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Research Article

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Numerical study on the failure characteristics and reinforcement effects of shield segmental linings subjected to localized overload

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Abstract: In typical shield tunnel construction projects, tunnels are several kilometers long and overloading will not occur in the full-line of the shield tunnel structures. When only a portion of the shield lining structures in a full-line tunnel are overloaded, their bearing and failure characteristics are significantly different from those in the full-line overloaded case. In existing studies, one or several segmental lining rings have been studied, with overload applied to selected lining rings to analyze the performance evolution of the lining structures; however, this approach fails to reveal the bearing and failure characteristics of shield lining rings under localized overload. To address this research gap, we employ three-dimensional finite element modeling to investigate the mechanical performance and failure mechanisms of shield segmental linings under localized overload conditions, and compare the results with full-line overload scenarios. Additionally, the impact of reinforcing shield segmental linings with steel rings (both within individual lining rings and spanning across multiple rings) is studied to address issues arising from localized overloads, which are more common in engineering practice. The results indicate that localized overloads lead to significant joint dislocation and higher stress on longitudinal bolts, potentially causing bolt failure, while the overall deformation of lining rings, segmental joint opening, and stress in circumferential bolts and steel bars are lower compared to full-line overloads. For the same overload level, the convergence deformation of lining under full-line overload is 1.5 to 2 times higher than under localized overload. For localized overload situations, a reinforcement scheme with steel rings spanning across two adjacent lining rings is more effective than installing steel rings within individual lining rings. This spanning ring reinforcement strategy not only enhances the structural rigidity of each ring, but also limits joint dislocation and reduces stress on longitudinal bolts, with the reduction in maximum ring joint dislocation ranging from 70% to 82% and the reduction in maximum longitudinal bolt stress ranging from 19% to 57% compared to reinforcement within rings.

Under review

Key words: Shield segmental lining; Localized overload; Failure characteristics; Steel ring reinforcement

1 Introduction

The shield tunneling technique has been extensively applied to construct highway, railway, and urban subway tunnels in soft soil strata. The approach is favored for its adaptability to challenging ground conditions and minimal impact on the surface environment (Hu et al., 2022; Elbady et al., 2022; Ran et al., 2023). For shield segmental linings assembled with precast reinforced concrete segments and bolts, there are many joints in the lining structures, and the existence of joints presents challenges in terms of stress and deformation of the lining structures (Pabodha et al., 2022; Wei et al., 2024; Lu et al., 2024). During the operational period, the changing of surrounding loads can cause excessive convergence deformation in shield lining structures, which can lead to issues such as joint opening and dislocation, water leakage, and cracking of concrete around the joints (Wang et al., 2023; Sharghi et al., 2023; Ye et al., 2024; Zheng et al., 2024). Therefore, many researchers have investigated the mechanical behavior of shield segmental linings in response to changes in surrounding loads.

For instance, Liu et al. (2016, 2017) conducted full-scale tests to analyze the bearing capacity and failure mechanisms of continuous-jointed and stag-

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ger-jointed segmental linings, with the results showing that segmental lining structures were more vulnerable to lateral unloading than to overload conditions. Zhou et al. (2022) conducted a full-scale experiment for a shield tunnel bearing inner water pressure, and proposed that the damage characteristics of the shield lining under the lateral unloading condition are the waterproof failure of the segmental and ring joints, as well as concrete cracking failure. Also, Chen et al. (2020), Wu et al. (2022, 2023), and Gao et al. (2024) analyzed the structural responses and failure processes of shield lining structures under upper overload and unloading conditions using refined 3D finite element models. In their research, the lining deformation and stiffness, internal forces (bending moment and axial force) of the lining rings, joint deformation and waterproof performance, and development of concrete cracks were studied. As such, four stages of change in the lining rings were revealed (the linear growth stage, quasi-linear growth stage, nonlinear growth stage, and failure stage). In addition, Su et al. (2022) investigated the internal force distribution and transverse deformation characteristics of a shield tunnel lining subjected to local soil loosening using numerical modeling.

Experimental and numerical studies have provided valuable insights into the structural responses and failure mechanisms of shield lining rings under various loading conditions, revealing the progressive stages of behavior and the importance of joint integrity. However, existing research has often focused on the mechanical behavior and failure mechanisms of isolated or multiple lining rings under changing load scenarios, which may not fully represent the behavior of an entire tunnel under localized load changes. Applying such a method is equivalent to a subsequent load when changes in the surrounding environment are applied to the full-line of a shield tunnel, which is different from actual engineering situations. In practice, shield tunnels can span several kilometers, and it is unlikely that a uniform overload would occur across their entire length. The behavior of a portion of the lining under localized overload differs significantly from a full-line overload scenario (Arnau and Molins, 2015). Additionally, previous studies have been limited by using shell elements and shell interface elements to represent segments and joints, which do not account for detailed three-dimensional structural characteristics, making it impossible to accurately describe the spatial structural characteristics of the shield segmental linings (Arnau and Molins, 2015).

This study aims to overcome existing limitations by developing refined three-dimensional finite element (FE) models. These models will enable analysis of the mechanical behavior and failure mechanisms of shield segmental linings subjected to localized overload conditions. Additionally, we incorporate practical reinforcement methods that are crucial for bolstering the structural integrity of shield tunnels. The standard approach for mitigating large deformation in shield tunnel linings involves affixing steel rings to the inner surfaces of the lining segments (Liu et al., 2022; Li et al., 2023, 2024; Zhai et al., 2023). We employ two distinct reinforcement strategies: embedding steel rings within each lining ring, and configuring steel rings to span multiple adjacent lining rings (Liu et al., 2020, 2023). By comprehensively evaluating the impact of reinforcement, an optimal reinforcement strategy for scenarios characterized by localized overload is formulated.

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2 Finite element model and validation

2.1 Establishing the finite element model

The numerical simulation focuses on shield lining structures within the Jinan Metro R2 Line, as illustrated in Fig. 1. The segmental lining of the Jinan Metro R2 Line is defined by its precise geometric specifications: an outer diameter of 6.4 meters, an inner diameter of 5.8 meters, a uniform thickness of 0.3 meters, and a width of 1.2 meters. Each ring of the lining structure is comprised of a single key segment (referred to as ‘K’), accompanied by two adjacent segments (denoted as ‘L1’ and ‘L2’), and complemented with three standard segments (labeled ‘A1’, ‘A2’, and ‘A3’). The key segment is characterized by a central angle of 22.5 degrees, in contrast to the adjacent and standard segments, which each possess a central angle of 67.5 degrees. The assembly of the lining rings is executed in a staggered-jointed fashion, with a staggered angle of 45 degrees, as depicted in Fig. 1.

