Case study on the model test of staggered-jointed assembling shield tunnel structure reinforced by bonded steel plates under large deformation

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Abstract. Transverse large deformation is one of the main diseases of metro shield tunnels, gravely threatening the safe operation of tunnel structure. Reinforcing bonded steel plates is a primary means for repairing excessively deformed tunnels. Nevertheless, evaluations and analyses on the reinforcing effect of bonded steel plates are currently unclear. Therefore, destruction tests were performed on the tunnel without the reinforced structure and on the excessively deformed tunnel with the reinforced structure respectively with the similar model test method in this paper, so as to analyse deformation behaviours and failure modes of segment lining and compare the reinforcing effect. Test results show that the carrying capacity of the tunnel structure can still be improved through reinforcing the staggered-jointed assembling shield tunnel with bonded steel plates under large deformation; also, the reinforcement of bonded steel plates can effectively prevent the tunnel from deforming. The failure mode of the reinforced structure consists of the reinforcement interface debonding failure and steel plate bending failure. To be specific, the debonding failure being frequently detecting in seams is composed of slab staggering and expansive debonding. Research achievements in this paper may provide theoretical and data support for similar reinforcement projects.

1. Introduction

Since the metro shield tunnel is in a certain engineering geological environment, the strained condition of the shield tunnel structure will be changed with the increase of service time and the change of external environment. As a result, excessive transverse deformation of the shield tunnel may be caused to trigger leakage, crack, slab staggering, corner breakage, and distortion, as well as induce to tunnel diseases (Yuan et al., 2013; Huang et al., 2017) such as local sedimentation, severe slab staggering, large crush, and long crack. Tunnel diseases have a huge impact on the tunnel structure, which will directly affect the service life and the operation safety of the metro tunnel, or even resulting in fatal accidents and heavy losses if those diseases are not treated.

For the safety of the tunnel structure, a specific requirement for the tunnel transverse convergence and deformation proposed in "GB50175-201 Metro Design Code" is that the total variation of diameter is less than 5‰D (D is the outer diameter of shield tunnel) under the effect of external load.
after the ring is assembled. However, the transverse deformation of the tunnel exceeded the limit frequently in recent years together with the occasional happening of accidents raised by abnormal structure in the shield structure with the increase of service time and the influence of surrounding environment based on measured data obtained from the operating tunnels (Huang et al., 2013; Li et al., 2014).

For tunnels with large transverse deformation, the tunnel structure should be reinforced to meet the needs of future tunnel operation apart from typical measures like lining repair and water penetration blocking, etc. Currently, the shield tunnel is mainly repaired and reinforced by bonded steel plates against the large transverse deformation (Kiriyama et al., 2013). Unfortunately, few research findings concerning the technique of reinforcing the metro shield tunnel projects can be found in the industry. What’s more, a big difference exists in the performance of structural mechanics under different assembling ways since the straight joint assembling shield tunnel is a main research focus (Tang et al., 2014; Zhao et al., 2016; Zhang et al., 2019;). Besides, researches of staggered joints reinforced by steel plates were mainly based on numerical analysis and engineering cases, lacking demonstration testing (Chang et al., 2001; Huang et al., 2016;).

With Xiamen Metro as the support project, a test study adopting the bonded steel plate as the reinforced structure was conducted in this paper against staggered-jointed 45° assembling shield tunnel at the sludge stratum under the original structure and the 15‰D large deformation based on the model test with a similarity ratio of 1:5. By observing the test, relationships among loads, segment deformation and stud shaft, etc. were established to analyze strained deformation behaviors and failure means of the segment structure and the reinforcement effects based on the ultimate bearing capacity.

