Research Article

Interfacial Behaviour of Shield Tunnel Segment Strengthened by Thin Plate at Inner Surface

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Received 21 March 2022; Accepted 18 July 2022; Published 10 August 2022

Academic Editor: Pasquale Gallo

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The strength and stiffness of a shield tunnel segment can be improved significantly by bonding a steel plate at its inner surface. In this kind of strengthened segment, interface debonding is usually the controlling failure mode, and it strongly depends on the interfacial stresses of the adhesive layer between the segment and the steel plate. To deepen the understanding of the interfacial behaviour, this study proposes a three-dimensional fine finite element (FE) model regarding the interfacial stresses. An existing full-scale experimental result is then employed to confirm the feasibility and accuracy of the proposed model. Further, the fine finite element model is used to calculate the interfacial stress distributions and to evaluate the structural parameters on the interfacial behaviour of the strengthened segment. A high concentration of interfacial stresses exists at the vicinity of the steel plate ends and the joints, which might result in premature failure at these locations. Both the normal and shear stresses at the interface are significantly influenced by the structural parameters. The findings in this study can provide guidance for the optimal design of strengthened shield segment that can prevent premature interfacial debonding.

1. Introduction

With the rapid development of shield tunnelling technology in recent years, segment lining has become the main structure of the urban rail transit system in China. However, with an increase in service life, structural problems are being revealed, namely, large deformation, cracks, leakage, and dislocation and opening of the joints. These problems result in the decrease in lining strength and rigidity [1–5]. Bonding thin plate to the inner surface of the segment is an effective treatment to alleviate such problems and has been widely applied in field [5–11]. However, debonding of the thin plate from the segment is often the governing failure mode of this treatment [5, 8]. As a result, the bearing capacity of the thin plate is underutilized, and the desired reinforcement effect is not achieved. Debonding depends strongly on the interfacial stresses in the adhesive layer between the segment and the thin plate. Consequently, it is essential to investigate the interfacial stresses for understanding the debonding failure and proposing anti-debonding measures.

In recent decades, interfacial stresses of concrete or metallic members reinforced by bonding a thin plate have been extensively investigated. Most of the existing studies on interfacial stresses are analytical and rely on two theoretical methods, namely, stress-based criterion [12–17] and fracture-based criterion [18–21]. All these analytical solutions were primarily developed based on the linear elastic theory. Moreover, these models include too many parameters and it is difficult to consider the parametric effects. The numerical method is another powerful tool for the analysis of interfacial stresses in such strengthened structures. By performing a finite element investigation, Teng et al. [22] proposed appropriate finite element meshes for the accurate determination of interfacial stresses and discussed the singularities of interfacial stresses. In the study conducted by Zhang and Teng [23], five different finite element modelling approaches were described regarding the plate-strengthened beams, and the adhesive layer was modelled by either spring elements or plane stress elements. In the finite element approach proposed by Teng et al. [24], a mixed-mode
cohesive law was employed to describe the interfacial behaviour under a combination of normal stresses and shear stresses. A quadratic strength criterion and a linear fracture energy-based criterion were used to define the damage initiation and the damage evolution, respectively. The same method was employed to determine the interfacial stresses of notched steel beams strengthened with a CFRP plate [25]. Most of the existing studies on interfacial stresses have dealt with straight beams, and relatively less attention has been devoted to curved beams [26].

Several numerical investigations about the shield lining strengthened by a thin plate were conducted, and the interface between the segment and the thin plate was modelled by a tie contact [27], zero-thickness contact elements [28], or friction springs [29]. The focus of these studies was the structural behaviour of the shield lining after it was strengthened by the thin plate, i.e., bearing capacity and deformation, rather than the interfacial behaviour. Overall, scarce information is available on the interfacial failure behaviour of the shield tunnel strengthened by bonding a thin plate at the inner surface. Liu et al. [10] proposed an analytical solution for the interfacial stresses between the segment and the thin plate, which markedly contributes to understanding the interfacial behaviour. However, this analytical solution does not consider the effect of segment joints on the interfacial stresses and is also limited to the elastic analysis. As an important component of the shield tunnel, the effect of joints on the interfacial stresses requires further investigation. As a direct and flexible approach that can cater for any geometric, sectional, loading, and boundary conditions, the finite element method can be utilized for investigating the interfacial stresses by considering the effect of joints.