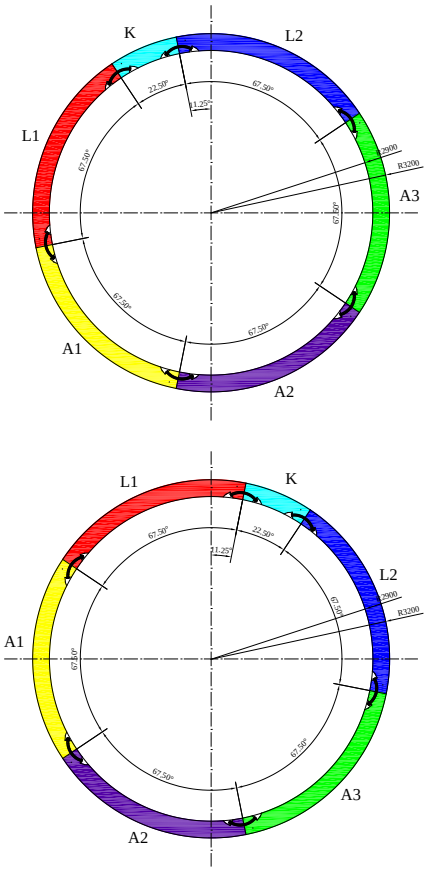

Fig. 1 Shield segmental lining diagram (Unit: mm)

The ABAQUS software was used for the numerical simulation in this study. The concrete utilized in the lining structures is grade C50, with an elastic modulus of 34.5 GPa and a Poisson's ratio of 0.2. The concrete behavior is modeled using a plastic damage model, with specified tensile and compressive strengths of 2.64 MPa and 32.4 MPa, respectively. Fig. 2 delineates the stress-strain relationship, as well as the strain and damage parameters of the concrete material. The simulation of concrete segments employs eight-node linear solid elements (type C3D8R). In the case of steel bars in the segments, the simulation considers only the main steel bars along the circular lining rings, with the transverse stirrups being excluded from the analysis. The strength grade of the main steel bars is HPB400 with a yield strength of 400 MPa, an elastic modulus of 210 GPa, and a Poisson's ratio of 0.3. Fig. 3a shows the relationship between stress and strain for the HPB400 steel bars.

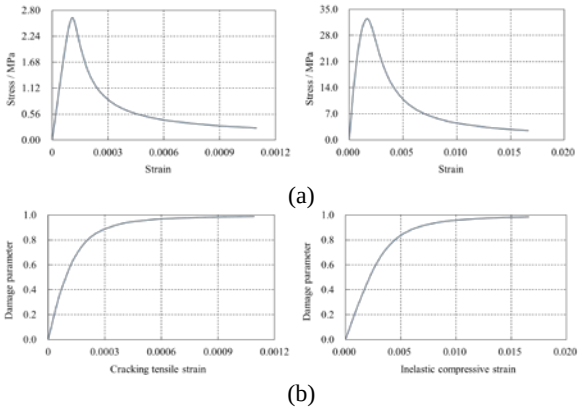

Fig. 2 Concrete plastic damage model (left: tension, right: compression): (a) stress-strain relationship; (b) damage parameter

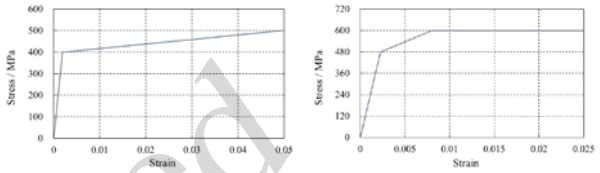

Fig. 3 Stress-strain relationship of steel bars and bolts: (a) steel bar; (b) bolt

The main longitudinal reinforcements are embedded within the concrete model, with their simulation facilitated through two-node truss elements (type T3D2). Two circumferential curved bolts with Grade 6.8 and a dimeter of 27 mm are applied to connect the segmental joint between the two adjacent segments. Sixteen longitudinal curved bolts with Grade 6.8 and a dimeter of 27 mm are used to connect the two adjoining lining rings, with each bolt positioned at an equal angular interval of 22.5°. The yield and ultimate strengths of the Grade 6.8 circumferential and longitudinal bolts are 480 MPa and 600 MPa, respectively. The elastic modulus and Poisson's ratio of the bolts are 210 GPa and 0.3, respectively. Fig. 3b shows the relationship between stress and strain of the bolts. For the simulation of the embedded bolts within the concrete model, a solid element of C3D8R is utilized. In constructing the three-dimensional FE models for the tunnel lining structures, the water-stop grooves at the segmental and ring joints are intentionally excluded. The ethylene propylene diene monomer (EPDM) rubber water-stop belts are also omitted from the models. The contact between the segments in the segmental and ring joint positions is realized with the surface-to-surface function in the ABAQUS software, with the parameters set to represent a hard contact in

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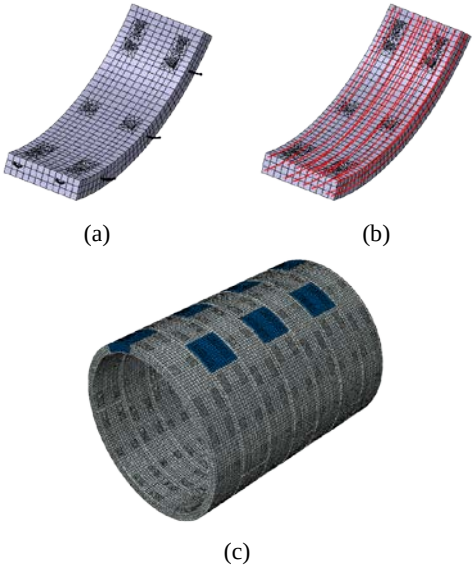
the normal direction and a friction coefficient of 0.5 in the tangential direction. Given the study requirements and computational constraints, seven lining rings were modeled for our analysis. The FE model of the seven assembled rings, including the concrete segments, bolts, and main steel bars, is depicted in Fig. 4.



(a) (b)

(c)

Fig. 4 Numerical model: (a) segment and bolts; (b) steel bars; (c) seven lining rings

2.2 Loading conditions

The Jinan Metro R2 Line is taken as a case study. The largest burial depth of the shield tunnel is 22.5 m, with a surrounding soil weight of 18 kN/m³ and a lateral earth pressure coefficient of 0.5. In the simulations, the burial depth is fixed at 22.5 m, and the earth pressure on the lining is depicted in Fig. 5. The ground reaction force is simulated by ground springs, and the radial and tangential ground springs are adopted at the outer surface of the lining rings. The radial ground springs have a stiffness k_n of 15000 kN/m³ in the compressive state and 0 in the tensile state. According to the "Standard for the design of shield tunnel engineering (GB/T 51438-2021)", the stiffness k_s of the tangential ground springs is taken as 1/3 of the radial ground spring stiffness (Ministry of Housing and Urban-Rural Development of the People's Republic of China, 2021). In Fig. 5, g represents the self-weight of the lining. Previous studies indicate that the longitudinal stress between rings can exceed 1.2 MPa (Liu et al., 2021). Thus, in the simulations we set the minimum longitudinal stress between adjacent rings at 1.2 MPa.

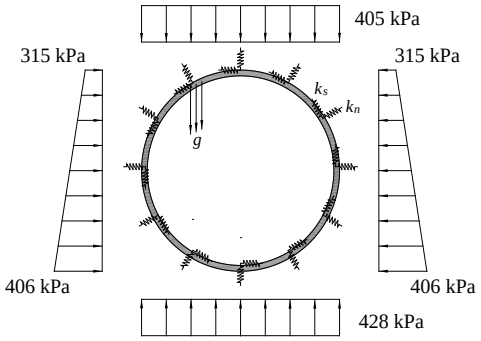


Fig. 5 Loading on lining structures

To assess the effects of localized versus full-line overload on the structural behavior of lining rings, two distinct loading scenarios are modeled with the finite element method as shown in Fig. 6. The first scenario involves a top overload (ΔP) on three consecutive rings (Rings 3-5), simulating a localized overload. The second scenario applies ΔP to all seven rings (Rings 1-7), representing a full-line overload. The simulation sequence begins with the application of existing earth pressure (Fig. 5) and longitudinal stress of 1.2 MPa to establish the post-construction stress and deformation state of the lining rings. Subsequently, the top overload ΔP is incrementally applied to the specified rings at levels of 100 kPa, 200 kPa, and 300 kPa.

