Rock–liner interaction mechanism involving a nonuniform deformation pattern in shallow shield tunnels

Cui Lan¹, ², ⁴, Tang Xiongjun³, ⁴, Zhu Zeqi¹, ², ⁴, Sheng Qian¹, ², Alfonso Rodriguez Dono⁵

1. State Key Laboratory of Geomechanics and Geotechnical Engineering, Institute of Rock and Soil Mechanics, Chinese Academy of Sciences, Wuhan 430071, China
2. University of Chinese Academy of Sciences, Beijing 100049, China
3. China Railway Siyuan Survey and Design Group Co., Ltd., Wuhan 430063
4. National and Local Joint Engineering Research Center of Underwater Tunneling Technology, Wuhan 430063
5. Jordi Girona 1-3, UPC Campus Nord D2-306/2, 08034 Barcelona

Email Address: lcui@whrsm.ac.cn

Abstract: A numerical plane-strain model is established on the basis of the nonuniform deformation pattern proposed by Loganathan and Poulos. A 2D analysis method of the rock–liner interaction is proposed for the shallow shield tunnel. The accuracy of the proposed method is verified by comparing the surface settlement curve obtained through the analytical method and field measurement. Variations in rock deformation and rock stress are investigated using the proposed method in terms of the influence of gap parameter, deformation modulus, unit weight of the rock, and diameter and depth of the shield tunnel. Results indicate that the liner stress is significant for a shield tunnel with a large diameter, and the buoyancy effect is evident, indicating that the liner is apt to the failures for the shield tunnel with a large diameter. The unit weight significantly affects the liner and rock stress, the effect of the depth of the shield is less, and the effect of the deformation modulus is the smallest.

Keywords: tunnel; shield; plane strain; rock; liner

1. Introduction

Ground surface settlement and liner deformation or failures induced by the construction of shallow shield tunnels are critical issues in the safety control of shield tunneling, which has raised concern in academic and industrial fields. Ground surface settlement can cause the failure and collapse of surrounding structures, resulting in immeasurable losses on public property. Peck et al. [1], Park [2], and Lee [3] derived the surface settlement curves by using the empirical, analytical, and numerical methods, respectively. In addition to settlement problem, support structure failure and ground instability pose a potential threat to constructions in shield tunneling. As such, the rock–liner interaction mechanism has
been widely explored. Bobet [4], Chou et al. [5], and Zhang et al. [6] applied analytical methods and derived the deformation and stress in the ground and the liner based on elastic theory. Cui et al. [7], He et al. [8], and Ye et al. [9] used numerical methods and generated the 3D tunnel modeling and obtained the flow pattern of liner stress at different construction stages and for practical cases. The 2D plane-strain numerical method is more simplified than the 3D numerical method because of a shorter calculation time. The 2D numerical method has also been widely used to solve the issue. For example, Yang et al. [10], Luo et al. [11], Ngoc et al. [12], and Ponlawich et al. [13] established the plane-strain model of the shallow tunnel and analyzed the interaction between a liner and a rock with multiple stress relief factors. Ramoni et al. [14], Wang et al., [15], and Liu et al. [16] simulated the tunneling process by via the stress relieving method, proposed variations in rock stress, and determined the most appropriate supporting time in various sections.

Most 2D analyses have focused on a liner that initially interacts with a rock immediately after the rock core is excavated. However, in engineering practice, liner installation is generally delayed [10-13] because a liner cannot come in contact with a rock at the time when the rock is removed. Hence, several studies [14-16] have assumed that rock stress is relieved before liner installation, but most of them have mainly explored deep tunnels. As for shallow shield tunnels, ground loss occurs, so the ground near the tunnels is susceptible to uneven deformation. However, the stress relief factor cannot be easily used to simulate this uneven rock deformation pattern. Therefore, in 2D analysis, the rock–liner interaction mechanism should be discussed with an appropriate nonuniform deformation pattern.

Based on the nonuniform deformation pattern proposed by Loganathan and Poulos [17], the excavation model of shallow shield tunnels is presented using the finite element method PHASE2. A calculation method of the rock–liner interaction of shallow shield tunnels is proposed. The accuracy of the proposed calculation method is verified by comparing the surface settlement curves obtained through the analytical method and field measurement. Through parametric analysis, the stress and deformation distributions of the rock and the liner stress are discussed in terms of the influence of the gap parameter, rock property, tunnel diameter, and depth. This study provides a theoretical basis for designing the support structures of shallow shield tunnels.