The main objectives of this study were to clarify the distribution of the interfacial stresses and to deepen the understanding of the interface failure of a shield tunnel lining strengthened by a thin plate at the inner surface. This study is organized as follows to achieve the objectives. Firstly, a three-dimensional finite element model of a shield lining is prepared by considering segment joints and bolts. The model is calibrated with attention to appropriate meshing and bond behaviour modelling for the accurate prediction of interfacial stresses between the segment and the thin plate. Secondly, the proposed model is validated against a full-scale experiment of a segment strengthened by a steel plate. Finally, the proposed numerical model is used to investigate the interfacial stresses by varying selected parameters, namely, reinforcement range, preload (the load acting on the segment when it was strengthened), thickness of the steel plate and the adhesive layer, and elastic moduli of the thin plate and the adhesive layer.

2. Finite Element Modelling

2.1. Structure Information. Finite element software ABAQUS™ was adopted to conduct the numerical analysis. The typical shield tunnel in Shanghai Metro was studied in this study. The outer diameter of the lining $D_{out}$ was 6.2 m, the thickness $t$ was 0.35 m, and the length of the ring was 1.2 m. As shown in Figure 1, an integrated ring was composed of one key segment $F$ (16°), two adjacent segments $L_1$ and $L_2$ (65° each), two standard segments $B_1$ and $B_2$ (65° each), and one bottom segment $D$ (84°). To connect the six segments, one lining ring had six longitudinal joints, and each longitudinal joint was connected by two straight M30 steel bolts of 5.8 grade, with a total of 12 longitudinal bolts in one lining ring. The bolt length was 400 mm, and the diameter of the bolt hole was 42 mm. The bolt hole was located at a distance of $1/3$ from the inside of the lining.

In practice, a steel plate, an FRP sheet, and FRP-like products such as textile-reinforced concrete and FRP grid can be used as the material of the thin plate. A steel plate is most adopted because it can significantly improve the strength and the stiffness of the segment. Therefore, a steel plate was employed in this study. The steel plate was pasted to the inner surface of the segment by epoxy resin. The thickness of the steel plate and epoxy resin was assumed to be 20 mm and 2 mm, respectively. The steel plate was employed in the range of $-30° \sim 30°$ along the inner surface of the lining, where the tunnel crown was set to 0°.

2.2. Element Type and Contact Relationship. In the ABAQUS element library, there are three degrees of freedom per node of the C3D8R element along three axes, respectively (eight-node linear hexahedron element, reduction integral, and hourglass control element). This element can be used for the analysis of three-dimensional stress and large strain, which provides a reliable solution to most applications including the investigation of the structural deformation and interfacial stresses in this study. To maintain the consistency of the axial and flexural stiffness, 3D brick elements were chosen in the numerical model to simulate the segment, bolts, and thin plate to facilitate model convergence. Based

![Figure 1: Typical cross section of shield lining in Shanghai.](image-url)
on the research conducted by Teng et al. [22], the adhesive layer was simulated by solid element, that is, C3D8R element too.

The interactions between the components were modelled by setting contact relationships. Tie and cohesive law are two methods commonly used for modelling the interface between the strengthened structure and the thin plate for different purposes. To be more specific, when the distribution of the interfacial stresses under different structural parameters needs to be clarified [22, 23], the interface is assumed to be perfectly bonded and the tie contact is adopted. The cohesive law is employed when the investigation is focused on the initiation and evolution of debonding [24, 25]. To analyse the interfacial stresses under different reinforcing parameters in this study, the segment and the thin plate were assumed to maintain strain compatibility. Therefore, the segment/adhesive interface and adhesive/thin plate interface were modelled as tie contacts. The longitudinal joint was connected by two straight M30 steel bolts of 5.8 grade, and friction contact was defined between adjacent segments. To be more specific, the penalty method available in the ABAQUS was utilized for the formulation of the tangential behaviour at the friction contact with a friction coefficient of 0.4. Hard contact was adopted to define the normal behaviour, such that the surfaces could separate under tensile force but could not penetrate each other under compressive force.