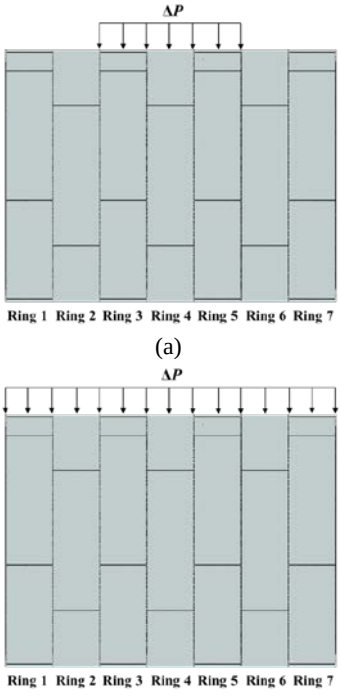


Fig. 6 Overload conditions: (a) localized overload on three lining rings; (b) full-line overload on seven lining rings

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2.3 Validation with a segmental joint bending test

To validate the numerical models, a bending test on a segmental joint of a lining ring was performed. The length of the segment in the experiment is 1/3 of the standard segment, and the two segments are assembled to form a segmental joint. The center angle of the experimental segment is 22.5°, with a thickness of 0.3 m and a width of 1.2 m. The strength grade of the segment concrete, the longitudinal main reinforcement arrangement, the hand hole size, and the bolt strength and size are all the same as those described in Section 2.1. The experiment is conducted using a loading system designed for shield tunnel joints at Tongji University. The setup includes a self-balancing frame, vertical and horizontal actuators, supports, and a controller, and is capable of bi-directional loading. The joint's internal forces, excluding the specimen's self-weight, are calculated as follows: bending moment $M = 0.7 \cdot P - 0.0936 \cdot F$, and axial force $N = F$, where P and F are the vertical and horizontal loads applied by the respective actuators. The details of the segmental joint bending test are shown in Fig. 7.

When constructing the segmental joint model in ABAQUS, the material properties and dimensions are consistent with those of the experimental setup. As with the lining ring model detailed in Section 2.1, the joint model omits water-stop grooves and EPDM rubber water-stop belts. A depiction of the model is presented in Fig. 8. In the loading process, the values and positions of the applied loads are the same as those in the joint experiment.

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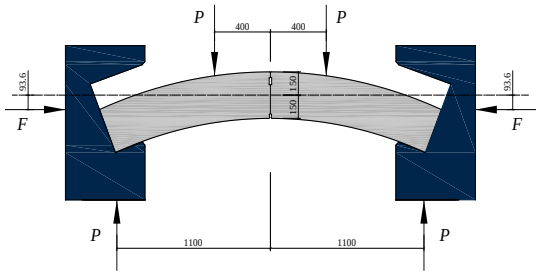

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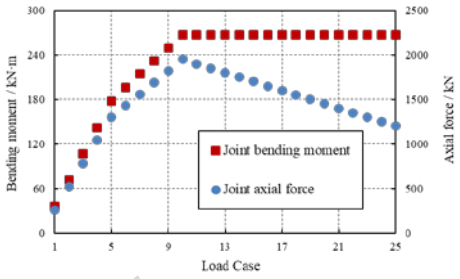
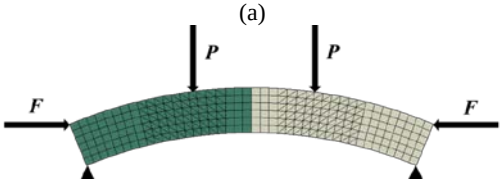


(c)

Fig. 7 Details of the segmental joint bending test: (a) loading equipment; (b) loading mode (unit: mm); (c) bending moment and axial force at joint position

(a)



(b)

Fig. 8 Joint numerical model: (a) segmental joint; (b) loading

Numerical calculations of the segmental joint opening are compared to experimental data. As depicted in Fig. 9, the numerical result closely mirrors the experimental pattern of joint opening. At a constant bending moment of 267 kN·m, a reduction to 1700 kN in axial force initiates joint opening in both the numerical model and the experiment. The numerical model predicts joint failure at an axial force of 1500 kN, which is a 3.2% discrepancy from the experimental failure force of 1550 kN. This variance

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may stem from the exclusion of features like water-stop grooves and EPDM rubber belts in the numerical model. Fig. 10 illustrates the compressive damage area during joint failure, with the numerical model closely aligning with experimental observations. Given the congruence in the joint opening patterns (Fig. 9) and the compressive damage areas (Fig. 10), the numerical model is deemed accurate for simulating the test results and representing the stress conditions of the joint.

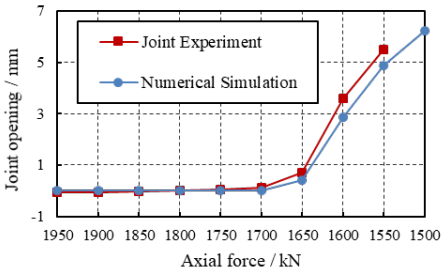
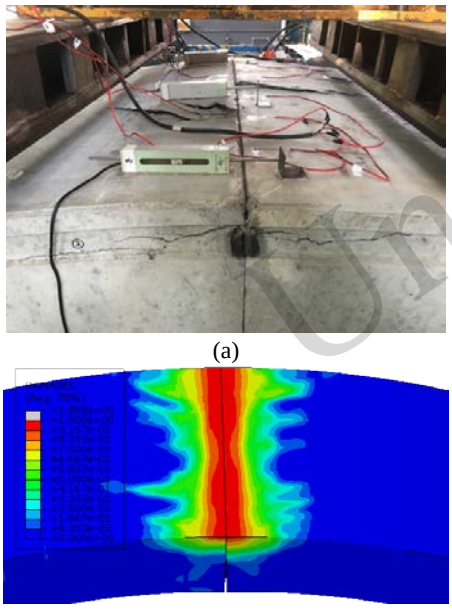


Fig. 9 Comparison of joint opening



(a)

(b)

Fig. 10 Comparison of concrete failure: (a) experimental result; (b) numerical simulation result

3 Performance of shield segmental linings under localized overload

3.1 Deformation and internal force of the lining rings

The numerical models encompass seven shield-lining rings. Given the symmetry of the Ring 1 - Ring 3 and Ring 5 - Ring 7 lining rings with respect to central Ring 4, our deformation and internal forces analysis focuses on the four rings from Ring 1 to Ring 4. When subjected to a top overload, the lining rings exhibit a horizontal egg-shaped deformation pattern, which means the distance between the tunnel crown and the tunnel bottom (vertical convergence deformation) decreases, while the distance between the two sides of the tunnel waists (horizontal convergence deformation) increases. In the overloading process, the horizontal convergence exceeds the vertical convergence in magnitude. Fig. 11 delineates the pattern of horizontal convergence deformation for Rings 1 through 4 as the overload is applied. Additionally, Fig. 12 illustrates the deformation profile of these rings under a 300 kPa overload scenario.