2. Similar model test scheme

2.1. Support project
Focusing on the section from Xiamen Island to Haicang of Xiamen Metro Line 2, the sludge stratum of the land section through which the metro tunnel pass was selected as the model test condition. The geological profile is shown in figure 1. Each segment is comprised of a capping (F), two adjoining blocks (L1 & L2) and three standard blocks (B1, B2, & B3), which is assembled at the 45°of staggered joints. The outer diameter, the inner diameter and the ring width of the tunnel are 6700mm, 6000mm, and 1500mm, respectively. The segment is made from C55 concrete and HRB400 reinforcing steel bar. And 16 circumferential bolts (M30) and 12 longitudinal bolts (M30) are contained in the segment connection. In this test, “1/2+1+1/2” three-ring segment structure was taken as the research object. Its assembling method is shown in figure 2.
2.2. Similarity models and reinforcement designs

By taking the geometric similarity ratio $C_L = 5$, and the volume-weight similarity ratio $C_\gamma = 1$ as the basic similarity ratios, similarity ratios of other physical and mechanical parameters according to the similarity theory are presented in Table 1.

Table 1. Similarity constants of physical quantities

| Geometry size $L$ (m) | Displacement $\delta$ (m) | Modulus of elasticity $E$ (N·m$^{-2}$) | Stress $\sigma$ (N·m$^{-2}$) | Elastic Resistance Coefficient $k$ (N·m$^{-3}$) | Strain $\varepsilon$ | Poisson ratio $\mu$ |
|-----------------------|--------------------------|-------------------------------|----------------------------|--------------------------------|-----------------|-----------------|
| 5                     | 5                        | 5                            | 5                          | 1                              | 1               | 1               |

Similar material of concrete is micro-concrete with high water-cement ratio (0.7), high sand percentage (50%), low elastic modulus (the maximum particle size of aggregate is 8mm), low-grade cement (R325). The similar material of the steel bar is 6063T4 aluminum. Reinforcement is performed in accordance with the prototype segment; the reinforcement cage is formed through binding thread main reinforcements, stirrups and distributing reinforcements converted by the similar ratio. The segment model is cast with a self-designed fine mold containing the structure of hand hole; meanwhile, rubber hoses are used for simulating the embedded sleeve during the casting, while the longitudinal ring bolts are simulated by M6 AZ91d magnesium alloy. Full-ring steel plate is applied in this test, which is reinforced in accordance with the flow chart shown in Figure 1. Specific reinforcement profile and parameters are presented in figure 3, Table 2 and Table 3, respectively. The reinforced tunnel structure is shown in figure 4.
M6 Expansion screw
Steel plate
The depth of implantation was 20 mm
Avoid hand holes and longitudinal seams

Figure 4. Reinforcement profile

Figure 5. Picture shot after the reinforcement.

Table 2. Anchorage parameters

| Block number | Material  | Thickness | Width  | Type    | Spacing | Row |
|--------------|-----------|-----------|--------|---------|---------|-----|
| 6            | 3005-O Al | 4mm       | 240mm  | M6 Q235 | 105mm   | 4   |

Table 3. Sealing and filling parameters

| circumferential seam, longitudinal seam and hand hole | Steel plate | Steel plate-concrete |
|-------------------------------------------------------|-------------|----------------------|
| Epoxy resin repairing mortar                          | Epoxy resin sealant | Epoxy resin grouting adhesive |

2.3. Stratum simulating and loading system
12 horizontal loading points and 6 vertical loading points are included in the loading system in the test, which consists of 2 hydraulic jacks and 1 hydraulic jack, respectively. All independently-controlled
hydraulic jacks are equipped with pressure sensors. Moreover, each horizontal loading point is provided with a stratum spring and a load distribution steel plate (the curvature is consistent with the outer diameter of the tunnel). Also, a flexible rubber gasket that can adapt to the deformed segment is added on the interface contacted by the steel plate and the tunnel lining. The device can apply the jack concentration force to the segment structure model in the form of uniformly-distributed load with the consideration of the interaction between the stratum and the structure, as shown in figure 6.

![Test loading device](image)

Figure 6. Test loading device.

According to the buried depth and the underground water level, the combined calculation of water and earth pressure is employed to calculate the prototype and the external water-earth pressure based on the full overburden earth pillar theory. Stratum parameters are shown in Table 4, and the water-earth pressure is shown in Table 5.

