Liner forces response to very close-proximity tunnelling in soft ground

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ABSTRACT

Since there are limited literature focused upon the tunnel liner forces response to the very close-proximity tunnelling, a consensus has not yet reached. This study characterised the response of tunnel liner forces to the excavation of soft soil twin tunnels in the very close proximity, with reference to the field measurements and 2D numerical simulations. The “belta” method and the “volume loss” method were to distribute the earth load to the liner and capture the effect of ground loss, respectively, and the “local strain” method was to reproduce the effect of tunnel advance. The predictions were in reasonable agreement with the field measurements except the tunnel invert where the liner might not be in good contact with the surrounding soil. The principle stress change and rotation were analysed. Parametric studies about the joint number and orientation, ground deformability and rotational stiffness of joint were conducted. The critical angles for the distribution of odd number joint were suggested.

Keywords: very close-proximity tunnelling, ground deformability, rotational stiffness ratio, joint number and distribution, principal stress

1 INTRODUCTION

With the development of infrastructure underground during urbanisation process, it is unavoidable to build new tunnel in the very close proximity of the existing running tunnel. The new tunnel construction not only endangers the stability of the existing running tunnel, but also impacts adjoining properties. Mitigating the impacts of new tunnel construction on the existing tunnel’s liner deformation and associated change in the internal force has been deemed to be essential (Zhang et al. 2014; Boonyarak et al. 2015; Cheng et al. 2017a; Cheng et al. 2017b; Cheng et al. 2018). Physical model test, theoretical analysis and numerical simulation have been used for analysing the interactive behaviour of twin tunnels. Kim et al. (2004) conducted a series of physical model tests of close-proximity tunnels and the results of the parallel tunnel tests showed that the interactive effects are linked to the redistribution of stresses within the soil caused by liner deformations and ground loss. Fuentes (2015) provided a solution to the long-standing problem of calculating internal force distributions based upon field displacement measurements and it worked without the input of any boundary conditions. Do et al. (2014) simulated the tunnel excavation process through three-dimensional finite element analysis to assess the structural forces of tunnel lining and associated ground displacements. However, since past studies focused upon the response of tunnel liner forces to construction of very close-proximity tunnels are very limited, a consensus has not yet reached.

The objectives of this study are: i) to characterise the response of liner forces to the effect of very close-proximity tunnelling, with reference to the field measurements and 2D numerical simulations, ii) to capture the response of liner forces by using the proposed FE method and iii) to highlight the adverse impacts of very close-proximity tunnelling on the preceding tunnel.

2 METHODOLOGY

Considering tunnel liner as a simple supported beam, its curvature function was assumed as a second-order function, meaning that six strainmeters should at least be deployed to three different locations, and for each location one strainmeter was responsible for measuring tensile strain and the other for measuring compressive strain. In this study, four strainmeters were installed at two different locations of one tunnel liner and two at one location of adjacent tunnel liner, enabling the ability to investigate the transfer characteristics of liner forces between adjacent liners. The measured strains
were first converted to the curvatures and subsequently the bending moment for further analysis. Although the deflection function for the tunnel liner can be obtained by integrating the curvature function, this is outside the scope of this study and would be discussed in another paper. A series of FE analyses were conducted through the commercial FE package PLAXIS where a calculation performed with a full hinge was utilised to estimate the equivalent rotational stiffness $k_{RO}$ for all joints. The “belta” method and the “volume loss” method were to distribute the earth pressures to the liner and capture the effect of ground loss, respectively, and the “local strain” method was to reproduce the effect of tunnel advance. The liner force predictions from the aligned-inclined close-proximity tunnels were compared to the field measurements to verify the applicability of the proposed method.

3 PROJECT DESCRIPTION

3.1 Background

The very close-proximity tunnels of 6.1 m in diameter are to link Station G14 and Station G16 together and belong to a part of the Songshan Line of the Taipei Rapid Transit System (TRTS) (Fig. 1). There were nine instrumented tunnel rings deployed along the tunnel alignment, as shown in Fig. 1.

Since the northbound tunnel was excavated following the southbound tunnel excavation, the instrumented tunnel rings containing earth pressure transducer and porewater pressure transducer as well as vibrating wire strainmeter at different liner positions of the southbound tunnel were utilised for measuring not only the liner internal forces, but also the earth and porewater pressures. Fig. 2 shows the configuration of the typical instrumented tunnel ring applied to the very close-proximity tunnels construction.