2.3. Constitutive Model and Parameters. To accurately predict the interfacial stresses in the adhesive layer, the segment, the steel bolts, and the thin plate must be modelled properly. The stress-strain diagram of concrete adopted in this study is presented in Figure 2(a), which is similar to the concrete model proposed by Park and Paulay [30] and has been widely adopted for the FE simulations of shield tunnel segments [29]. In compression, the stress and strain responses follow a parabolic stress-strain curve up to the compressive strength $\sigma_c$, and then, the stress decays linearly with strain until ultimate strength $\sigma_{cu}$ is reached. Unloading is characterized by the initial stiffness followed by a degraded slope. In tension, a linear stress-strain behaviour is assumed until the tensile strength $\sigma_t$ is reached. The values of key points of the stress-strain curve of the concrete are given in Figure 2(a). For both the steel bolts and the steel plate, a bilinear constitutive model was adopted in the numerical simulation [31], as shown in Figure 2(b). The adhesive layer was made of epoxy resin, and consequently, a linear elastic model was adopted for the adhesive layer. Young’s modulus and Poisson’s ratio were 3 GPa and 0.35, respectively. The material properties of the concrete, bolts, and steel plate used in this study are illustrated in Figure 2.

2.4. Loads and Boundary Conditions. Ignoring the influence of adjacent rings, only one ring was considered. In addition, the geometry and loading conditions of the tunnel were symmetrical about the longitudinal axis, and therefore, only a half ring was simulated in the numerical analysis. Figure 3 presents the demonstration of the loads applied to the tunnel. In the figure, $p_v$ is the vertical earth load at the top of the lining, $p_l$ and $p_t$ are the lateral earth load at the depth of the tunnel crown and tunnel invert, respectively, and the load between $p_l$ and $p_t$ is distributed linearly. $p_r$ is the reaction load at the bottom of the tunnel. Calculation of those loads acting on the lining can be referenced by Zhang et al. [32] and Liu et al. [10]. Besides, nonlinear ground springs were employed in the range of $225^\circ \sim 315^\circ$ along the tunnel circumference to simulate the soil resistance load caused by the transverse deformation of the lining. The soil spring constant was set to 15000 kN/m$^3$. The three-dimensional FE model was a 3D plane strain model, and out-of-plane degrees of freedom were restrained. Due to the symmetry of the structure and external loads, the nodes of the vault and invert surface did not experience normal displacement.

2.5. Convergence Study. Different mesh sizes were tried to find a reasonable mesh that provides both accurate results and less computational time. A particularly fine finite element mesh was adopted near the steel plate ends and longitudinal joints. More specifically, in the radial direction, the steel plate and the segment were meshed with the offset mesh technology, and the minimum sizes were 1.0 mm and 2.0 mm for the steel plate and the segment, respectively. The thickness of the adhesive layer was taken as 2 mm. In this study, the number of elements used across the adhesive layer (at the thickness direction or radial direction) was 2, 4, 8, and 16, corresponding to four different minimum element heights of 1.0 mm, 0.5 mm, 0.25 mm, and 0.125 mm, respectively. In the circumferential direction, a graded mesh was used starting with an aspect ratio of 1 for the minimum height elements, and the final aspect ratio of the graded mesh was 8. The same pattern was employed for the steel plate. A graded mesh of matching fineness was also adopted for the segment. The finest mesh is presented in Figure 4, where the number of elements used in the thickness direction of the steel plate, adhesive layer, and lining is 9, 8, and 34, respectively.