### Table 4. List of stratum parameters.

| Geotechnical name | Stratum thickness (m) | Gravity density (kN·m⁻³) | Coefficient of subgrade (kN·m⁻³) | Lateral pressure coefficient |
|-------------------|-----------------------|---------------------------|----------------------------------|-----------------------------|
| Prime fill        | 5.56                  | 18.7                      | 10                               | 0.43                        |
| Sand filling      | 1.88                  | 22.0                      | 20                               | 0.43                        |
| Silt              | 9.57                  | 16.2                      | 5                                | 0.72                        |

### Table 5. Water-earth pressure calculation sheet.

|                     | Design value | Prototype | Model |
|---------------------|--------------|-----------|-------|
| Top load \(q_1\) (kPa) | 190.0        | 38.0      |       |
| Lateral load \(q_2\)(kPa) | 136.8        | 27.4      |       |
| Intermediate load \(q_3\)(kPa) | 163.4        | 32.7      |       |

The test loading mode is horizontal static cyclic loading. The horizontal evenly-distributed load that is simulated by the “spring-steel plate” device of 12 directions, is divided into \(q_1\), \(q_2\) and \(q_3\). These three groups are fully synchronized during loading. Load relationships \(q_2=0.72 q_1\) and \(q_3=0.5(q_1+q_2)\) should be maintained in the test before the stratum spring is compressed completely. The circumferential load distribution is shown in Figure 7. The load application is loaded symmetrically to the test piece step by step. The single-step load increments are \(\Delta q_1= 2.0 \text{ kPa}\), \(\Delta q_2= 1.44 \text{ kPa}\), \(\Delta q_3= 1.72 \text{ kPa}\). By simulating the residual jacking force of the tunnel shield with the vertical load, the jacking force of the six vertical loading points is 1200N.

The failure mode and the ultimate bearing capacity of the original structure and the reinforcement structure in the overload condition were simulated in this test. The test condition is shown in Table 6. The test piece of the original structure increases the circumferential uniformly-distributed load according to the load step before the structural failure; and the reinforcement structure firstly increases the circumferential load according to the load step. The bonded steel plates are adopted for reinforcement when the vertical convergence is 15%\(D\) (\(D\) is the outer diameter of the tunnel); After
the reinforcement is completed, the reinforcement parts continuously increases the circumferential load according to the load step before the structural failure.

| Name                      | Number | Loading process                                                                 |
|---------------------------|--------|----------------------------------------------------------------------------------|
| Unreinforced structure    | S₀     | 0→Actual burial depth→Increase $q_1$, $q_2$, and $q_3$ to structural failure according to load step |
| Reinforced structure      | S₁     | 0→Actual burial depth→The vertical convergence reaches 15‰D by increasing $q_1$, $q_2$, and $q_3$ according to the load step→Reinforcement by bonded steel plate→Continue to increase $q_1$, $q_2$, and $q_3$ to structural failure according to load step |

2.4. Measurement system

For disclosing deformation behaviors and failure process of the original structure and the reinforcement structure of the shield tunnel segment, test contents involve strain of the mid-ring segment, strain of the reinforcement, strain of the bolt, strain of the steel plate, segment displacement, the opening amount of circumferential joint, the staggering amount of longitudinal joint, and debonding displacement of the steel plate. The monitoring point arrangement for each physical quantity is summarized in Table 7.

| Measurement contents          | Testing instruments or means | Accuracy | Measuring points |
|-------------------------------|------------------------------|----------|-----------------|
| displacement                  | Electronic displacement meter| 0.01mm   | 6               |
| Main reinforcement strain     | Foil strain gauge            | 1με      | 8               |
| Concrete strain               | Foil strain gauge            | 1με      | 12              |
| Bolt strain                   | Foil strain gauge            | 1με      | 48              |
| Dislocation of circumferential joints | Vernier caliper | 0.001mm  | 16              |
| Opening of longitudinal joints| Photographic Ranging         | 0.001mm  | 12              |
| Crack observation             | Fracture observation instrument | 0.01mm | -               |

