Study on the deformation and fracture laws of surrounding rock of a TBM tunnel in a deep composite stratum

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Abstract. To solve the key problems of tunneling and excavation, deformation and rupture of surrounding rocks in TBM tunnel model test research in deep composite stratum was conducted. This research employed a combined strategy of physical model test and numerical simulation for studying the deformation and fracture laws of the surrounding rock in a vertical section of a TBM tunnel in deep composite strata. In this study, the main research results are 1) The “soft and hard unevenness” and “combination effect” of the composite stratum affected the overall bearing capacity of the tunnel resulting in failure at a shallower buried depth or a lower stress concentration factor. 2) When the model was only excavated and unloaded, the plastic zone was basically near the periphery of the tunnel, resulting mainly in shear failure. In the lower layer of the composite stratum tunnel, the plastic zone due to its higher strength parameters was smaller than that in the upper layer. 3) Under the premise of the axial loading and surrounding constraints, the deformation and failure mode of the TBM tunnel in the deep composite strata exhibited “X”-type failure characteristics. The vertical section of the partially excavated rock mass revealed that the rock mass at the top layer of the tunnel caused a sudden and integral shear sliding of the palm face along the oblique direction upward 50°. This research provides significant and important guidelines for solving the problems of safety in TBM tunnel construction in a deep composite stratum.

1. Introduction

The prevalent problem of TBM construction safety in deep composite strata is the instability of the surrounding rock caused by large squeezing deformation. For the construction of TBM tunnels in deep composite strata, the outstanding problems are the instability of the surrounding inhomogeneous rock caused by its large squeezing deformation and collapse of the palm face, which, in turn, results in jamming of TBM machines, equipment damage, and other accidents. In previous researches, Chen [1] studied the mechanism of large deformation of surrounding rock squeezing, advanced prediction methods, monitoring identification indicators in TBM tunnels and summarized the commonly used disposal methods for squeezing strata. Zhang [2] applied the block theory to analyze the stability of surrounding rock, and he realized that the geological occurrence characteristics of the composite rock...
mass and the disturbance of TBM eventually led to the instability of the surrounding rock. The studies by Zhou [3] revealed that soft rock formations were more susceptible to causing jamming machine tools during TBM construction than hard formations. Yu [4] measured the internal deformation process of the surrounding rock of the model tunnel through two-dimensional digital photography and developed a time–space evolution law of the internal deformation of the surrounding rock of the deep tunnel, which provided a basis for the prevention of accidents in the deep tunnel. Progressively, Hasanpour [5] analyzed the jamming accident of TBM machines and applied numerical calculation software to predict the TBM thrust required for excavation under poor geological conditions. Furthermore, Galvan [6] studied the effect of residual axial load on segment stability after TBM propulsion using a finite element model. The study revealed that the axial load produced a coupling effect between segment rings and adversely affects the segment stability. Bayati [7] employed local grouting and reinforcement methods to provide a certain reference experience and strategy for the TBM across the fault fracture zone. Also, Qin [8] adopted chemical grouting for strengthening the initial support to effectively solve the problem of the TBM card machine and make the TBM trouble-free. Zhai [9] calculated the wear rule of the double shield TBM in different positions of the Shenzhen hard rock region and the principle of the hob inspection and replacement of the maximum wear of the front hob as a tool-replacement control parameter based on the Shenzhen rail transit line (10 Yan–Ya tunnel double shield TBM project). These calculations provided suggestions for future arrangement and replacement of hobs in hard rock formations. According to the characteristics of alternating hard and soft rock masses of the rock face in the TBM excavation process, Shi [10] employed several model tests and numerical simulation methods to study the dynamic response law of the surrounding rock of the tunnel during the TBM excavation of the composite stratum. These analyses serve as a relevant reference and provide significant guidelines for directing TBM tunnel construction in a deep composite stratum.

In sum, the existing research focused on theoretical analysis and numerical calculation. However, in experimental research, there are limited physical model tests to study the stability of the TBM tunnel surrounding rock which is affected by the loading conditions, model size, and excavation methods. Also, during the construction of the TBM tunnel, the size of the deformation and fracture range of the surrounding rock of the vertical section largely influences its stability and the quantitative design of a support. Considering this, on the premise of satisfying the similarity theory, this paper designed an experimental system, including TBM road header simulator and model box, to study the deformation law of the surrounding rock in the deep TBM tunnel’s vertical section and conducted related experiments and numerical simulations. The research results provide important and significant research guidelines for solving the safety problems of TBM tunnel construction in a deep composite stratum.