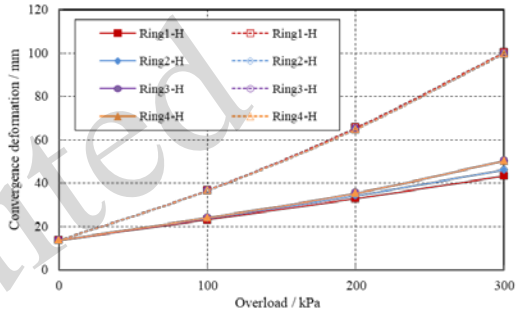
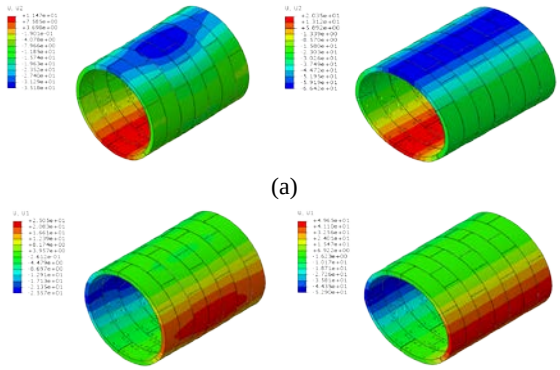


Fig. 11 Lining deformation (H means horizontal convergence deformation; the solid and dotted lines represent overload on Ring 3 - Ring 5 and on Ring 1 - Ring 7, respectively)



(a)

(b)

Fig. 12 Lining deformation (left: 300 kPa overload on Ring 3 - Ring 5; right: 300 kPa overload on Ring 1 - Ring 7; unit: mm; deformation magnification: 10 times): (a) vertical deformation; (b) horizontal deformation

The deformation of the lining rings under full-line overload is notably higher than under localized overload, as shown in Figs. 11 and 12. Under full-line overload, all lining rings converge uni-

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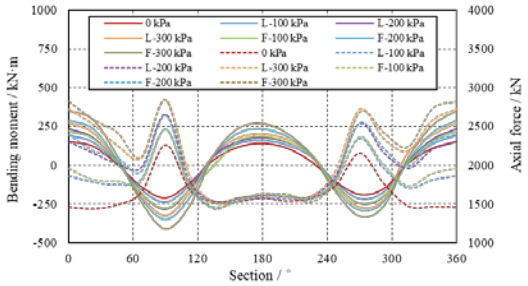
formly, but with localized overload, the deformation varies. Specifically, when Rings 3 to 5 are overloaded, they show more similar and larger deformations compared to Rings 1, 2, 6, and 7. The deformation diminishes with increasing distance from the overloaded Rings 3 to 5. At a localized overload of 300 kPa, the deformations for Rings 4, 3, 2, and 1 are 50.20 mm, 50.20 mm, 46.07 mm, and 43.19 mm, respectively.

Fig. 13 illustrates the internal force (bending moment and axial force) changes in Rings 1 to 4 during the overloading process, with 0° and 180° indicating the tunnel crown and bottom, and 90° and 270° indicating the tunnel waist sides. The bending moment and axial force in each lining ring increase with larger overload, in both the full-line and localized overload scenarios. For a 300 kPa overload, the internal force and corresponding lining deformation for Rings 1-4 are shown in Table 1. When overloaded, linings deform towards the soil, which reacts with increasing force as deformation grows. Full-line overloads cause greater deformation, and thus higher bending moments and axial forces, compared to localized overloads (see Table 1). For full-line overloads, Rings 4 to 1 show similar deformations (99.69 mm, 99.69 mm, 99.69 mm, and 100.20 mm) and naturally have similar forces. However, under localized overload, Rings 4 and 3 experience higher forces due to greater deformations (50.20 mm and 50.20 mm for Rings 4 and 3, versus 46.07 mm and 43.19 mm for Rings 2 and 1).



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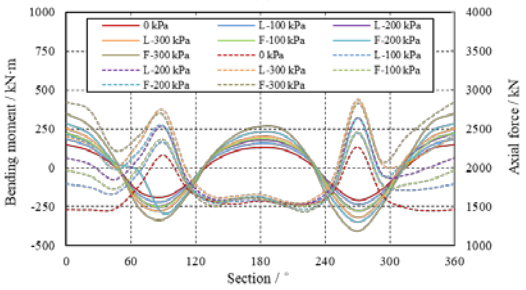

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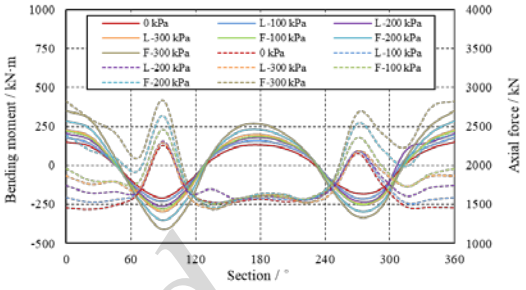

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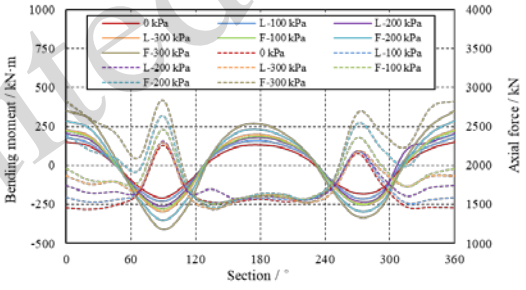

(d)

Fig. 13 Lining internal force (L and F mean 300 kPa overloads on Rings 3 to 5 and Ring 1 - Ring 7, respectively; solid and dotted lines represent bending moment and axial force, respectively): (a) Ring 4; (b) Ring 3; (c) Ring 2; (d) Ring 1

Table 1 Internal force and corresponding lining deformation for Rings 1-4 at 300 kPa overload

Ring	Full-line overload			Localized overload		
	Bending moment (kN·m)	Axial force (kN)	Deformation (mm)	Bending moment (kN·m)	Axial force (kN)	Deformation (mm)
Ring 4	–413-350	1435-2838	99.69	–324-276	1512-2843	50.20
Ring 3	–409-350	1443-2848	99.69	–320-255	1564-2880	50.20

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Ring 2	–413-351	1439-2834	99.69	–295-227	1447-2300	46.07
Ring 1	–413-351	1485-2939	100.20	–291-209	1488-2397	43.19

In summary, the analysis reveals that localized overload simulation produces a more varied response, with stronger forces concentrated in the immediate vicinity of the overload. The difference in these forces for full-line and localized overload scenarios highlights the importance of understanding the conditions and distribution of the load when assessing the structural integrity and safety of tunnel linings. Thus, our analysis suggests that when simulating the mechanical behaviors and failure mechanisms of lining rings under loading conditions, it is crucial to use the FE model with localized overload.

3.2 Joint deformation and bolt stress

Next, we discuss segmental joint opening and ring joint dislocation of the lining rings when subjected to localized overload. The joint deformation and bolt stress escalate with an increase in lining ring deformation. Fig. 14 shows the maximum values of ring joint dislocation and stress of longitudinal bolts at different ring joint positions. Fig. 15 illustrates the segmental joint opening in Ring 4 under a 300 kPa overload. Larger joint openings occur at the tunnel bottom, which experiences positive bending moments, as well as at the tunnel waist, which experiences negative bending moments. Circumferential bolts, which are positioned near the segmental joints' inner sides, restrict joint openings. Their constraining effect is stronger on the inner sides of the joints compared to the outer sides, leading to the largest joint opening occurring at the tunnel waist under negative bending. As depicted in Fig. 15, the L2-A3 joint opening measures 1.89 mm under localized overload at 300 kPa, contrasting with a 4.78 mm opening under full-line overload. The stress on circumferential bolts at the joints of locally overloaded lining rings is lower than in full-line overload scenarios, which is due to the smaller joint openings.