3. Result analysis

3.1. Failure process and mode

The failure mode of segment structure is characterized by the shear extrusion failure of oblique section, reinforcement interface debonding and steel plate bending failure. Failure of the unreinforced structure was initially characterized by concrete cracking (in the middle ring at 0°, the camber inside the 180°of the upper and lower rings, and the outer camber of the 90°and 270°of the middle ring). The increased opening amount of the joint seam results in the yield of partial circumferential bolts at the unfavorable positions, causing weakened joint stiffness. As a result, the crack mode is turned into the cracked joint...
seam at the outer side (the outer side of 1# and 2# joint seams in the middle ring as well as the outer side of 3# and 4# joint seams in the outer side). In that case, the overall stiffness of the structure is lowered due to the deepened and widened cracks. Excessive deformation causes that the resistance passive on both sides surpass the loads at the top and bottom. Under the assembling condition set in this test, the maximum resistance passive can be witnessed on both sides of the structure at 270° with distinct shear and extrusion effects on the camber inside the segment. Ultimately, two connecting diagonal cracks were generated on the segment, at the #5 joint position of the mid and low ring, occurring the failure of sheared and extruded oblique section. The reinforcement structure before reinforcement had consistent failure feature, that is, cracks were found in the concrete in the tensile zone (camber inside 0° of the middle ring and 180° of the upper and lower rings) at the early stage of the unreinforced structure. The reinforced structure was remarkably lifted in overall stiffness. Although the load carrying properties of components of the segment may be well maintained at a certain stage, the interface bonding the steel plate and the concrete started to crack and debond (at the inner side of 2#, 4#, and 6# joints of the upper ring, the inner side of 5# and 6# joints in the mid ring and 1# joint in the lower ring) with the increase of load. In that case, the structural fracture mode was changed to the cracks on the bonding interface of joint seams, leading to directly-weakened stiffness of the joint. Meanwhile, the ever-increasing opening amount of joint seams and the staggering amount may result in increasing the deformation rate of the segment. Consequently, the overall stiffness of the structure was weakened, causing that a large number of cracks were found on the concrete in the tensile zone (the external cambers at 90° and 270° of the upper, mid and lower rings) and stressed joint seam (1#, 2#, 5#, and 6# joint seams outside of the middle ring as well as 2# and 4# joint seams of the upper ring). With the ever-widening and deepening increase of loads, the fracture mode covered the cracked concrete in the tensile zone and the extruded cracks at the outer joint seams. Gradually, the cracks on the bonding interface of the joint were extended to both sides. With the ever-deepening debonding level and cracked degree on the bonding interface, the steel plate rings were engaged in sharing part of pressure of the segment. After the concrete the 1# outer joint of the upper, middle and lower rings of the segment was crushed, the joint lost the load-carrying property and the dome slipped inward lead to the sheared and bent steel plate. At last, the defending reinforcement interface and the bending failure of the steel plate happened to the structure. The final failure mode of segment structure is shown in figure 8.

3.2. Load-displacement relationship
The load-displacement curve of the test segment is shown in figure 9. The inward segment deformation is defined as positive, while the outward is negative. The load and deformation of the unreinforced structure S0 is developed as follows: 1) when the structure is elastic in the stage of the top load q1 ranging from 0 to 146kPa, the segment deformation will be increased linearly with the rate of rise in vertical deformation and transverse deformation reaching 0.105mm/kPa and -0.102mm/kPa, respectively. 2) concrete in the tensile zone are cracked in succession in the stage of top load q1 ranging from 146 to 200kPa, the overall stiffness of the structure will be decreased with the increased rate of rise in segment deformation. And an inflection point A occurs on the load displacement curve when the coefficient of boundary resistance is increased due to the completely compressive spring. 3)
In the stage of the top load $q_1$ ranging from 200 to 446 kPa, a new inflection point B occurs in the load-displacement curve as the overall restraint of the outer boundary of the segment is increased. The joint lost its load-bearing properties due to the yield of part of bolts and the extruded cracks on the joint seam of the outer camber although the rate of rise in segment deformation slow down. Under such circumstance, the overall stiffness of the structure is still weakening. 4) When the top load $q_1$ is 455 kPa, two diagonal cracks were generated at the 4# joint part of the upper, middle and lower rings of the segment with the ever-deepening crack propagation. 5) When the top load $q_1$ is 523.6 kPa, two diagonal cracks at camber inside the left side of the segment are penetrated with the failure of the test piece. The vertical deformation value and the transverse deformation value of the structure during the failure of load are 60.7 mm and -53.92 mm, respectively.