Normally, there are three types of interface failure of such strengthened structure, namely, within the adhesive layer, at concrete/adhesive interface, and at steel/adhesive interface. Of these three adhesion failure modes, the latter two can be avoided in practice through the proper preparation of the concrete surface and the steel surface, respectively. It should be noted that Teng et al. [22] presented an FE investigation of interfacial stresses across the adhesive layer, with attention to the interfacial stresses located at the adhesive-to-concrete interface, the mid-adhesive interface, and the plate-to-adhesive interface. The results showed that stress singularity occurred at the adhesive-to-concrete interface and at the plate-to-adhesive interface, while the interfacial stresses at the mid-adhesive interface had good natured behaviour. Consequently, in this study, the interfacial stresses were obtained by a horizontal cut at the mid-thickness of the adhesive layer.
A reinforcement range of 60° (symmetric reinforcement along the crown of the lining ring, i.e., −30° ~ 30°) was employed, of which only the 8° joint and the 352° joint lay within the reinforcement range. Figure 5 shows the variations of the interfacial normal and shear stresses as the height of the smallest element of the adhesive layer was decreased from 1.0 mm to 0.125 mm in four steps, corresponding to the number of elements across the adhesive layer as 2, 4, 8, and 16, respectively. Notable peaks were observed for both the normal and shear stresses near the edges of the joints and the steel plate ends, and they decayed rapidly away from the joints and steel plate ends. Both the maximum normal stress and the maximum shear stress near the steel plate ends and the joints increased with mesh refinement, but the difference between the successive values was decreasing (Figure 5(a)–5(d)). A similar pattern was also observed in the research conducted by Teng et al. [22] and Elmalich and Rabinovitch [33]. As the element size decreased, the normal stress at the steel plate ends converged to a constant value of approximately 1.25 MPa (Figure 5(a)), while the shear stress approached zero (Figure 5(b)), matching the zero-shear stress condition of the free end. The interfacial stresses at the joints also generally converged to a fixed value when the element size was smaller than 0.25 mm.
(Figures 5(c) and 5(d)). The differences between the stresses produced by the two finest meshes used, namely, 0.25 mm and 0.125 mm element sizes, were strongly localized and could be ignored. Consequently, the minimum element size of 0.25 mm was used in the following sections by also considering the computational efficiency.

3. Verification of the Model

3.1. Experiment Overview. Experimental results of a full-scale test for a steel plate reinforced segment conducted by Liu et al. [5] were used as baselines to verify the proposed numerical model. The typical shield tunnel structure of Shanghai Metro, which is the same as shown in Figure 1, was employed as the test specimen. The concrete grade of the segments was C55. The yield limit and ultimate strength of the bolts used to connect the segments were 400 MPa and 500 MPa, respectively. The segment was strengthened by a steel plate at its inner surface along the whole ring. The thickness and the width of the steel plate ring were 20 mm and 850 mm, respectively. The steel plate ring was fixed along the segment circumferential midline with plug bolts, and the gap between the steel plate ring and the segment was filled with epoxy resin. The yield limit and ultimate strength of the steel plate were 340 MPa and 420 MPa, respectively. As shown in Figure 6, hydraulic jacks, comprising \( P_1 \) (6 loading points), \( P_2 \) (10 loading points), and \( P_3 \) (8 loading points), were used to apply pressure at the outer surface of the segment. All lines of action of the loads meet at the centre of the steel ring, as follows from Figure 6(a). 150 round balls were placed at the bottom of the specimen to satisfy the condition of rolling support, characterized by low friction between the specimen and the supporting steel panel. The experiment process included the following 4 stages: first, \( P_2 \) was increased from 0 kN to the value of the passive earth pressure, i.e., 275 kN, meanwhile \( P_1 = 1.54 \times P_2 \) and \( P_3 = 0.5 \times (P_1 + P_2) \). Following that, loading was continued until \( P_1 \) reached the value for the segment before strengthening. \( P_2 \) was kept constant at 275 kN, and \( P_3 \) was obtained as \( P_3 = 0.5 \times (P_1 + P_2) \). Then, the steel plate was installed. Finally, loading was further continued until the segment failed, while \( P_2 = 275 \text{ kN} \) and \( P_3 = 0.5 \times (P_1 + P_2) \) were maintained continuously. More detailed information about the test can be obtained from a previous report [5].