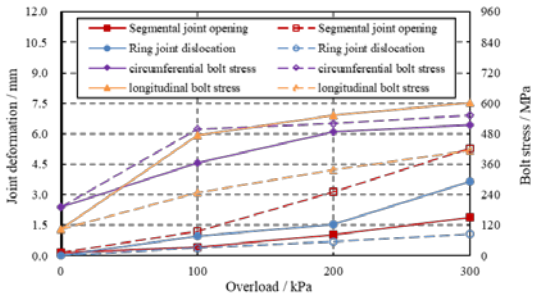

Fig. 14 Maximum joint deformation and bolt stress (solid and dotted lines represent overloads on Rings 3 to 5 and Rings 1 to 7, respectively)

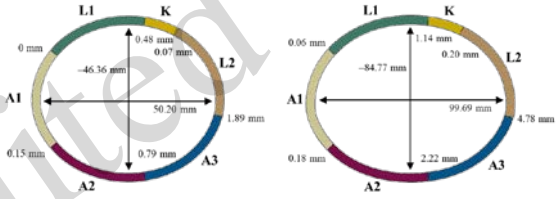

Fig. 15 Segmental joint opening in Ring 4 (left: 300 kPa overload on Rings 3 to 5; right: 300 kPa overload on Rings 1 to 7; deformation magnification: 10 times)

Fig. 14 indicates that the maximum bolt stress in the lining rings reaches 553.6 MPa under full-line overload, surpassing the 514.8 MPa observed under localized overload conditions; however, localized overload poses a greater risk to the structural integrity of lining rings due to increased joint dislocation and bolt stress. Under full-line overload, deformation is uniform across all lining rings, leading to minimal joint dislocation (maximum 1.06 mm), while with localized overload, Ring 3 and Ring 4 show greater deformation (50.20 mm for both) than Ring 2 and Ring 1 (46.07 mm and 43.19 mm), causing significant joint dislocation (maximum 3.66 mm) between adjacent rings. The stress on longitudinal bolts at ring joints is higher in localized overload (maximum 601.6 MPa) than in full-line overload (415.3 MPa), due to greater joint dislocation.

3.3 Concrete plastic strain and steel bar stress

The plastic strain in the concrete of the shield

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lining rings is depicted in Fig. 16, for an overload of 300 kPa. Notably, the concrete experiences tensile strain at the inner surfaces of the tunnel crown and bottom, and at the outer surfaces of the tunnel waists, where plastic strain is substantial. This is due to the positive and negative bending moments putting the concrete in a tensile state. Furthermore, significant plastic compressive strain is observed at the segmental joints near the tunnel waists, while substantial plastic tensile strain occurs around certain bolt handholes. The former phenomenon is attributed to the opening of the external sides of the joints, compressing the internal concrete. The latter is due to the tensile load from the bolts when the joints open or dislocate, affecting the concrete at handhole locations.

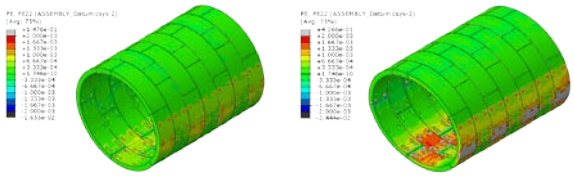

Fig. 16 Concrete plastic strain distribution (left: 300 kPa overload on Rings 3 to 5; right: 300 kPa overload on Rings 1 to 7; deformation magnification: 10 times)

Fig. 17 presents the stress distribution in the steel bars of Ring 4, which exhibits the greatest deformation among the seven rings. The steel bars in Ring 4 experience higher stress at the tunnel crown and bottom, and at both sides of the tunnel waists. The positive bending moments at the crown and bottom result in tensile stress in the steel bars near the inner surface, and compressive stress in the bars near the outer surface. Conversely, at the tunnel waists with negative bending moments, the outer surface steel bars are under tension, and the inner surface bars are under compression.

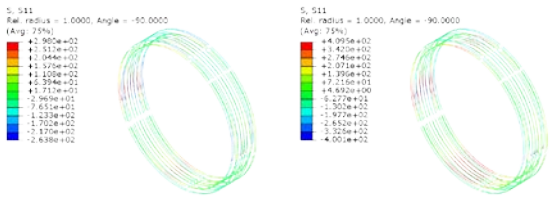

Fig. 17 Stress distribution of the steel bars in Ring 4 (left: 300 kPa overload on Rings 3 to 5; right: 300 kPa overload on Rings 1 to 7)

Due to the fact that the deformation of Ring 4 in the full-line overload scenario of 300 kPa is greater than in the localized scenario with the same value, the plastic strain distribution area of the concrete (see Fig. 16) and maximum stress of the steel bars under localized overload are smaller than those under full-line overload. These maximum stresses of the tensile and compressive reinforcements are 298.0 MPa and 263.8 MPa, compared to 409.5 MPa and 400.1 MPa, respectively.

3.4 Discussion of the failure characteristics of lining rings

The analysis from Sections 3.1 to 3.3 indicates that when a full-line shield tunnel is subjected to an overload of 300 kPa, the deformation of the lining rings, the opening of the segmental joints, the stress on circumferential bolts at the segmental joints, the extent of the plastic zone in the segmental concrete, and the stress on the main reinforcements are all greater than under a localized overload of the same magnitude. Conversely, ring joint dislocation and the stress on longitudinal bolts at ring joints are lower under full-line overload, showing the opposite relationship compared to localized overload conditions.

For the shield segmental linings, which are composed of prefabricated reinforced concrete segments and bolts, the segmental and ring joints are identified as the weak points. Failure of these joints, particularly when bolts break, can lead to a continuous collapse within the lining structure, as cited by Liu et al. (2016, 2017) and Gao et al. (2024). The bolts in the shield lining structures of the Jinan Metro R2 Line have a yield strength of 480 MPa and an ultimate strength of 600 MPa. Under full-line overload, the maximum stress at the segmental joint bolts reaches 553.6 MPa, surpassing the yield strength and entering the plastic deformation phase, while the stress at the ring joint bolts remains at 415.3 MPa within the elastic stage. Under localized overload, the segmental joint bolts experience a maximum stress of 514.6 MPa and also enter the plastic deformation phase, whereas the ring joint bolts are subjected to 601.6 MPa of stress, indicating potential damage.

It is evident that when only certain lining rings in the tunnel are overloaded, significant dislocation can occur at the ring joints between the overloaded and adjacent non-overloaded rings; this may lead to bolt stresses exceeding their ultimate strength and result in bolt failure. This suggests that the primary

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risk of failure for shield segmental lining rings originates from bolt failure at the ring joint positions. Therefore, when examining the mechanical properties of shield segmental linings under localized overload (which is a more common scenario in practice), it is crucial to thoroughly consider the impact of ring joint dislocation and the stress in longitudinal bolts at ring joints on the structural integrity and failure characteristics of the linings.

4 Reinforcement of shield segmental linings under localized overload

4.1 Reinforcement measures

The standard approach for mitigating large deformation issues in shield tunnel linings involves affixing steel rings to the inner surfaces of the lining rings. This technique creates a composite system that enhances the overall rigidity and load-bearing capacity of the tunnel (Liu et al., 2020, 2022, 2023; Li et al., 2023, 2024; Zhai et al., 2023). Fig. 18 presents two different reinforcement strategies which we will model numerically: one involving steel rings placed within individual lining rings, and another with steel rings spanning two adjacent rings. The steel rings have an outer diameter of 5.8 m and a uniform thickness of 20 mm. For the internal reinforcement scheme (referred to as the 'inside ring reinforcement scheme'), each lining ring is equipped with a steel ring that is 1.1 m wide, resulting in a total of seven such rings within the structure, as depicted in Fig. 18a. The spanning reinforcement scheme (referred to as the 'spanning ring reinforcement scheme') features steel rings that are also 1.1 m wide, but instead span across two adjacent lining rings, totaling six steel rings. Additionally, a narrower steel ring, measuring 0.55 m in width, is placed within the first and seventh rings, as depicted in Fig. 18b. It is important to note that in both reinforcement schemes, the cumulative weight of the steel rings is the same.