2. Nonuniform deformation pattern

A nonuniform deformation pattern influences ground deformation in shield tunneling. Sagaseta [18], Verruijt and Booker [19], and Bobet [4] assumed that ground deformation around a shield tunnel is uniform; then, they derived the analytical solution of surface settlement based on this assumption (Fig. 1(a)). However, practical engineering [20-23] indicates that the convergence of the ground is unevenly deformed during tunneling [17]. Loganathan and Poulos [17] and Park [2] modified the pattern by proposing a nonuniform deformation pattern (Fig. 1(b)).

Fig. 1 Deformation pattern in the tunnel section:

(a) uniform and (b) nonuniform deformation patterns

In the nonuniform deformation mode, vault settlement and invert heave determine the ground deformation at the tunnel periphery with various locations [2,17]. In Fig. 2, vault settlement is g, and invert heave is 0 [21-22]. g is the gap parameter proposed by Lee et al. [23].

Fig. 2 Nonuniform deformation pattern
g denotes the vertical settlement at the tunnel vault, which is an important parameter that quantitatively illustrates the ground loss caused by the advancement of the tunnel face. The definition of \( g \) is shown in Fig. 3 and presented as follows:

\[
g = G_p + U_{3D} + \omega, \tag{1}
\]

where \( U_{3D} \) is the equivalent 3D elasto-plastic deformation induced by the excavation of the tunnel face, \( \omega \) is the equivalent ground loss with regard to the construction technique, and \( G_p \) is the physical gap between the outer external diameter of the segment and the outer external diameter of the shield shell, which is obtained as

\[
g = 2\Delta + \delta, \tag{2}
\]

where \( \Delta \) and \( \delta \) are the thickness of the shell and the physical gap of the shield tail, respectively.

3. 2D analysis of the rock–liner interaction of a shallow shield tunnel

For the shallow shield tunnel, before liner installation, the surrounding ground shows a nonuniform deformation pattern. Afterward, as the liner is added, the ground begins to interact with the liner. This process is difficult to simulate by using the stress relieving method. Thus, based on the displacement control method, a 2D analysis method of the rock–liner interaction of shallow shield tunnels is proposed by utilizing the finite element software PHASE2. The procedure is described as follows:

(i) A model is generated.
(ii) Gravity stress is added, the boundary conditions of the model are set, and the model is run until equilibrium is reached.
(iii) The tunnel is excavated, the nonuniform deformation pattern is added to the tunnel periphery, the model is run until equilibrium is reached, and the rock stress at the tunnel periphery is extracted.
(iv) The liner is installed, the rock stress is relieved, the model is run until equilibrium is reached, and the stress and deformation of the rock and liner are captured.

4. Verification of the proposed method

For comparison, four typical shield tunnel cases are described. Table 1 displays the geometrical parameters of the tunnel and mechanical parameter of the ground. The ground settlement solved by the proposed method is compared with three analytical solutions and the field test results of the four cases. As shown in Fig. 4, methods I and II refer to the analytical solution by Verruijt and Booker [19]. In this solution, the uniform ground deformation at the tunnel periphery and the convergence deformation are \( g/2 \) and \( g \), respectively. Method III corresponds to the analytical solution proposed by Loganathan and Poulos [17]. In this method, the nonuniform deformation pattern is utilized as in the proposed method by which the vault settlement and the bottom heave are \( g \) and 0, respectively. This section aims to verify the proposed method by comparing the settlement of the ground surface, and the interaction between the ground and the liner is not considered.

In this research, the ground is assumed to show the elastic behavior. Several cases of plastic behavior with similar soil types have been calculated. Almost no plastic zone appears in the ground. This finding implies that the ground mostly behaves as elastic. The assumption in this research is appropriate.
Table 1 Geometrical parameters of the tunnel and mechanical parameters of the ground

| Tunnel name | Bangkok tunnel | Barcelona tunnel | Heathrow tunnel | Thunder bay tunnel |
|-------------|----------------|------------------|-----------------|-------------------|
| g/mm       | 81             | 31               | 58              | 164               |
| E/MPa      | 20             | 25               | 35              | 10                |
| γ/kN·m⁻³   | 17             | 18               | 19              | 18                |
| d/m        | 2.66           | 8                | 8.5             | 2.47              |
| h/m        | 18.5           | 10               | 19              | 10.7              |

Note: g is the gap parameter, E is the deformation modulus, γ is the unit weight, d is the tunnel diameter, and h is the tunnel depth.