3.2. 3D Fine Finite Element Model of the Experiment. Due to the symmetry of the structure and loads, only half of the specimen was modelled, as shown in Figure 7. Element types, contact relationships among different components, constitutive models and parameters, and meshing methods were the same as those introduced in Section 2. The boundary conditions were consistent with the experiments, which limited the horizontal displacement of the crown and the invert and the vertical displacement of the arch waist and also limited the out-of-plane degrees of freedom. To avoid the difficulty of convergence caused by the stress...
Figure 5: Interfacial stresses under different meshing. (a) Normal stress near plate end. (b) Shear stress near plate end. (c) Normal stresses near 352° joint. (d) Shear stress near 352° joint.

Figure 6: Full-scale experiment [5]. (a) Load distribution. (b) Top view of the experiment.
concentration at the thirteen loading points, “loading surfaces” were added at those locations (Figure 7), where the area of each loading surface was the same as the load block given in Figure 6(b). When the load was applied, the concentrated force of the jack was uniformly distributed to the “loading surface.” The reinforcement point was defined as the load acting on the segment when the vertical convergence (relative displacement between the crown and the invert) reached 120 mm. In the numerical simulation, when the load reached the strengthening point, elements of the adhesive layer and the steel plate were activated to simulate the installation of the steel plate. The vertical displacement contour at the reinforcement point is shown in Figure 8.

3.3. Comparison with the Experimental Results. The numerical results showed that under the load, the lining expanded along the horizontal direction and shrunk along the vertical direction, which agrees with the experimental results. Figure 9 compares the load $P_1$ versus vertical convergence curves obtained from the numerical analysis and the experiment, where the lining deformed inward was positive. At the initial stage, the structure behaved elastically, and with an increase in load, the behaviour gradually became nonlinear. When the segment was strengthened by the steel plate, the overall stiffness of the strengthened segment was improved noticeably. Thereafter, the displacement increased rapidly, while the load could not increase further indicating the failure of the strengthened segment. Similar responses were observed in the experiment. Overall, the load $P_1$ versus vertical convergence curve obtained from the numerical analysis agreed well with the experiment.

To verify the reliability and feasibility of the FE model, this study compared the differences in the strengthening point and the ultimate capacity between the numerical analysis and the experiment. As summarized in Table 1, at the strengthening point, $P_1$ obtained from the numerical analysis and the experiment was 427 kN and 418 kN, respectively, with a difference of 2.2%. At the ultimate capacity point, $P_1$ from the numerical analysis and the experiment was 672 kN and 616 kN, respectively, and the deviation was 9.1%.

In the experiment, the tensile strength and the shear strength of the epoxy resin, which was used to fill the gap between the steel plate and the lining, were 1.65 MPa and 6.83 MPa, respectively [29]. When the normal stress and the shear stress of the adhesive layer exceeded their corresponding strengths, bonding failure occurred. In the experiment, when the load $P_1$ reached 586 kN, slippage and stripping between the steel plate and the concrete started to occur near the 8° joint, meaning that the shear stress and normal stress exceeded the shear strength and tensile strength, respectively. Under the same load, the normal stress and the shear stress obtained from the numerical analysis were 1.67 MPa and 6.53 MPa, respectively, with the corresponding deviation of 1.21% and -4.39% compared with the values considered in the experiment.

In this way, the finite element model was verified against a full-scale experiment, in terms of both mechanical behaviour and interfacial stresses. Thus, the proposed finite element model has good accuracy to predict the interfacial stresses of a shield tunnel strengthened by a steel plate at inner surface.

4. Parametric Study

A parametric study of the interfacial stresses was performed by utilizing the proposed finite element model. Six different parameters, namely, the arrangement range of the steel plate, the preload of the shield lining, the thickness and elastic modulus of the steel plate, and the adhesive layer were considered. The segment of Figure 1 was taken as the reference lining. Material properties were
the same as those considered in Section 2. In the parametric study, only the parameter to be studied was varied, while the other parameters were kept fixed unless otherwise stated. For the normal stress at the interface, tensile stress was taken as positive, while compressive stress was taken as negative. For the shear stress at the interface, stress tending to rotate the section clockwise was considered positive.