The load and deformation of the reinforced structure $S_1$ is developed as follows: 1) when the structure is elastic in the stage of the top load $q_1$ ranging from 0 to 144 kPa before reinforcement, the segment deformation will be increased linearly with the rate of rise in vertical deformation and transverse deformation reaching 0.116 mm/kPa and -0.112 mm/kPa, respectively. 2) Part of the concrete in the tensile zone are cracked in the stage of the top load $q_1$ ranging from 144 to 164 kPa, the overall stiffness of the structure will be decreased slightly while the rate of rise in segment deformation will be slightly increased. 3) In the stage of the top load $q_1$ ranging from 164 to 342 kPa after the reinforcement, the structural stiffness is remarkably lifted after the reinforcement without any failure occurring in the structure at that stage. Meanwhile, the load-bearing properties of all components are well maintained, so that the structure is restored to the elastic working state. The segment deformation is increased linearly with the rate of rise in the vertical deformation and the transverse deformation reaching 0.038 mm/kPa and -0.036 mm/kPa, respectively. In comparison to the deformation in the elastic stage before reinforcement, the rate of rise is dramatically decreased. 4) In the stage of the top load $q_1$ ranging from 342 to 505 kPa after reinforcement, the overall stiffness of the structure is decreased with the increased and deepened fractured joint bonding interface and concrete cracks in the tensile zone. 5) In the stage of the top load $q_1$ ranging from 505 to 781 kPa after reinforcement, the extruded cracks on the outer joints of the camber of the segment vault are gradually penetrated, while the joints gradually lose the load-carrying property. 6) When the top load $q_1$ is 789 kPa after reinforcement, concrete on the camber of the outer vault is crushed with the debonding of the reinforcement interface. The steel plate is sheared and deformed, resulting in the failure of test piece. The vertical deformation and the transverse deformation of the structure are 58.74 mm and -56.28 mm, respectively, in failure load.
3.3. Longitudinal joint opening and bolt strain

The deformation and mechanical property curves of the typical failure positions 2# and 5# joints are shown in figure 10. The opening joint is defined as positive, the closing joint as negative; while the bolt strain is positive with tension, and negative with compression. By analyzing the figure a, regarding the original structure, in the stage of the top load $q_1$ ranging from 0 to 149 kPa, the opening amount of 2# outer joint is increased continuously; also, the strain of the circumferential bolt of #2 joint is increased rapidly to the plateau region. Moreover, as the load and deformation enlarges, joints inside the position are extruded and staggered. At this moment, the opening amount of the outer joint seam stops increasing. When the top load $q_1$ is 355 kPa, concrete on the camber #1 outer joint seams are crushed. Since the capping blocks are affected by staggered extrusion, the opening amount of #2 joint seam stops increasing till the structural failure. Regarding the reinforced structure, in the stage of the top load $q_1$ ranging from 0 to 144 kPa before reinforcement, the opening amount of transverse joint seams is increased continuously, while the strain of the circumferential bolt of #2 joint is increased rapidly to the plateau region. However, there is no obvious change in the opening amount of outer joint seams and bolt strain at a certain stage. When the top load $q_1$ is 450 kPa, the bonding interface of the 2# joint seam is cracked; and the opening of outer joint seams and bolt strain are enlarged with the load increase. When the top load $q_1$ is 625 kPa, the concrete on the outer camber is debonded and cracked in large areas. With the decreasing opening amount of the outer joint seam and the dramatic increase of bolt strain, the joint gradually loses the load-carrying property.