2. Engineering background

The tunnel depth of the test was 700 m with a circular cross-section of 7 m diameter. A medium-sized TBM with a diameter of 7 m was used for the excavation process. The sample area was a deep composite stratum composed of mudstone and argillaceous siltstone in Si Chuan, China. The mechanical properties of the rock are presented in Table 1.

| Lithology     | Compressive strength (MPa) | Elastic modulus (MPa) | Poisson’s ratio | Bulk weight (10^4 N/m³) |
|---------------|-----------------------------|-----------------------|-----------------|-------------------------|
| mudstone      | 14.3                        | 2240                  | 0.23            | 2.55                    |
| siltstone     | 26                          | 2500                  | 0.24            | 2.53                    |

*The rock parameters were selected from the 1991 edition of the Manual of Rock Mechanics Parameters.

3. Experiment design

3.1. The similarity relationship satisfied by the experiment
The simulation test was based on the tunnel and model sizes, whereas it was required that the simulation range be at least three times larger than the excavation space. A large-scale similar model was utilized to determine the geometric similarity ratio as $C_L = 50$ and the bulk weight similarity ratio $Cr = 1.4$. According to the similarity theory, the similar criteria that satisfied the requirements of this test were deduced as follows:

$$C_c = C_E = C_o = C_R = C_pC_L = 70$$  (1)

From the expression, the elastic modulus, compressive and tensile strengths of the model material were 1/70 of the prototype material. The lithology indicators of the model are presented in Table 2.

### Table 2. Model rock mechanical parameters

| Lithology  | Compressive strength (MPa) | Elastic modulus (MPa) | Poisson’s ratio | Bulk weight (10^4 N/m^3) |
|------------|----------------------------|-----------------------|-----------------|-------------------------|
| mudstone   | 0.21                       | 32                    | 0.23            | 2.5                     |
| siltstone  | 0.34                       | 36                    | 0.24            | 2.5                     |

3.2. Selection of similar materials and model making

In the test, a mixture of sand and paraffin was utilized to simulate the composite stratum. The ratio of 100:2 and 100:2.5 were employed to simulate the composite stratum composed of mudstone and argillaceous siltstone. A ratio of 100:2 was employed to simulate the composite stratum composed of mudstone and argillaceous siltstone and a ratio of 100:2.5 was used to simulate a homogeneous formation consisting of argillaceous siltstone. The basic mechanical parameters of the sand and paraffin composite formation are presented in Table 3 [11].

### Table 3. Basic mechanical parameters of sand and paraffin mixed strata

| Sequence number | Compressive strength (MPa) | Elastic modulus (MPa) | Poisson’s ratio | Friction (°) | Cohesion (kPa) |
|-----------------|----------------------------|-----------------------|-----------------|-------------|----------------|
| Upper strata (100:2) | 0.18                       | 32                    | 0.29            | 40.25       | 13.91          |
| Lower strata (100:2.5) | 0.31                       | 35.8                  | 0.35            | 49.3        | 23.48          |

The specific steps of the model making were as follows:

1. A mixture ratio of 100:2 or 100:2.5 of sand and paraffin was placed in an oven and heated to about 140 °C.
2. When the paraffin was fully melted, the sand and paraffin were thoroughly mixed. Then the 100:2.5 mixture was poured layer by layer and compacted to the middle line of the tunnel. Similarly, the same method was also adopted to cast and compact the mixture of 100:2 to the design height.
3. The model was maintained under natural conditions for 2–3 days after completion.

3.3. Design of Roadheader Simulator and Model Box

3.3.1. Roadheader Simulator.

The existing TBM simulator was too large and the structure too complicated, which did not meet the requirements of small-scale tests. According to the load size of the test loading system and the size of the model box, the “boring experimental device” [12] was specially designed. The main structures of the test system [13] include:

1. Rack stabilizer: The primary component was a frame with an affixed rail consisting of a boring device.
2. Driving device: This component comprised a power unit, drill pipe, cutter head, and a spiral blade. The roadheader was driven by a motor and the tunneling speed was adjusted by a deceleration switch. The drill pipe was adjustable according to the testing duration. The hollow drill pipe had a cutter head fixed at the front end while the axially extended back end was affixed with spiral blades.
The output shaft, drill pipe, and cutter head were coupled on the same axis. The experimental device structure simulation is presented in Figure 1.