Figure 10 shows the distribution of interfacial stresses under the reinforcement range of 60°. Although only one half of a ring was simulated, to better display the distribution of the interfacial stresses, the interfacial stresses along the whole reinforcement range were depicted by mapping along the longitudinal axis. It can be observed that the peak interfacial stresses appeared near the plate ends and the joints, and the interfacial stresses away from the plate ends and the

The same as those considered in Section 2. In the parametric study, only the parameter to be studied was varied, while the other parameters were kept fixed unless otherwise stated. For the normal stress at the interface, tensile stress was taken as positive, while compressive stress was taken as negative. For the shear stress at the interface, stress tending to rotate the section clockwise was considered positive.
joints rapidly reduced to almost zero. Since the joints were connected by bolts, stiffness discontinuity and displacement catastrophe occurred resulting in the concentration of the interfacial stresses. This observation is consistent with the literature [34–36] that reported the concentration of the interfacial stresses in the vicinity of the cracked sections in reinforced concrete beams. Accordingly, this study focuses on the interfacial stresses near the plate ends and the joints in the following part, except Section 4.1.

4.1. Effect of Reinforcement Range. The reinforcement range of 30°, 45°, 60°, 75°, and 90° was investigated, and only 8° and 352° joints were included in those reinforcement ranges. The distribution of interfacial stresses under different reinforcement ranges is depicted in Figure 11. It shows a large stress concentration near the plate ends and the joints (Figures 11(a) and 11(b)). With an increase in the reinforcement range, the maximum interfacial stresses near the plate ends decreased rapidly. However, the rate of decrease was larger for the shear stress compared with the normal stress (Figure 11(c)). The peak interfacial stresses near the joints had no visible difference, especially for the shear stress. This is attributed to the fact that the internal force of the lining at the joint section was basically the same for different reinforcement ranges. When the reinforcement range was less than a certain value, the interfacial stresses near the plate ends were greater than those near the joints. This value was approximately 45° and 75° for the normal stress and the shear stress, respectively (Figure 11(c)). When the reinforcement range was greater than 85°, the normal stress near the plate ends began to be negative. In other words, the later the reinforcement was applied, the more likely was the interface failure. At the same time, even if there was no interface failure, an increase in preload was less effective in improving the structural stiffness and the ultimate capacity [8]. As a result, the increase in preload would make the bonding failure easier, which would further reduce the effectiveness of the reinforcement. Therefore, if reinforcement must be provided, the sooner the better it will be.

4.2. Effect of Preload. Figure 12 presents the numerical results for the segment reinforced by a steel plate when the value of the vertical convergence was 0 mm, 10 mm, and 20 mm. It can be observed that an increase in preload affected the interfacial stresses, especially near the joint (Figures 12(c) and 12(d)). Overall, the interfacial stresses increased with an increase in preload. However, the rate of increase in the interfacial stresses showed a decreasing trend with an increase in preload. In other words, the later the reinforcement was applied, the more likely was the interface failure. At the same time, even if there was no interface failure, an increase in preload was less effective in improving the structural stiffness and the ultimate capacity [8]. As a result, the increase in preload would make the bonding failure easier, which would further reduce the effectiveness of the reinforcement. Therefore, if reinforcement must be provided, the sooner the better it will be.