Fig. 5 plots the settlement of ground surface proposed by the three analytical solutions, the field test, and the proposed method. For the Bangkok tunnel, the maximum settlement by the proposed method is 6.67% smaller than that by the field test, and this value is between the results of methods II and III. For the Barcelona tunnel, the maximum settlements of method I and the proposed method decrease by about 45.65% and 17.39% compared with those of the field test. For the Heathrow and Thunder Bay tunnels, the settlement of the ground surface by the proposed method is approximated to three other analytical methods and the field test results. Therefore, the result obtained by the proposed method is basically consistent with those detected by other methods. Its accuracy is verified.

In Figs. 5(c) and 5(d), the settlements predicted by method III are more similar to those obtained by the field test than to those found by the proposed method. However, the analytical solution derived by method III focused on ground displacement, which is unable to present the stress field of the ground as determined by the proposed method. The stress field is a key aspect of rock–liner interaction analysis.

5. Analysis and discussion

5.1. Analysis cases

Eleven analysis cases are carried out to investigate the influences of the gap parameter, the mechanical property of the rock (i.e., deformation modulus and unit weight), the tunnel diameter, and the tunnel depth on the deformation and stress of the rock and liner. Among them, Cases 1 and 3 are the basic conditions; Cases 1–3, Cases 1, 4, and 5, Cases 1, 6, and 7, Cases 3, 8, and 9, and Cases 1, 10, and 11 represent the variations in the gap parameter, the deformation modulus, the unit weight, the tunnel diameter, and the tunnel depth, respectively. Table 2 presents the related parameters of the 11 cases.
Among them, the deformation modulus of the liner is 25 GPa, Possion’s ratio of the liner is 0.3, and the thickness of the liner is 0.1. The ground and liner are assumed elastic. The height and width of the 2D model are 30 and 80 m, respectively. The boundary of the model at the bottom is fixed in all directions, whereas the boundary at the top is free, and the boundaries of the two sides are normally fixed.

Table 2 Geometrical parameters of the tunnel and mechanical property of rock with different analysis cases

| Analysis case | g/mm | E/MPa | y/kN·m$^{-3}$ | $\nu$ | d/m | h/m |
|---------------|------|-------|--------------|------|-----|-----|
| 1             | 10   | 20    | 17           | 0.2  | 2.66| 18.5|
| 2             | 15   | 20    | 17           | 0.2  | 2.66| 18.5|
| 3             | 20   | 20    | 17           | 0.2  | 2.66| 18.5|
| 4             | 10   | 30    | 17           | 0.2  | 2.66| 18.5|
| 5             | 10   | 40    | 17           | 0.2  | 2.66| 18.5|
| 6             | 10   | 20    | 17           | 0.2  | 2.66| 18.5|
| 7             | 10   | 20    | 24           | 0.2  | 2.66| 18.5|
| 8             | 20   | 20    | 17           | 0.2  | 3.66| 18.5|
| 9             | 20   | 20    | 17           | 0.2  | 4.66| 18.5|
| 10            | 10   | 10    | 17           | 0.2  | 2.66| 12.5|
| 11            | 10   | 20    | 17           | 0.2  | 2.66| 24.5|

Note: $\nu$ is Possion’s ratio

5.2. Influence of gap parameter on rock–liner interaction

Fig. 6 plots the initial and final locations of the rock under three different gap parameters. The initial location corresponds to the time when the liner is just installed, whereas the final location corresponds to the moment when the liner stops interacting with the rock at the final state. Thus, the liner uplifts continuously as it starts to be installed. As the gap parameter increases, the uplift becomes great. For example, at $g$ of 10, 15, and 20 mm, the liner uplifts at the vault are 7.33, 9.01, and 10.90 mm, respectively. At the initial location, the rock stress at the invert is greater than that at the vault. Consequently, an uplift force develops because of the stress relief, leading to the buoyancy effect of the tunnel structure at the final stage. A larger $g$ causes a higher uplift force; thus, the buoyancy effect of the tunnel is greater.