**Figure 1.** Schematic diagram of the model experimental device of TBM

3.3.2. The design of the Model Box.
To observe the deformation and fracture laws of the surrounding rock in vertical section of the TBM tunnel in deep composite stratum, the following specifications which are presented in Table 4 and Figure 2 were adopted for the model box and its steel frame considering the combined friction effects of the tunnel’s symmetry of the tunnel and glass plate on the model material. The glass box comprised 5 pieces of 15 mm thick transparent Plexiglas bonded with glass adhesive. The front glass plate was reserved for Φ80 mm excavation holes. The outer steel frame included 8 Φ10 mm and 4 pieces of 15 mm thick steel plates. Among them, the right steel plate was designed as a combined type. The middle “one-piece steel frame” was designed to be easily dismantled and installed to ensure that the observable area of the right glass plate was sufficiently large under low load, as presented in Figure 2(d).

**Table 4.** Specifications of the model box and its steel frame

| Specimen specifications (mm) | Model box specifications (mm) | Steel frame specifications (mm) |
|------------------------------|-------------------------------|---------------------------------|
| 190 × 200 × 300              | physical dimension: 220 × 230 × 360 | physical dimension: 290 × 260 × 330 |
|                              | Front and rear panels: 220(L) × 375(H) | Front and rear panels: 290(L) × 290(H) |
|                              | Front panel opening Φ80 | See Figure 2(b) (c) |
|                              | Bottom plate: 230(L) × 220(H) | Side panel: 270(L) × 330(H), See Figure 2(d) (e) |
|                              | Left- and right-side panels: 230(L) × 375(H) | |
(a) Glass box

(b) Front panel

(c) Back panel

(d) Right panel
3.4. Tunnel excavation and loading design
Upon completion of the tunnel model, the tunnel excavation loading test was conducted. The specific steps were:

1. Test preparation: The constant loading pressure on the top surface of the model was ensured to be 10 kN before starting the test, the camera and photography lights were set afterward.

2. Tunnel excavation: The similarity ratio between the prototype and the model was 7:0.7. The selected TBM daily footage was 50 m/day while the TBM’s pure tunneling footage was observed to be 8.3 m/h. Therefore, it can be concluded that the excavation process was performed once in 5 min.

3. Tunnel loading: The loading method was increased at 3 kN per each load level. The tunneling duration was 15 min at a loading rate of 10 N/s until the tunnel became unstable and failed. During loading, a computer-controlled digital camera was used for an automatic image collection at a collection frequency of 10 s.

3.5. Digital photography measurement technology
After the test, PhotoInfor software was used to convert (RAW to BMP) the collected images and analyzed them. The measuring point grid was divided into elements using ANSYS and imported into PostViewer [14]. The short excavation time had little effect on the deformation of the surrounding rock. This research only analyzed the deformation graph of the surrounding rock during loading.

4. Deformation and fracture laws of the surrounding rock in the vertical section of the TBM tunnel in the deep composite stratum
After the model was excavated to 2/3 of the total length, the top of the tunnel was loaded at a rate of 10 N/s, as presented in Figure 3. There were four stages from loading to failure, and the ultimate bearing capacity of the surrounding rock was obtained to be 0.54 MPa.

![Figure 3](image-url)

**Figure 3.** Variation curve of model top load with test time

The main test images are presented in Figure 4:
Figure 4. The main experimental images

(1) Vertical displacement

The simulation of the vertical displacement of the model is presented in Figure 5. Since only the top layer of the model was loaded, the vertical displacement of the rock mass mainly occurred in the rock at that layer of the tunnel. It is evident from the figure that when the top load was less than 0.47 MPa, the change in vertical displacement mainly extended from the right region above the vault to the upper right zone of the model. When the top load was 0.52 MPa, the vertical displacement of the rock mass at the top layer of the tunnel started increasing significantly with the maximum value appearing at that layer of the tunnel, and the rock mass at the top layer of the tunnel resulted in the tunnel face to shear along an oblique direction of about 45° upward. When the top load increased to 0.54 MPa, the rock mass at the top layer collapses and the vertical displacement above the middle line of the tunnel surface also changed considerably, indicating a rock block bulge occurrence in front of the tunnel surface. Furthermore, the vertical displacement of the rock mass on the same horizontal line at the top of the tunnel was basically the same under different loads, which indicated that the rock mass at the top layer of the tunnel can maintain high deformation consistency along the longitudinal direction under different loads.
Figure 5. The vertical displacement cloud of the model