4.3. Effect of Thickness of Thin Plate. To study the effect of the thickness of the bonded steel plate, the thickness was varied as 10 mm, 20 mm, or 30 mm. Figure 13 shows the curves of interfacial stresses near the plate end and the joint corresponding to different thicknesses of the steel plate. It can be clearly seen that an increase in the steel plate thickness led to an increase in the interfacial stresses near the plate end (Figures 13(a) and 13(b)). Conversely, an increase in the steel plate thickness resulted in the decrease in the interfacial stresses near the joint (Figures 13(c) and 13(d)). More specifically, when the thickness of the plate increased from 10 mm to 20 mm to 30 mm, the maximum normal stress near the plate end increased from 1.28 MPa to 2.15 MPa to 2.86 MPa, with an increment of 0.87 MPa and 0.71 MPa, respectively. Similarly, the maximum shear stress increased from 1.98 MPa to 2.92 MPa to 3.52 MPa, with an increment of
0.94 MPa and 0.6 MPa, respectively. When it comes to the joint, the maximum normal stress corresponding to the thickness of 10 mm, 20 mm, and 30 mm was 5.05 MPa, 3.05 MPa, and 2.01 MPa, with the successive increment of −1.04 MPa and −0.71 MPa. Similarly, the maximum shear stress was 1.94 MPa, 1.23 MPa, and 0.86 MPa with the successive increment of −0.71 MPa and −0.37 MPa. These results indicate that the steel plate thickness markedly affects the magnitude of the interfacial stresses. However, the degree of effect gradually decreases with the increase in the plate thickness. Consequently, to avoid the interface failure and to achieve economy, the steel plate should be adopted with a variable section, such that the thickness is gradually decreased toward the ends. With the same material consumption, a steel plate with a variable section will be beneficial to enhance the strengthening effect [37].

4.4. Effect of Elastic Modulus of Thin Plate. Interfacial stresses corresponding to thin plates with different values of elastic modulus (10, 50, 100, and 200 GPa) were also investigated. The effect of elastic modulus of the thin plate on the interfacial stresses is given in Figure 14. The peak interfacial stresses were located near the thin plate end and the joint as before. Generally, an increase in elastic modulus of the thin plate caused an increase in the interfacial stresses near the plate end (Figures 14(a) and 14(b)) and a decrease in the interfacial stresses near the joint (Figures 14(c) and 14(d)). The maximum normal stress near the plate end corresponding to four different values of elastic modulus was 0.63 MPa, 1.45 MPa, 1.86 MPa, and 2.15 MPa, respectively, with the successive increase of 0.82 MPa, 0.41 MPa, and 0.29 MPa. Similarly, the normal stress near the joint was 11.04 MPa, 6.98 MPa, 4.93 MPa, and 3.05 MPa, with
successive reduction of 4.06 MPa, 2.05 MPa, and 1.88 MPa. As the elastic modulus increased from 50 GPa to 200 GPa, the maximum shear stress near the plate end increased from 0.35 MPa to 2.92 MPa, while that near the joint decreased from 4.25 MPa to 1.23 MPa. These data indicate that the elastic modulus of the thin plate has a significant effect on the interfacial stresses; i.e., the larger the elastic modulus, the larger are the interfacial stresses near the plate end and the smaller are the interfacial stresses near the joint. However, the effectiveness of an increase in elastic modulus decreased with an increase in the modulus.

4.5. Effect of Thickness of Adhesive Layer. The reasonable thickness of the adhesive layer on the inner surface is suggested to be 2 mm for strengthening the lining [10]. Consequently, three thicknesses, viz., 1.0 mm, 2.0 mm, and 3.0 mm, were selected to study the influence of the thickness of the adhesive layer on the interfacial stresses. The interfacial stresses corresponding to different adhesive layer thicknesses are shown in Figure 15. The thickness of the adhesive layer showed a significant role in the distribution and magnitude of the interfacial stresses. Obviously, the peak interfacial stresses decreased as the thickness of the adhesive layer increased, for both the interfacial stresses near the plate end and the joint. The rate of reduction showed a downward trend. To some extent, this suggests that the capacity of the adhesive interface develops with an increase in the thickness of the adhesive layer.

4.6. Effect of Elastic Modulus of Adhesive Layer. As plotted in Figure 16, peak interfacial stresses near the plate end and the joint were obtained from the numerical simulation with four different elastic modulus values of the adhesive layer (viz., 1, 3, 5, and 7 GPa). As the elastic modulus of the adhesive layer increased, interfacial stresses increased both near the plate end and the joint. Again, the rate of increase showed a
Figure 13: Interfacial stresses under steel plates with different thicknesses. (a) Normal stress near plate end. (b) Shear stress near plate end. (c) Normal stress near 352° joint. (d) Shear stress near 352° joint.