For the numerical models, the dimensions, materials, and methods for establishing the components of the seven lining rings are the same as described in Section 2.1. The material of the steel rings is Q345 steel with a yield strength of 345 MPa, an elastic modulus of 210 GPa, and a Poisson's ratio of 0.3. The simulation of the steel rings employs solid elements of type C3D8R. The interaction between the outer surface of the steel rings and the inner surface of the lining rings is achieved by setting a "Cohesive Behavior" in the ABAQUS software. The interfacial parameters between the lining rings and steel rings are drawn from existing literature (Liu et al., 2023), with a normal stiffness set to 144 MPa/mm and a tangential stiffness set to 40 MPa/mm; other specific parameters can be found in Li et al. (2024). The numerical models of shield segmental linings strengthened by steel rings for the two reinforcement schemes are shown in Fig. 19. The applied localized overload in the numerical models is the same as used in Section 2.2. The construction of steel rings is set to occur after the application of the load shown in Fig. 5, and the longitudinal stress of 1.2 MPa is applied to the lining rings. The workflow is as follows: (1) first, the "Model Change" function in the ABAQUS software is used to set the steel ring to the inactive state; (2) then, the load shown in Fig. 5 and the longitudinal stress of 1.2 MPa are applied to the segmental lining rings; (3) finally, the steel rings are activated and the overload ΔP (shown earlier in Fig. 6a) is subsequently applied.

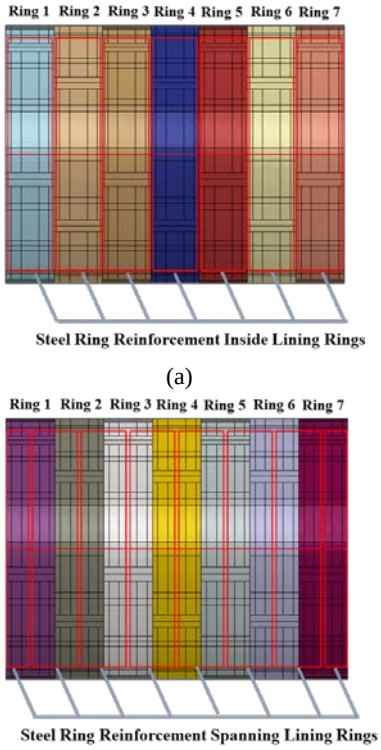

Fig. 18 Reinforcement measures for lining rings: (a) inside ring scheme; (b) spanning ring scheme

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(a)

(b)

Fig. 19 Numerical models of segmental lining rings strengthened by steel rings: (a) steel rings inside lining rings; (b) steel rings spanning lining rings

The linings of Rings 1 to 4 are investigated in terms of deformation and internal forces due to their symmetry with the reinforced rings from Ring 5 to Ring 7, on either side of the central Ring 4. These rings show a "horizontal egg"-shaped deformation pattern, where the horizontal deformation exceeds the vertical deformation under localized overload. Fig. 20 illustrates the horizontal deformation trend for Rings 1 to 4, while Fig. 21 shows their deformation under a 300 kPa localized overload. These results indicate that both reinforcement schemes significantly reduce horizontal convergence deformation under a 300 kPa localized overload. Specifically, for the inner ring reinforcement scheme, the deformations for Rings 1 to 4 are 23.96 mm, 25.71 mm, 28.76 mm, and 29.05 mm, respectively. In contrast, for the spanning ring reinforcement scheme, these deformations are slightly lower at 25.25 mm, 25.83 mm, 26.51 mm, and 26.51 mm. This reduction in deformation ranges from 40% to 50% compared to an un-reinforced scenario, where the deformations are 43.19 mm, 46.07 mm, 50.20 mm, and 50.20 mm for Rings 1 to 4, respectively.

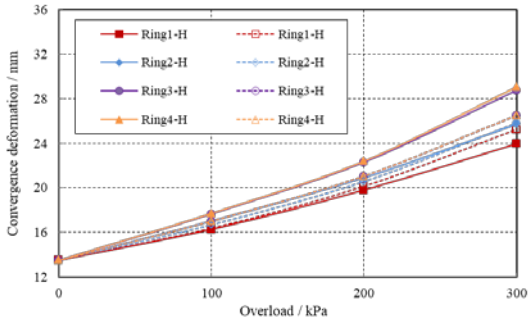
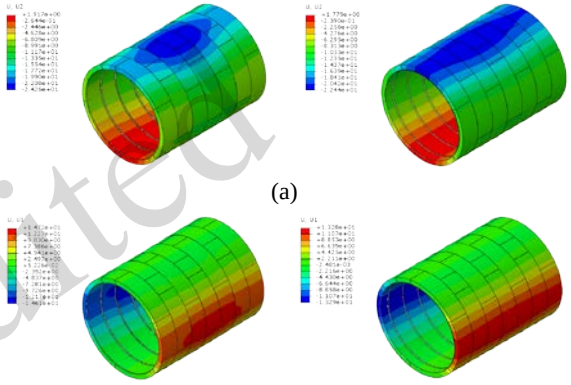


Fig. 20 Deformation of reinforced lining rings (H refers to horizontal convergence deformation; solid and dotted lines represent the inside and spanning ring reinforcement schemes, respectively)



(a)

(b)

Fig. 21 Deformation of reinforced lining rings with localized overload of 300 kPa (left: inside ring reinforcement scheme; right: spanning ring reinforcement scheme; deformation magnification: 10 times): (a) vertical deformation; (b) horizontal deformation

The inner ring reinforcement scheme enhances the stiffness of individual rings but does not effectively restrict joint dislocation between adjacent rings. Conversely, the spanning ring reinforcement scheme increases ring stiffness and leverages longitudinal bolts at joint positions to control joint dislocation. This results in a more uniform deformation across adjacent rings. For instance, under a 300 kPa overload, the difference in deformation between overloaded Ring 3 and adjacent Ring 2 without overload is 3.05 mm for the inner ring scheme, but only 0.32 mm for the spanning ring scheme. This highlights the superior performance of the spanning ring reinforcement in minimizing deformation disparities between neighboring rings.

Fig.22 compares the bending moments and axial

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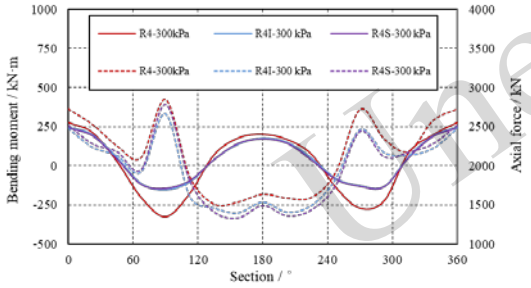
forces in the lining rings of Rings 1 to 4, both with and without steel reinforcement, under a 300 kPa localized overload. The data reveals that reinforcement with steel rings leads to a reduction in both the bending moment and axial force for each ring; however, the bending moments and axial forces are nearly identical for the lining rings at the same position, regardless of which reinforcement scheme is used. The reduction rates of the maximum negative and positive bending moments and the maximum axial force are shown in Table 2. The maximum reduction rates of the negative bending moment, positive bending moment, and axial force for the inner reinforcement scheme are 63.9%, 12.2% and 31.9%, and those for the spanning reinforcement scheme are 59.1%, 13.0% and 27.7%, respectively. There are two reasons for this internal force reduction. The first is that the steel ring reinforcement reduces the deformation of the shield lining by decreasing the ground reaction force. In addition, the reinforced steel rings also share part of the load, leading to a reduction in the internal forces within the lining rings.