(2) Maximum shear strain

Figure 6 presents the maximum shear strain cloud simulation of the model. The load increase on the top layer of the model only affected the maximum shear strain of the rock mass on that layer of the tunnel, while its influence on other rock masses was negligible. It is evident from Figure 6(c) that at a top load of 0.52 MPa, the maximum shear strain of the surrounding rock was concentrated at the junction of the vault and tunnel face, indicating that the rock mass at the top layer of the tunnel began to shear along an oblique arc direction of about 50° upward into the tunnel from the hance. Figure 6(d) revealed that the shear sliding of the top rock mass triggered the stress concentration at the corner, which caused the collapse and spalling of a section of the rock mass. Besides, the maximum shear strain of the rock mass on the same horizontal line at the top of the tunnel was basically the same under different loads, which indicated that the rock mass at the top of the tunnel can maintain a high fracture consistency along the longitudinal direction under different loads.

Figure 6. The maximum shear strain cloud of the model

(3) Quantitative analysis of the displacement evolution of the surrounding rock

Three measuring points were selected from the analysis images for the quantitative analysis of the total displacement of the measuring points: top measuring point, center of the tunnel, and top of the tunnel face, as presented in Figure 7. Before the top layer load was 0.52 MPa, the displacement of the above three points gradually increased, though the increase was imperceptible. Beyond 0.52 MPa, the displacement at the top of the tunnel and top of the tunnel face undergone an apparent differentiation and growth, while the displacement at the center of the tunnel face was relatively lagging. This
indicates that a sudden shear slip of the top rock mass resulted in a stress concentration at the top of the tunnel surface and the junction of the tunnel, which, in turn, caused the rock in the front region of the tunnel surface to rupture and the top rock mass to collapsed and spalled, resulting in the ultimate failure of the surrounding rock. When the surrounding rock was finally destroyed (0.54 MPa), the total displacements of the center of the tunnel, top measuring point, and top of the tunnel face of the surrounding rock were 27.561, 30.279, and 38.11 mm, respectively.

Figure 7. The top load–displacement curve of the model

(4) Description of composite strata influence

Through the deformation range and mode analysis, combined with the vertical displacement cloud images of different models (0.54 MPa) in Figure 8, the results presented are as follows:

1) Deformation range: when the top load was 0.54 MPa, the deformation range of Figure 8(c) was smaller than Figure 8(a) and (b). Considering the load variation at the top layer of the model as the change in the buried depth or the stress concentration factor around the dynamic pressure tunnel, Figure 8 presented that failure occurred at a shallower buried depth or a smaller stress concentration factor. Compared with the ordinary stratum, the vertical, horizontal, and total displacements of the composite stratum were concentrated in the upper softer stratum.

2) Deformation mode: the “X” type of damage was more obvious in Figure 8 (a) and (b), which was related to the combination effect. The uneven softness and hardness of the composite stratum led to the poor integrity of the surrounding rock, and the horizontal stress shearing effect was pronounced.
Figure 8. Vertical displacement cloud images of different models (0.54 MPa) (a) vertical section-composite strata model (b) cross section-composite strata model (c) cross section-homogeneous strata model

5. Study of the numerical simulation of deformation and fracture laws of the surrounding rock in vertical section of the TBM tunnel in the deep composite stratum

5.1. Numerical modeling
Tunnel excavation diameter \( D \) was 7 m, X and Z ranges were 110 m, Y (excavation direction) range was 78 m, and the depth was 700 m. The model included surrounding rocks and shield shells, with a total of 67392 model elements and 66597 nodes. The compound strata calculation model is presented in Figure 9. The boundary condition of the model was the normal displacement constraint of six planes. The tunnel adopted the elastic body model and the constitutive relationship of the model adopted the Mohr–Coulomb model. The thrust of the cutter head on the palm surface was 17000 kN which was applied on the palm surface as a uniform load during the calculation. The pressure generated by the weight of the nose section on the surrounding rock was 2 MPa and the action range was 60° around the lower section of the shield. The TBM excavation parameters are presented in Table 5 and the physical and mechanical parameters of its shield structure are presented in Table 6.