3.2 Engineering geology

The ground comprises a thick soft alluvial formation (the Songshan formation) of alternating soft clay and silty sand layers, of which six has been distinguished, underlain by a gravel formation (the Chingmei formation). The thickness of the Songshan formation varies from 40 m to 55 m at the worksite, while for the Chingmei formation, it varies from 53 m to 58 m. Because the silty clay presents in Songshan II, the underlying Songshan I and Chingmei formation can be regarded as a confined aquifer, with the piezometric pressure about 7 m below the ground surface. The phreatic surface was assumed to be 3.3 m deep (Fig. 3). The displacements were constrained in both directions at the bottom, and zero horizontal displacement was imposed at the lateral boundaries. Apart from that, the direct shear (DS) test results show that the cohesion and friction angle for the silty sand vary in the ranges of 5-8 kPa and 31.7-33.3 deg., respectively, while the cohesion for the silty clay varies in the range of 11-22 kPa according to the unconsolidated undrained (UU) triaxial test results.

4 NUMERICAL SIMULATION

4.1 Proposed FE model

The phreatic surface was assumed to be 3.3 m deep (Fig. 4). The displacements were constrained in both directions at the bottom, and zero horizontal displacement was imposed at the lateral boundaries. The silty clays were modeled using the soft soil model, while the tunnel liner behaved as elastic material. Tables 1, 2 and 3 summarise the properties of the soil
and tunnel liner used in the numerical simulation.

Table 1 Soil properties used in the Mohr-Coulomb model

| Layer | c' (kN/m²) | φ' (deg.) | E_s (kN/m²) | ν | K_0NC |
|-------|------------|----------|-------------|---|-------|
| 1  | Silty clay (CL) | 13 | 0 | 4500 | 0.49 |
| 2  | Silty sand (SM) | 5 | 33.3 | 10000 | 0.25 |
| 3  | Silty sand (SM) | 8 | 31.7 | 11000 | 0.25 |
| 4  | Silty clay (CL) | 13 | 0 | 10000 | 0.49 |

Note: c'=cohesion; φ'=friction angle; E_s=Young’s ground modulus; ν=Poisson’s ratio.

Table 2 Soil properties used in the Soft Soil model

| Layer | c' (kN/m²) | φ' (deg.) | λ* | κ* | K_0NC |
|-------|------------|----------|-----|----|-------|
| 1  | Silty clay (CL) | 11 | 0 | 0.076 | 0.015 |

Note:λ*=modified compression index; κ*=modified swelling index; K_0NC=coefficient of lateral stress in normal consolidation.

Table 3 Properties of plate element (tunnel liner)

| Thickness (m) | Radius (m) | EA (kN/m) | EI (kN·m²/m) | W (kN/m²) | ν |
|---------------|------------|------------|--------------|------------|---|
| 0.25          | 2.925      | 1.05×10⁷   | 5.46×10⁴   | 6.3        | 0.15 |

Note: EA=axial stiffness; EI=bending stiffness; W=weight of the tunnel liner minus out the weight of the soil removed.

Fig. 4. FE mesh used in the analysis.

The “belta” method, while commencing the southbound tunnel (preceding tunnel) excavation, was used to distribute the earth and porewater pressures to the tunnel liner. Upon the activation of tunnel liner, the ground loss induced by the inability of grouts to fill up the void left between the liner and the surrounding soil was simulated by imposing a positive volumetric strain, referred to as the “volume loss” method. The effect of tunnel advance, induced when the northbound tunnel (succeeding tunnel) approached to the southbound tunnel, was achieved through imposing a positive volumetric strain to soil clusters in the northbound tunnel. When the succeeding tunnel passed through the preceding tunnel, the hoop force of the preceding tunnel was reduced to a great extent by the pass-through effect. The ground in fact contracted more, most likely due to ground closure induced by the presence of surrounding gaps or due to dissipation of excess porewater pressure generated during the succeeding tunnel excavation. Thus, a negative volumetric strain was applied to soil clusters surrounding the succeeding tunnel to capture the combined effects of pass-through and ground closure. This is also termed “local strain” method.

