 times as large as that at the sagging region. Thus, it should be conservative to use the maximum tensile strain of tunnel induced at basement centre as a design parameter. Accordingly, the longitudinal tensile strain of tunnel at basement centre is presented in the following section. Upon completion of excavation, the computed maximum tensile strain of tunnel (i.e., 81 με) is 17% larger than the measured one (i.e., 69 με). It is demonstrated that adopted numerical modelling procedures and soil parameters are reasonable.
maximum tunnel heave and aspect ratio ($L/B$) of basement. It is found that, at a given normalised excavation width ($B/H_e$), induced maximum tunnel heave at basement centre increases with an increase in normalised aspect ratio, but at a reduced rate. This is due to larger stress relief and inward wall movement are induced in a longer excavation. Excavation induced tunnel heave at basement exceeded the allowable movement limit (i.e., 15 mm given by Land Transport Authority, 2000) when the aspect ratio and normalised excavation width are larger than 2. Moreover, the maximum tunnel heave is larger than the movement limit (i.e., 20 mm by Buildings Department, 2009) when the aspect ratio is larger than 3.4 and normalised excavation width is wider than 2. This implies that basements with a smaller aspect ratio impose less adverse effects on existing tunnel.

Fig.4b shows relationships between transverse tensile strain of tunnel and aspect ratio of basement. At a given normalised excavation width, induced transverse tensile strain in the tunnel at basement centre increases with normalised aspect ratio, but at a reduced rate. As the aspect ratio of basement is increased, larger inward wall movement results in bigger tunnel deformation. When the aspect ratio of basement was larger than 1.2, excavation induced maximum transverse tensile strain of tunnel exceeded allowable cracking strain limit of unreinforced concrete (i.e., 150 με given by American Concrete Institute, 2001). In order to reduce excavation induced transverse tensile strain in the tunnel, short & wide basement (i.e., short side of basement is parallel to tunnel axis) rather than long & narrow basement should be excavated above existing tunnel. For example, excavation area of case A (i.e., $L/H_e = 4$, $B/H_e = 2$) is the same as that in case B (i.e., $L/H_e = 2$, $B/H_e = 4$). However, the aspect ratios are 2 and 0.5, respectively, in cases A and B. Excavation induced maximum tunnel heave and transverse tensile strain in cases A and B are 2.1 and 3.1 times of those in case B.

Fig.4c shows variations of induced longitudinal tensile strains in the tunnel with aspect ratio of basement. When the normalised excavation width is equal to one, the longitudinal tensile strain induced in the tunnel decreases with an increase in the aspect ratio. For remaining cases, induced tensile strain of tunnel increases with aspect ratio first, then decreases rapidly with an increase in aspect ratio. As the aspect ratio is increased, the basement-tunnel interaction at basement centre approaches a plane strain condition. It seems to suggest that basements with a larger aspect ratio impose less adverse effects on existing tunnel. It should be noted that induced maximum longitudinal tensile strain of tunnel is within the cracking strain limit given by American Concrete Institute (2001). Moreover, the tensile strain induced in the tunnel along the transverse direction is significantly larger than that along the longitudinal direction. This is due to the flexural stiffness of tunnel along the transverse direction is significantly smaller than that along the longitudinal direction. If possible, a small aspect ratio of basement excavated above an existing tunnel should be adopted to reduce excavation induced adverse effects on the tunnel.
7. CONCLUSIONS

Based on the current study, the following conclusions may be drawn.
1. Based on a dimensional analysis of the basement tunnel interaction, key dimensionless groups affecting the complex interaction can be summarised into five groups, i.e., excavation geometry, tunnel location with respect to basement, stiffness of retaining system, relative tunnel-soil stiffness and soil properties.
2. Due to larger stress relief and inward wall movement in a longer excavation, induced heave and transverse tensile strain of tunnel increase with an increase in the aspect ratio. This implies that basements with a larger aspect ratio are more likely to induce adverse effects on existing tunnel.
3. When the aspect ratio ($L/B$) and normalised excavation width ($B/H_e$) are larger than 2, excavation induced tunnel heave at basement centre exceeds the allowable movement limit (i.e., 15 mm given by Land Transport Authority, 2000). Moreover, excavation induced transverse tensile strain of tunnel is larger than the cracking strain of unreinforced concrete (i.e., 150 $\mu$ε given by Transport Authority, 2000). Moreover, excavation induced movement limit (i.e., 15 mm given by Land Transport Authority, 2000).

ACKNOWLEDGEMENT

The authors would like to acknowledge the financial support provided by the Research Grants Council of the HKSAR (General Research Fund project no. 617511).

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