By analyzing the figure b, regarding the original structure, in the stage of the top load $q_1$ ranging from 0 to 192 kPa, the opening amount of 5# outer joint is increased continuously; also, the strain of the circumferential bolt of #5 joint is increased rapidly. When the top load $q_1$ is ranged from 192 to 444 kPa, circumferential bolts with enlarged strain gradually reach the plateau region due to the extruded joints inside. However, there is no obvious change in the opening amount of outer joints. When the top load $q_1$ is 445 kPa, two oblique cracks are generated on the camber concrete inside the 5# joint as well as increasingly deepened and widened with the load increase. And the bolt strain is abruptly changed and decreased remarkably under the shearing action. In this regard, the 5# joint gradually loses the load-carrying property. Regarding the reinforcement structure, when the top load $q_1$ is ranged from 0 to 164 kPa before reinforcement, the opening amount of the 5# outer joint is increased, while the circumferential bolt strain is also increased linearly. In the stage of the top load $q_1$ ranging from 164 kPa to 462 kPa after reinforcement, the stiffness of the joint after reinforcement is obviously lifted. But no obvious change can be witnessed on the opening amount of outer joint and bolt strain. In the stage of the top load $q_1$ ranging from 464 to 682 kPa, the bonding interface of the 5# joint is cracked and debonded, and adjoining blocks are staggered and slipped. Although the opening amount of the outer join is increased gradually, the bolt strain is increased slowly. When the top load $q_1$ is 684 kPa after reinforcement, affected by the extruded cracks of the outer camber at the 1#, 2#, and 6# joint seams, the stiffness of the structure is reduced and deformed severely. At the same time,
the bolt strain is increased rapidly, while the #5 joint is staggered inwards with the decreased opening amount of the outer joint seam. Finally, the structure is failed in the forms of crushed concrete outside of the 1# joint, debonding failure of the reinforcement interface and the sheared and bent steel plate. At this time, the 5# joint still has certain load-carrying property.

3.4. Debonding steel plates and concrete

The debonding of the reinforcement structure is shown in figure 11. As can be seen from the figure, the reinforcement interface is debonded under the joint action of shear force and radial force. It should be noted that the debonding failure typically occurs in the joint seams and can be divided into opening debonding and staggered debonding. In the stage of the top load q1 ranging from 342 to 505kPa, the increase of the shear stress and the radial stress at the joint interface seams lead to the cracked and debonded between the steel plates and the joint seams. Gradually, the joint seam staggering and opening are developed in the process of the bonding interface of the joint seam losing the bonding property, resulting in deepened degree of debonding. Debonding at 2# and 3# joints as well as at 4# and 5# joints are characterized by staggered debonding between the steel plate and the concrete as well as opened debonding, respectively. When the top load q1 is ranged from 505 to 789kPa, the elliptical degree of the structure is continuously increased with the load increase. Inner and outer cabmers of the joint seam are broken in large quantity under the shearing and extrusion action, severely weakening the overall stiffness of the structure. At last, the structure failure occurs in the reinforcement interface due to shear and extrusion with the crushed concrete at the camber of the 1# joint, the failure of the reinforcement interface, inwards staggered of B2 blocks in the middle ring and the bent steel plate.

4. Analysis of the reinforcement effect based on bearing capacity and deformation

The ultimate bearing capacities of the original structure and the reinforced structure are compared in Table 8. It can be observed from the table that the ultimate bearing capacities of the original structure and the bonded steel plate are 427.7 kPa and 788.9 kPa, respectively, improved by about 49.5%. It indicates that the improvement of the bearing capacity of the tunnel structure reinforced by the bonded steel plates is apparent under the vertical convergence of 15‰D.

In comparison to the unreinforced structure, the decreased percentage of the deformation of the tunnel structure reinforced by the bonded steel plate is regarded as a standard for evaluating the reinforcement efficiency.

| Test condition | Reinforcement form                      | Ultimate bearing capacity | Improvement of reinforcement |
|----------------|-----------------------------------------|---------------------------|------------------------------|
| S0             | Unreinforced                            | 527.7kPa                  | -                            |
| S1             | Reinforcement by bonded steel plates    | 788.9kPa                  | 49.5%                        |
Where: $\eta$ is the reinforcing efficiency (%) of the bonded steel plate; $\Delta D$ is the deformation of the unreinforced structure under a specific load condition; $\Delta D'$ is the deformation of the unreinforced structure under a specific load condition; $v$ and $t$ represent vertical and transverse directions, respectively, and $D$ is the outer diameter of the tunnel.