Figure 14: Continued.
Figure 14: Interfacial stresses under steel plates with different elastic modules. (a) Normal stress near plate end. (b) Shear stress near plate end. (c) Normal stress near 352° joint. (d) Shear stress near 352° joint.

Figure 15: Interfacial stresses under adhesive layers with different thicknesses. (a) Normal stress at plate end. (b) Shear stress at plate end. (c) Normal stress at 352° joint. (d) Shear stress at 352° joint.
downward trend. Therefore, to reduce the interfacial stresses and avoid the early debonding failure, the adhesive layer should be selected with relatively smaller elastic modulus.

5. Conclusions

This study presented a finite element investigation on the interfacial stresses of a shield tunnel lining strengthened by a steel plate bonded at its inner surface. A finite element model was proposed and validated against existing experiment results of a segment reinforced by a steel plate. Using the proposed numerical model, the interfacial stresses between the steel plate and the segment corresponding to different strengthening parameters were analysed, focusing on the regions near the plate end and near the joint. Based on the findings from this study, the following conclusions were derived:

(1) The peak interfacial stresses occurred near the plate ends and the joints, and they rapidly decreased to almost zero value away from the plate ends and the joints. The state of the normal interfacial stress was reversed at the joint, such that the adhesive layer on one side of the joint was under tensile-shear state, while the other side was under compressive-shear state.

(2) With an increase in reinforcement range, the interfacial stresses near the plate end decrease sharply, while those near the joint were barely affected. The reinforcement range was expanded from 30° to 90°, the peak value of the normal stress decreased from 4.2 MPa to −0.5 MPa, and the peak value of the shear stress decreased from 5.5 MPa to −1 MPa; the peak value of the normal stress and the shear stress near the joint was relatively small, which were 0.3 MPa and 0.1 MPa, respectively. In contrast, the interfacial stresses near the joint increased significantly and those near the plate end increased slightly due to an increase in preload. The interfacial stress near the joint was positively correlated with the preload, but the increase rate of the interfacial stress decreased with the increase in the preload.

(3) The interfacial stresses near the plate end were larger than those near the joint when the reinforcement

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Figure 16: Interfacial stresses under adhesive layers with different elastic modules. (a) Normal stress near plate end. (b) Shear stress near plate end. (c) Normal stress near 352° joint. (d) Shear stress near 352° joint.
range was less than a certain value, and for the normal stress and shear stress, these certain values were about 45° and 75°, respectively. The plate end was the key part to prevent debonding failure. When the reinforcement range exceeded these certain values, the interfacial stresses near the plate end were less than those near the joint. Therefore, attention should be paid to the joint to avoid the interface failure.

(4) The thickness and the elastic modulus of the steel plate had an appreciable effect on the interfacial stresses. The interfacial stress near the plate ends was positively correlated with the thickness and elastic modulus of the steel plate, while the increase in thickness and elastic modulus of the steel plate results in a negative correlation with the interfacial stress near the joint.

(5) As the thickness of the adhesive layer increased, the interfacial stresses both near the plate end and the joint decreased and the reduction rate showed a downward trend. With an increase in the elastic modulus of the adhesive layer, the interfacial stresses increased both near the plate end and the joint, but the growth rate was on a downward trend. Therefore, under the premise of meeting the requirements of bonding performance, selecting an adhesive with a relatively small elastic modulus could avoid the early interface debonding failure of the inner surface reinforcement structure of the shield tunnel.

Data Availability

The data that support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this study.

Acknowledgments

This study was substantially supported by the National Natural Science Foundation of China (nos. 51878658, 51808359, and 51978425), Joint Fund of State Key Laboratory of Coal Resources and Safe Mining-the Beijing Outstanding Young Scientist Program (nos. SKLCRSM20LH03 and BJWZYJH01201911413037), Yueqi Laboratory of Coal Resources and Safe Mining-the Beijing Natural Science Foundation of China (nos. 51878658, 51878659, and 51978425), Joint Fund of State Key Laboratory of Coal Resources and Safe Mining-the Beijing Outstanding Young Scientist Program, and the Fundamental Research Funds for the Central Universities (2022JCCXLJ05).

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