![Fig. 6 Initial and final locations of the rock under three different gap parameters](image)

Fig. 7 plots the minimum principal stress under the three gap parameters. The minimum principal stress is directed approximately to the center of the tunnel, so the minimum principal stress can be regarded as the rock stress added to the liner. In Fig. 7, the minimum and maximum values of the rock stress occur at the invert and vault, respectively. This finding is attributed to the presence of the smallest rock displacement that needs to be fixed at the invert; conversely, the rock displacement at the vault is the largest. Furthermore, as the gap parameter increases, the rock stress decreases.
Fig. 7 Minimum principal stress under three different gap parameters

(a) \(g = 10\) mm, (b) \(g = 15\) mm, and (c) \(g = 20\) mm

Fig. 7 Minimum principal stress under three different gap parameters

Fig. 8 plots the axial force of the liner under different gap parameters. The minimum and maximum values of axial force occur at the vault and invert, respectively. The axial force decreases as the gap parameter increases because the axial force is correlated with rock stress. The larger the gap parameter is, the smaller the rock stress will be; as a result, the axial force of the liner is low.

(a) \(g = 10\) mm, (b) \(g = 15\) mm, and (c) \(g = 20\) mm

Fig. 8 Distribution of the axial force of liners with different gap parameters

Fig. 9 illustrates the bending moment of the liner under different gap parameters. The liner mainly suffers from the axial force, and the bending moment is relatively small. The distribution of the bending moment is basically equivalent, which is correlated with the shape of the liner at the final stage. In this case, the shape of the liner resembles a lateral ellipse; as a result, the bending moment varies at different locations.

(a) \(g = 10\) mm, (b) \(g = 15\) mm, and (c) \(g = 20\) mm

Fig. 9 Distribution of bending moment of liner

5.3. Influence of rock property on rock–liner interaction

5.3.1. Deformation modulus of the rock

Fig. 10 shows the initial and final locations of rocks under three different deformation moduli. When \(E\) is 20 MPa, the difference in rock stress from the vault to the invert is the minimum, whereas the buoyancy effect of the tunnel is the most obvious because the liner uplift is affected by the ratio of the difference in rock stress to the deformation modulus. As the deformation modulus is relatively small, the ratio of the difference in rock stress to the deformation modulus is still large although the difference in rock stress is low; consequently, a relatively great liner uplift occurs.
Fig. 10 Initial and final locations of the rock under three different deformation moduli

(a) $E = 20$ MPa, (b) $E = 30$ MPa, and (c) $E = 40$ MPa

Fig. 11 Minimum principal stress under three different deformation moduli

5.3.2 Unit weight of rock

Fig. 12 shows the initial and final locations of the rock under the three unit weights of rock. The larger the unit weight of the rock is, the greater the difference in the rock stress from the invert to the vault will be; thus, the liner uplift is more evident.

Fig. 12 Initial and final locations of the rock under three different unit weights

Figs. 13 and 14 illustrate the axial force of the liner and the minimum principal stress of the rock. The rock stress and the liner force grow as the unit weight increases. The minimum principal stress grows by about 3.46 and 2.79 times at the vault and the invert when $\gamma = 24$ kN·m$^{-3}$ compared with that when $\gamma = 10$ kN·m$^{-3}$, respectively. The axial force of the liner increases by about 3.34 and 2.85 times.

The influence of the unit weight on the liner force and rock stress is relatively more remarkable than that of other parameters. Thus, when the unit weight is large, the liner is prone to failures, which should be carefully considered in the support design.
(a) $\gamma = 10 \text{kN} \cdot \text{m}^{-3}$, (b) $\gamma = 17 \text{kN} \cdot \text{m}^{-3}$, and (c) $\gamma = 24 \text{kN} \cdot \text{m}^{-3}$

Fig. 13 Minimum principal stress under three different unit weights

(a) $\gamma = 10 \text{kN} \cdot \text{m}^{-3}$, (b) $\gamma = 17 \text{kN} \cdot \text{m}^{-3}$, and (c) $\gamma = 24 \text{kN} \cdot \text{m}^{-3}$

Fig. 14 Axial force of the liner under three different unit weights

5.4. Influence of tunnel diameter on rock–liner interaction

Fig. 15 plots the initial and final locations of the rock under the three shield diameters. The rock deformation at the vault increases by about 28.76% and 67.40% when $d = 3.66$ and $4.66$ m compared with that when $d = 2.66$ m. The larger the shield diameter is, the more remarkable the liner uplift will be. Sensitivity analysis revealed that the effect of the shield diameter on the liner uplift is the most evident among the parameters. Therefore, the buoyancy effect during the construction of a shield with a large diameter is universal. This finding is consistent with previous results [24].