The diameter deformation rate $\Delta D / D$ of the tunnel is increased continuously with the load increase. The relationship between the reinforcement efficiency $\mu$ of the bonded steel plate and the diameter deformation rate $\Delta D / D$ of the tunnel.

The relationship between the reinforcement efficiency and the developed diameter deformation rate of the structure is shown in figure 12. The general trend is that the reinforcement efficiency is increased with the increase of the rate of diameter change. In terms of the reinforcement effect of vertical deformation, the reinforcement efficiency is enlarged with the increase of change rate of diameter when $\Delta D_v / D$ is ranged from 13.5 to 29.1‰. Also, the acceleration of the reinforcement efficiency is increased with the increased deformation rate of the vertical diameter. However, the reinforcement efficiency is decreased with the increase of the rate of diameter change due to the generation and development of structure failure. The maximum vertical reinforcement efficiency at this stage is 35.3%. When $\Delta D_v / D$ is ranged from 29.1 to 45.3‰, the structure gradually reaches the ultimate state because of the weakened rigidity of the overall reinforcement structure and the loss of the load-carrying property of the joint. As a result, the reinforcement efficiency is hardly increased and changed in fluctuation. The maximum vertical reinforcement efficiency is 38.5% at this stage when $\Delta D_v / D$ is 45.1‰. In terms of the reinforcement effect of transverse deformation, when $\Delta D_t / D$ is ranged from 12.2 to 26.3‰, the reinforcement efficiency is increased with the increase of the transverse diameter deformation rate. And the reinforcement efficiency is increased with the increased rate of diameter deformation affected by the generation and development of structural failure. The maximum transverse reinforcement efficiency at this stage is 28.7% when $\Delta D_t / D$ is 26.3‰; In the stage of $\Delta D_t / D$ ranging from 26.3 to 33.6‰, the overall stiffness of the structure is decreased due to the cracked bonding interface of the joint seam, while the reinforcement efficiency of transverse deformation is unchanged at this stage. In the stage of $\Delta D_t / D$ ranging from 33.6 to 40.1‰, the efficiency of the transverse reinforcement is increased since the failure of the original structure is found at the left haunch; and the failure of the reinforcement structure is the vault. It indicates that the reinforcement effect of transverse deformation is lifted significantly. The maximum transverse reinforcement efficiency is 30.2% when $\Delta D_t / D$ is 40.1‰.
5. Conclusion

Two sets of laboratory model tests based on the similarity ratio of 1:5 were performed in this paper against the staggered-jointed assembling shield tunnels at 45° in the sludge stratum, so as to explore the deformation behaviours and the deformation process of the original structure and the structure reinforced by the bonded steel plate under the deformation of 15‰D. After comparing both reinforcement effects, the following conclusions can be made:

1) The bearing capacity of the tunnel structure can be enhanced to 49.5% through reinforcing the staggered-jointed assembling shield tunnel with the bonded steel plates under large deformation. Reinforcing the steel plate can effectively prevent the tunnel from deformation. The maximum reinforcement efficiencies for vertical deformation and transverse deformation are 38.5% and 30.2%, respectively, indicating that the reinforcement with the bonded steel plate may be an effective improvement measure for the excessively deformed staggered-jointed assembling shield tunnel.

2) The failure modes for the unreinforced structure and the reinforced structure under overload conditions are shearing and extrusion failure of oblique section as well as reinforcement interface debonding and steel plate bending failure, respectively. The reinforcement interface is debonded under the joint action of shear force and radial force. It should be noted that the debonding failure typically occurs in the joint seams and can be divided into opening debonding and staggered debonding. At the same time, the reinforcement bar that is not yielded during the structure failure, is part of the overreinforced failure. Hence, it is suggested to improve the safety stock of the structure by means of improving the anti-pull and shear-resistance properties of the bonded interface as well as the strength grade of concrete in the similar designs.

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