4.2 Model validation

(1) Response of liner forces to very close-proximity tunnelling: Comparisons of predictions with field measurements are deemed to be essential for performance evaluation prior to applying the proposed FE method to engineering practice. Amongst the nine instrumented tunnel rings, Ring H was thus chosen for analysis because the aligned-inclined close-proximity tunnels were excavated with a spacing less than 1.5 m. The variations of the hoop forces and bending moments against the side walls just showed a tendency similar to that of very close-proximity tunnelling vanished or approached to zero after the succeeding tunnel excavation passed through the Ring H. The sudden changes in the hoop forces were increased very quickly. This indicates that the earth and porewater pressures began to act upon the tunnel liner. There were sudden increases in the hoop forces in the second stage, most likely because of the excess porewater pressure generation, resulting from the grout injection into the possible voids behind the tunnel liner (termed backfill grouting hereafter). The significant variations of hoop forces in the third stage were attributed to the succeeding tunnel excavation at some distance from the Ring H. The relatively small variations in the fourth stage were caused by the succeeding tunnel excavation in the very close proximity of the Ring H. In the final stage, the hoop forces first declined sharply and then showed sudden changes. The declines were because the effect of very close-proximity tunnelling vanished or approached to zero after the succeeding tunnel excavation passed through the Ring H. The sudden changes in the hoop forces were due to the backfill grouting-induced excess porewater pressure generation. It is interesting to note that the hoop forces against the tunnel invert, however, showed little changes throughout the very close-proximity tunnelling process. It seemed to be resulted from the poor contact between the tunnel liner and the surrounding soil, impeding the transfer of earth and porewater pressures to the liner. The variations in the bending moments against the tunnel arch and invert presented a tendency similar to but smaller than those in the hoop forces, while the variations against the side walls just showed an opposite tendency, as shown in Fig. 5b. The sudden changes in the bending moments in the final stage appeared to be related to the regime of excess porewater pressure generation induced by the backfill grouting.
in the phase of reaching the preceding tunnel, and (c) response of hoop force in the phase of passing through the preceding tunnel.

(2) Comparisons of field measurements with predictions: Fig. 6 shows the comparisons of the hoop force between the field measurements and the predictions against the phases of approaching to, reaching and passing through the preceding tunnel, while Fig. 7 shows the associated displacement vector maps and principal stress distributions. The predictions against various positions were found in reasonable agreement with the field measurements except the tunnel invert, as indicated in Fig. 6. The measured hoop force due to a poor contact with the surrounding soil reduced to a negative value. This led to an inability of the proposed model to simulate the separation of tunnel liner from the surrounding soil. The hoop forces slightly deviated at the reference angle (ω) ranges of 36-72° and 216-288°. This deviation from the field measurements might be related to the difference in the stress relief phenomenon induced by the succeeding tunnel excavation between the reality and the proposed model. Notwithstanding that, the results of an application of the proposed model have been verified its applicability to the very close-proximity tunnelling.

5 RESULTS AND DISCUSSION

5.1 Principle stress change and rotation

As discussed, the preceding tunnel was squeezed initially and then deformed towards the succeeding tunnel. Thus, it can be postulated that the preceding tunnel deformed towards the succeeding tunnel as the surrounding earth pressures due to the stress relief were greater in the short axis of the deformed elliptic preceding tunnel than in the long axis. This also implies that the principal stress changed both in magnitude and direction because of the effect of very close-proximity tunnelling. The effect of very close-proximity tunnelling on the change of principal soil stresses in magnitude was illustrated in Fig. 7. Most of the soils surrounding the preceding tunnel showed a significant change after the succeeding tunnel excavation except the soil named as “3” for which the mean and deviatoric stresses (p and q) showed small change. This was attributed to the regime of stress relief. It is interesting to note that the deviatoric stress q for the soil named as “4” was aggravated to a considerable extent by the effect of very close-proximity tunnelling, which also indicated a high potential of shear failure. In addition to the change in the magnitude of principal stress, the rotation of the principal stress was investigated against the phases of reaching and passing through the preceding tunnel, as illustrated in Fig. 8. The rotation angle can be derived from Eq. (1). A positive α value indicates that the principal stress rotates counterclockwise, and a negative value indicates that the principal stress rotates clockwise.
The rotation of the principal stress was distinct as the \( \omega \) angle was at ranges of 36-72\(^\circ\) and 216-288\(^\circ\). The \( \omega \) angle ranges were consistent with those appeared the distinct change in the magnitude of principal stress. The principal stress initially rotated clockwise with respect to the \( \omega \) angle range of 36-72\(^\circ\) when the succeeding tunnel reached the preceding tunnel, and it, in turn, rotated counterclockwise when the succeeding tunnel passed through the preceding tunnel. With respect to the \( \omega \) angle range of 216-288\(^\circ\), the principal stress rotated counterclockwise when reaching the preceding tunnel and then rotated clockwise when passing through it. To summarise, the effect of very close-proximity tunnelling had distinct implications on the change in the magnitude of principal stress. Also, the rotation of the principal stress when passing through the preceding tunnel worked the other way around as compared to reaching the preceding tunnel.