| Table 5. TBM excavation parameters |
| Radius R (m) | Shield length L (m) | Overdigging \( \Delta R \) (cm) | TBM weight (MPa) | Depth h (m) |
| --- | --- | --- | --- | --- |
| 3.5 | 8 | 10 | 2 | 700 |

| Table 6. Physical and mechanical parameters of the shield tunnel lining structure with TBM |
| --- | --- | --- |
| Elastic modulus \( E \) (GPa) | Poisson’s ratio \( \mu \) | thickness (cm) |
| Shield shell | 212 | 0.2 | 6 |

Figure 9. Model diagram

5.2. Numerical simulation results and analysis
The plastic zone diagram of the tunnel model is presented in Figure 10. The slices were the vertical section and the front side of the tunnel respectively. It can be observed that the simulation results were basically consistent with the experimental values, indicating a high similarity, while the deformation and failure of the surrounding rock were presented as “X” type failure characteristics.
It is evident from Figure 10(a) that when the model was excavated but unloaded, the range of the plastic zone was basically near the perimeter of the tunnel with shear failure. In the lower strata, due to its higher strength parameters, the plastic zone was smaller than that in the upper strata. Similarly, in Figure 10(b) it was observed that when the top layer load was 27 MPa, the plastic zone of the upper strata was obviously larger than that of the lower strata. Due to the “hard and soft unevenness,” the upper strata were more severely damaged primarily due to shear failure. A tensile shear failure occurred at the upper strata, while the shear failure zone was developed along the arc of the arch waist approximately 50° toward the top of the tunnel, which indicates that the entire top rock mass slid into the tunnel integrally along the arch waist. Compared with the excavation without loading, the plastic zone range of the rock mass at the top of the tunnel and the top of the tunnel face suddenly increased, while the development of the plastic zone at the center of the tunnel face was lagging. This indicates that sudden shear slip of the top rock mass caused stress concentration at the top layer of the tunnel face and the junction of the tunnel, which in turn led to the rock in front of the tunnel surface to rupture and the top rock mass to collapsed and spalled, resulting in the ultimate failure of the surrounding rock.

Figure 10. Plastic zone cloud

5.3. Comparison of numerical simulation and model test results
The comparison between numerical simulation and model test results is presented in Table 7.
Table 7. Results comparison

| Numerical simulation and model test | Connection | Difference |
|-----------------------------------|------------|------------|
|                                   | The results of the numerical simulation were basically consistent with that of the model test; the results presented a high similarity. The deformation and fracture of the surrounding rock were all characterized by “X” shaped failure, and the entire top rock mass slid into the tunnel in an oblique direction along an arc of about 50° upward from the arch waist. | Numerical simulation calculation considered the influence of the cutter head thrust on the tunnel face and the influence of the weight of the machine head on the surrounding rock, which solved the problem that the model experiment could not consider the influence of excavation on the deformation of the surrounding rock. |

6. Conclusion

In this research, the method of combining physical experiment and numerical simulation was used to study the deformation and fracture laws of the surrounding rock in the vertical section of the TBM tunnel in deep composite stratum, the following main conclusions were obtained:

(1) Characteristics of the composite stratum: The “hard and soft unevenness” and “combination effect” of the composite stratum affected the overall bearing capacity of the tunnel, causing the tunnel to fail at a shallower buried depth or a small stress concentration factor.

(2) When the model was only excavated and unloaded, the plastic zone was basically near the periphery of the tunnel, resulting mainly in shear failure. In the lower section of the composite stratum tunnel, due to its higher strength parameters, the plastic zone was smaller than that in the upper region.

(3) Under the premise of axial loading and surrounding constraints, the deformation and failure mode of the TBM tunnel in deep composite strata exhibited “X”-type failure characteristics. The vertical section (partially excavated) rock mass revealed that the rock mass at the top of the tunnel caused a sudden and integral shear sliding of the palm face along the oblique direction of 50° upward.

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