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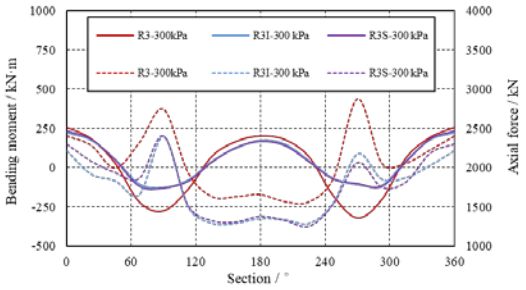

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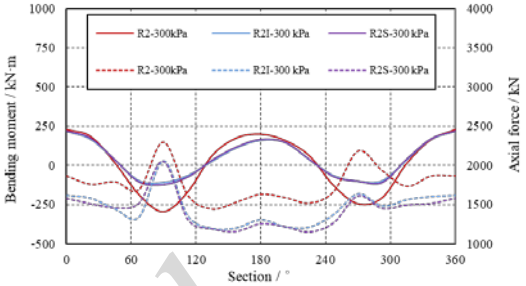

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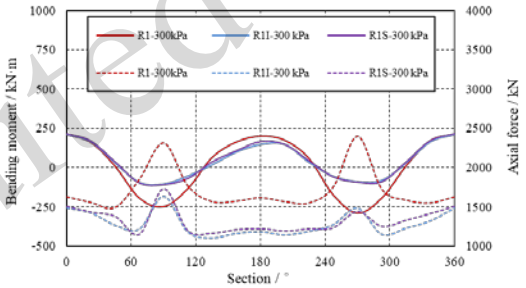

(d)

Fig. 22 Comparison of bending moment and axial force in lining rings with and without steel ring reinforcement (I and S refer to the inside and spanning ring reinforcement schemes, respectively; solid and dotted lines represent the bending moment and axial force, respectively): (a) Ring 4; (b) Ring 3; (c) Ring 2; (d) Ring 1

Table 2 Internal force reduction for reinforced lining rings

Ring	Non-reinforced		Reinforced with steel rings inside lining rings			Reinforced with steel rings span lining rings		
	Bending moment (kN·m)	Axial force (kN)	Bending moment (kN·m)	Axial force (kN)	Reduction rate of maximum negative and positive bending moment, and axial force (%)	Bending moment (kN·m)	Axial force (kN)	Reduction rate of maximum negative and positive bending moment, and axial force (%)
Ring 4	–324-276	1512-2843	–153-244	1396-2664	52.8, 11.6, 6.3	–146-240	1334-2780	54.9, 13.0, 2.2
Ring 3	–320-255	1564-2880	–130-224	1291-2397	59.4, 12.2, 16.8	–131-236	1251-2402	59.1, 7.4, 16.6
Ring 2	–295-227	1447-2300	–110-218	1192-2061	62.7, 4.0, 10.4	–121-220	1153-2053	59.0, 3.1, 10.7
Ring 1	–291-209	1488-2397	–105-209	1104-1632	63.9, 0.0, 31.9	–121-220	1146-1732	58.4, –5.2, 27.7

Fig. 23 illustrates the maximum segmental joint openings and circumferential bolt stresses at the joint positions, as well as ring joint dislocations and longitudinal bolt stresses at the ring joint positions in the

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reinforced lining rings of Rings 1-4 during localized overloading. The maximum joint opening in the lining rings reinforced with steel rings spanning the lining rings is less than that in the rings reinforced with steel rings inside the lining rings, due to reduced deformation from the spanning reinforcement scheme. Fig. 24 showcases the joint opening distribution in Ring 4, which experiences the greatest deformation among the seven reinforced lining rings under a 300 kPa localized overload. The maximum opening for the L2-A3 segmental joint in Ring 4 is 1.72 mm for the inside ring reinforcement scheme, and 1.21 mm for the spanning ring reinforcement scheme. Because the maximum opening of the segmental joint in the lining rings with the inside ring reinforcement scheme is larger, the stresses in circumferential bolts at the segmental joint positions are also greater. Fig. 23 shows that under a 300 kPa localized overload, the maximum circumferential bolt stress reaches 497.3 MPa for the inside ring reinforcement scheme, surpassing the 487.7 MPa for the spanning ring reinforcement scheme.

The reduced deformation disparity between adjacent lining rings results in lower ring joint dislocation for the spanning ring reinforcement scheme as compared to the inside ring reinforcement scheme, as shown in Fig. 21a. The data from Fig. 23 indicates that the maximum ring joint dislocations for the inside ring and spanning ring reinforcement schemes are 2.31 mm and 0.67 mm, respectively. Furthermore, the maximum stress for longitudinal bolts at the ring joint positions in the reinforced lining rings is 597.3 MPa for the inside ring scheme, and 481.7 MPa for the spanning ring scheme.

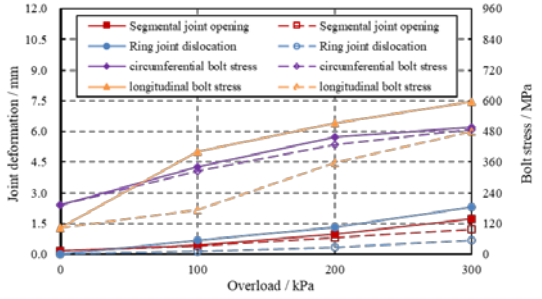


Fig. 23 Maximum joint deformation and bolt stress in reinforced lining rings (solid and dotted lines represent the inside and spanning ring reinforcement schemes, respectively)

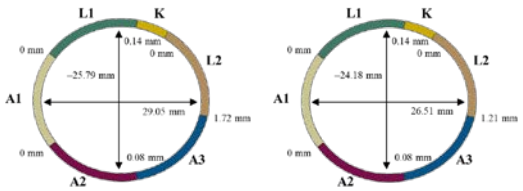
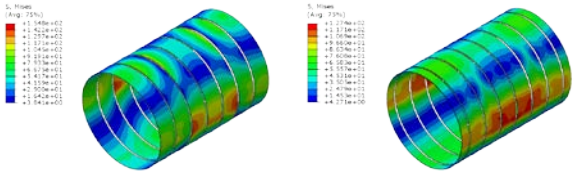


Fig. 24 Segmental joint opening in reinforced Ring 4 (left: inside ring reinforcement scheme; right: spanning ring reinforcement scheme; deformation magnification: 10 times)

4.2 Discussion of reinforcement effects

Fig. 25 presents the stress distribution in the steel rings for the two reinforcement schemes under a 300 kPa localized overload. It is evident that the stress levels in both schemes are significantly lower than the yield strength of 345 MPa for the rings' Q345 steel. As depicted in Fig. 25a, the steel rings within the lining rings for the inside ring reinforcement scheme experience greater deformation, resulting in a higher maximum Mises stress of 154.8 MPa. This contrasts with the steel rings of the spanning ring reinforcement scheme, which exhibit a lower maximum Mises stress of 127.4 MPa. Fig. 25b further illustrates that the maximum hoop stress in the steel rings for the inside ring reinforcement scheme is 108.4 MPa, which exceeds the 73.68 MPa observed in the spanning ring reinforcement scheme. However, the shearing stress in the steel rings is influenced by the differential deformation between adjacent lining rings. As shown in Fig. 25c, the maximum shearing stress for the steel rings in the spanning ring reinforcement scheme is 27.71 MPa, which is notably higher than the 16.42 MPa found within the lining rings for the inside ring reinforcement scheme. This suggests that while the spanning reinforcement scheme may have lower Mises and hoop stresses, it must also be designed to accommodate the increased shearing stress.