\[
\alpha = \frac{1}{2} \arctan \left( \frac{2 \tau_{\text{ref}}}{\sigma_2 - \sigma_1} \right)
\]

5.2 Influence of joint number and orientation

Since the number and the orientation of joints have significant impact on the maximum bending moment induced in a tunnel liner, the analysis took the joint number of 4-7 and the reference angle \( \omega \) of 15-90\(^\circ\) into account, as shown in Fig. 9. Generally, the greater the joint number, the smaller the maximum bending moment, as illustrated in Fig. 9a. The variation of the maximum bending moment from the distributions of odd and even number joint showed their own feature, as illustrated in Fig. 9b. The results also indicate that the liner joint design should be aware of the distribution of odd number joint. The critical orientation \( \omega \) was at 55-75\(^\circ\) for the 4-joint distribution and at 0-20\(^\circ\) and 45-70\(^\circ\) for the 6-joint distribution in case the distribution of even number joint was designed. The main cause of the maximum bending moments, resulting from the 4-joint and 6-joint distributions, in excess of 116 kN-m/m was attributed not only to the regime of aligned-inclined close-proximity tunnelling, but also to the distance of installed joint to the location of maximum bending moment.

5.3 Influence of ground deformability

A dimensionless factor, termed the rotational stiffness ratio, \( \lambda = k_{\text{rot}}/E_l l_l \), proposed by Lee et al. (2001), was introduced to illustrate the relationship between the liner forces and the ground deformability.
which is represented using Young’s ground modulus $E_s$, in function of the rotational stiffness $k_{RO}$. For simplification purposes, the rotational stiffness $k_{RO}$ assigned to all the joints in the tunnel ring were assumed to be similar. The effect of ground deformability on the change in liner forces was investigated by considering a range of Young’s ground modulus $E_s$ of 2 MPa to 20 MPa, corresponding to a ground condition range from soft soils to hard soils, for each given $\lambda$ value of 0.01, 0.1, and 1, as indicated in Fig. 10a. The maximum hoop force was not sensitive to the change in the $E_s$ for a given $\lambda$ value. The maximum hoop force was larger as subjected to larger $\lambda$ values. Similarly, the maximum bending moment was not sensitive to the change in the $E_s$ for a given $\lambda$ value. The maximum bending moment increased with increasing the $\lambda$ value. Additionally, the reduction in the maximum bending moment with increasing the $E_s$ value was more significant for larger $\lambda$ values than smaller $\lambda$ values. Larger the $\lambda$ value, the larger the liner forces. The liner forces decreased with increasing the $E_s$ and this reduction became more pronounced as subjected to larger $\lambda$ values.

Fig. 10. Influence of Young’s ground modulus on the liner forces for different rotational stiffness ratios: (a) maximum hoop force and (b) maximum bending moment.

6 CONCLUSIONS

Based upon the results and discussion, some main conclusions can be drawn as follows:

(1) The change in the magnitude of principal stress was affected to a considerable extent by the effect of very close-proximity tunnelling. As passing through the preceding tunnel, the rotation of the principal stress worked the other way around as compared to reaching the preceding tunnel.

(2) Generally, the maximum bending moment was reduced with increasing the joint number. The variation of the maximum bending moment from the distributions of odd and even number joint showed their own feature. The joint design should be aware of the distribution of odd number joint while aligned-inclined tunnelling in the very close proximity of the preceding tunnel. The critical orientation $\omega$ was at 55-75° for the 4-joint distribution and at 0-20° and 45-70° for the 6-joint distribution in case the distribution of even number joint was designed.

(3) Hard soils sheared more loads from the tunnel liner, and this would especially hold true as subjected to larger $\lambda$ values. On the other hand, the liner forces were affected to a significant extent by the $\lambda$ range of 0.01 to 1 and the relationship between the liner forces and the rotational stiffness ratio $\lambda$ was not sensitive to the change in the $E_s$ values varying from 2 to 20 MPa.

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