(a) $d = 2.66$ m, (b) $d = 3.66$ m, and (c) $d = 4.66$ m

Fig. 15 Initial and final locations of the rock under the three shield diameters

Fig. 16 presents the distribution of the axial force of the liner under different shield diameters. In the 11 analyzed cases, the axial force of the liner at $d = 4.66$ m is the maximum. This finding implies that a large shield tunnel is more susceptible to liner failure. Therefore, in liner installation, the liner should be reinforced to ensure construction safety.

(a) $d = 2.66$ m, (b) $d = 3.66$ m and (c) $d = 4.66$ m

Fig. 16 Distribution of the axial force of the liner under different shield diameters
Fig. 17 presents the distribution of the bending moment of the liner under different shield diameters. The distribution of the bending moment at \( d = 4.66 \) m is different from that in other cases. This difference is correlated with the shape of the liner at the final state. As the shield diameter increases, the liner changes from a lateral ellipse to a vertical ellipse.

![Fig. 17 Distribution of the bending moment of the liner under different shield diameters](image)

5.5. Influence of tunnel depth on rock–liner interaction

Fig. 18 presents the initial and final locations of the rock under the three tunnel depths. Although the difference in the rock stress from the invert to the vault is minimum when \( h = 12.5 \) m, its buoyancy effect is the most remarkable. This phenomenon is influenced by the boundary effect. When the distance of the tunnel to the lower boundary of the model is greater, the ground displacement around the tunnel becomes larger, thereby yielding a higher liner uplift.

![Fig. 18 Initial and final locations of the rock under the three tunnel depths](image)

In Figs. 19 and 20, in comparison with \( h = 12.5 \), as \( h \) increases to \( 18.5 \) m, the minimum principal stress increases by about 69.23% and 55.56%, and the axial force increases by about 68.77% and 55.96%. As \( h \) increases to \( 24.5 \) m, the minimum principal stress increases by about 146.15% and 116.68%, and the axial force increases by about 136.47% and 111.78%. Therefore, the greater the depth is, the larger the rock stress and liner force will be. Sensitive analysis reveals that the influence of the tunnel depth on the rock–liner interaction is lower than that of the unit weight of the ground. In terms of the safety problem of tunnel engineering, tunnel depth is a fairly important factor.

![Fig. 19 Minimum principal stress under three tunnel depths](image)
6. Conclusions
In this study, a calculation method of the rock–liner interaction of a shallow shield tunnel is proposed. Through this method, the stress and deformation distributions of the rock and the liner stress with the influence of the gap parameter, rock property, tunnel diameter, and depth are discussed. Our results provide a theoretical basis for designing a support for shallow shield tunnels. The following conclusions are drawn:

(1) Tunnel uplift is affected by the difference in relieving stress from the invert to the vault. As the gap parameter, the unit weight and the shield diameter are great, the relieving stress is large, and the tunnel uplift is remarkable. However, the analysis of the deformation modulus of the rock and the tunnel depth indicates that the uplift effect decreases as the difference in relieving stress increases. The uplift is more affected by the ratio of the difference in relieving stress to the deformation modulus. The latter mainly focuses on the distance of the tunnel to the lower boundary of the model.

(2) Through the sensitive analysis of several parameters, the effect of the shield diameter on the liner uplift is the greatest among other parameters. This finding confirms the universality of the buoyancy effect during the construction of the shield with a large diameter. Furthermore, the large shield tunnel is more apt to face uplift with the liner failure. In installation, the liner should be reinforced to ensure the construction safety.

(3) Liner force and rock stress are largely affected by the relieving stress. The larger the relieving stress is, the greater the liner force and rock stress will be. The effect of unit weight on the relieving stress is the most evident, the tunnel depth is smaller, and the deformation modulus of the rock is the minimum.

(4) The distribution of the bending moment of the liner is related to its shape at the final stage. The effect of tunnel diameter on the liner is quite obvious. Hence, the distribution of the bending moment changes more remarkably because of a large shield diameter than that of a relatively small diameter.

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