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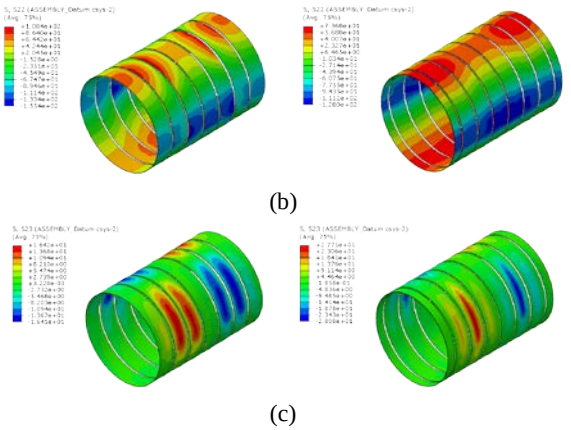

Fig. 25 Stress distribution in the steel rings (left: inside ring reinforcement scheme; right: spanning ring reinforcement scheme; unit: MPa; deformation magnification: 10 times): (a) Mises stress; (b) hoop stress; (c) shearing stress

Overall, from evaluating the effects of localized overload on shield tunnel lining structures, the reinforcement scheme in which steel rings span the lining rings proves to be more effective. The spanning reinforcement strategy mitigates joint dislocation and reduces stress on longitudinal bolts, with the reduction in maximum ring joint dislocation ranging from 70% to 82%, and the reduction in maximum longitudinal bolt stress ranging from 19% to 57% compared to the within ring reinforcement strategy. There are two primary advantages of the spanning steel ring scheme: it enhances the stiffness of each individual lining ring. and effectively restricts dislocation deformation at the ring joints. This restriction is achieved through the steel rings working in conjunction with longitudinal bolts at the ring joint positions, which in turn reduces the stress experienced by the longitudinal bolts. By limiting joint dislocation and sharing the load, the spanning steel ring scheme minimizes deformation differences between adjacent lining rings, leading to a more uniform and controlled structural response under localized overload conditions.

5 Conclusions

In this study we utilized three-dimensional finite element (FE) models to investigate the evolution of mechanical performance and underlying failure mechanisms in shield segmental lining rings. This investigation compared the effects of localized overload with those of full-line overload. Furthermore, we delved into the reinforcement impact of steel rings placed within individual lining rings and those set to span multiple lining rings, under localized overload conditions. An optimal reinforcement strategy, tailored for localized overload situations, was proposed based on these analyses.

In scenarios where a subset of the shield segmental lining rings is subjected to overload, significant joint dislocation occurs between the overloaded and adjacent non-overloaded rings. This can lead to longitudinal bolt failure when the bolt stress surpasses its ultimate strength. Compared to full-line overload, localized overload results in greater ring joint dislocation and longitudinal bolt stress. However, the deformation of lining rings, the segmental joint opening, the circumferential bolt stress, the extent of plastic zones in segment concrete, and the steel bar stress are comparatively lower. It is therefore imperative to consider the influence of ring joint dislocation and longitudinal bolt stress on the bearing capacity and failure characteristics of the linings in the more common practical scenario of localized overload.

The reinforcement scheme involving steel rings that span across two adjacent lining rings was shown to be the more effective of the two tested schemes. This approach results in reduced lining deformation and internal forces, segmental and ring joint deformation, circumferential and longitudinal bolt stress, and stress in the reinforcing steel rings as compared to the strategy of installing steel rings within individual lining rings. While employing steel rings within individual lining rings does augment the rigidity of each ring, this strategy falls short in addressing the dislocation of ring joints and the high stress experienced by the longitudinal bolts at these critical points. Conversely, the strategy of using steel rings that span multiple lining rings not only enhances the structural rigidity of each ring, but also mitigates the dislocation at the ring joints and reduces the stress on the longitudinal bolts.

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Author contributions

Long ZHOU: Conceptualization, Methodology, Formal analysis, Writing – original draft, Funding acquisition. Zhiguo

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YAN: Project administration, Supervision, Resources. Mengqi ZHU: Investigation, Visualization, Validation, Software, Data curation, Writing – review & editing.

Conflict of interest

Long ZHOU, Zhiguo YAN, and Mengqi ZHU declare that they have no conflict of interest.

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中文概要

题　目：局部超载作用下盾构隧道管片衬砌破坏特性与加固效果数值模拟分析

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目　的：当盾构隧道全线中只有部分衬砌环周边超载时，其承载性能明显不同于全线超载的情况。现有研究多取一环或几环衬砌环为研究对象，在选取的衬砌环上施加荷载分析衬砌结构的性能演化规律，未能揭示局部超载时盾构衬砌环的承载与破坏特性。本文建立三维精细化有限元模型对比分析盾构衬砌环在全线与局部超载时的承载特性与破坏机制，并提出适用于局部超载作用时的衬砌环加固方法。

创新点：1. 研究了局部超载作用时盾构衬砌环的力学性能演化规律，揭示了其破坏机制；2. 分析了局部超载作用时钢环施作在衬砌环环内与环间两种加固方式时的加固效果，提出了最佳的衬砌环加固方式。

方　法：1. 在管片接缝抗弯试验验证的基础上，建立三维精细化有限元模型分析超载作用时盾构衬砌环内力与变形、衬砌接缝变形及螺栓应力等变化规律，在此基础上提出局部超载作用时盾构衬砌环的破坏特性。2. 对比分析局部超载作用下采用钢环在衬砌环环内与跨环加固时衬砌环内力与变形、衬砌接缝变形、螺栓应力及钢环应力的变化规律，进一步地，提出适用于局部超载作用时的钢环加固衬砌环布置方法。

结　论：1. 盾构隧道全线超载时衬砌环变形、管片接缝张开量与管片接缝螺栓应力、管片中混凝土塑性区分布范围均大于局部超载作用时的情况，但局部超载时衬砌环中环间接缝的错台量及环间接缝位置螺栓的应力大于全线超载作用时的情况。在分析实际工程中更为常见的局部超载作用下衬砌环的性能演化规律时，应充分考虑环间接缝错台变形及环间接缝螺栓的受力对盾构衬砌环承载与破坏特性的影响。2. 采用钢环环内加固的方案时，因钢环施作在衬砌环内，无法与环间接缝的螺栓一起限制相邻衬砌环之间环间接缝的错台变形。采用钢环环间加固的方案，不仅可提高衬砌环的刚度，布置在相邻衬砌环之间的钢环还可与环间接缝位置的纵向螺栓共同限制环间接缝的错台变形，减小相邻环之间纵向螺栓的应力。在盾构隧道上部某一区间存在局部超载时，钢环环间加固的方案更为有效。

关键词：盾构衬砌；局部超载；破坏特性；钢环加固

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