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Study on the Ultimate Plastic Failure of Tunnels in Layered Rock Masses under High In-Situ Stress Based on Full-Section Conditions

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Abstract: The mechanism of rock mass failure at the face of high geostress tunnels and the ultra-strength support measures are among the most problems in tunnel engineering. Meanwhile, excavating tunnels in weak surrounding rocks is also a difficult issue, and scholars both domestically and internationally have conducted extensive research on problem. However, the stability of tunnels excavated in alternating soft and hard rock masses still needs systematic and in-depth analysis. The main reason is that in highostress areas, the dynamic response of alternating soft and hard rocks after tunnel excavation is different. The full-face method of construction can reduce the disturbance to the surrounding, which is beneficial for tunnel excavation under these conditions.

1. Introduction

The mechanism of rock mass failure at the face of high geostress tunnels and the corresponding ultra-strength support measures among the most challenging issues in tunnel engineering. Although numerous studies have been conducted by scholars both domestically and internationally, systematic and in-depth analyses are still needed for stability of tunnels excavated in alternating soft and hard rock formations. The main reason is that in high geostress areas, the dynamic response of alternating soft and rocks after tunnel excavation is different. Therefore, constructing tunnels in such geological conditions has been a long-standing concern in the industry. China has a vast territory with a proportion of mountainous areas, which cover about 2/3 of the country's total area. In recent years, the demand for infrastructure construction such as highways high-speed railways, water conservancy projects, and urban metros has increased, leading to a growing demand for tunnel construction. During this period, the construction of high railway projects has experienced unprecedented rapid development. During tunnel construction, when tunnels pass through high geostress areas, especially when there are weak and broken rock layers, complex terrain and geological conditions, combined with low accuracy in preliminary surveys, can lead to serious engineering accidents such as large deformation of the surrounding rock, rock fractures, landslides if inappropriate construction plans are chosen. These accidents not only increase the total cost of the project but also endanger the lives of construction workers[1,2,3,4]. Therefore, avoid such major safety accidents, it is necessary to conduct research on the stability of the surrounding rock and the face during tunnel excavation.

There are many factors that affect the stability of surrounding rock, including both the factors of rock structure and external factors, which are the result of the combined reaction of various factors[5]. In engineering practice, it is generally believed that the main factors affecting the stability of rock mass geological factors and non-geological factors. Among them, geological factors mainly include the structure of soil and rock, lithology, initial stress conditions of surrounding rock the action of groundwater in the construction area, and regional geological structure. Non-geological factors mainly include the design parameters before construction and the impact of later constructionLi et al[6,7,8,9,10]. (2015) ranked the factors affecting the stability of tunnel surrounding rock according to their influence, and the results showed that the grade surrounding rock is the main factor affecting the stability of tunnel surrounding rock, followed by the terrain and geological conditions of the tunnel area, the initial stress conditions of the mass, the characteristics of the rock mass, the geological structure, the action of groundwater, and the later construction disturbance. In recent years, many scholars at home abroad have discussed the various factors that affect the stability of tunnel surrounding rock[11].

Currently, some scholars have proposed that the geostress characteristics and rock mass strength of rock mass have an important impact on its stability. At the same time, experts and scholars have also proposed the factor of tunnel section shape. However, due to diverse properties of soil and rock, it is impossible to quantify the impact of each factor in a specific project. Qin et al[12,13,14]. (2021)ensively considered the tunnel section shape, the elastic-plastic state of the tunnel surrounding rock, and the tunnel span size, and used a combination of theoretical numerical simulation. By comparing the deformation of underground tunnel surrounding rock with different spans, they found that in circular tunnels where the surrounding rock is in an elastic-plastic state, stability of the surrounding rock decreases as the span increases[15]. Wang (2018) combined various common influencing factors and provided the following fuzzy empirical judgment[16].

Taking Yuanling Tunnel of the Xi'an-Shiyan high-speed railway as the research object, the geological sketch data at the site were analyzed to identify the distribution of geology; the rock mechanics parameters of representative soft and hard rocks were obtained through indoor rock mechanics tests, and the geostress was analyzed; according to the-site monitoring of surrounding rock deformation, the deformation characteristics of the tunnel surrounding rock were analyzed.

2. Rock Mechanics Testing

2.1 Test Methods

To facilitate testing at the tunnel construction site, rock samples were cut from the blasted rock blocks for. The detailed statistics of the test quantities are shown in Table 1.

Table 1. Test Quantity Statistics Table Test Group Number

Test group Numbering	Mileage	Rock type	Point load test/blocks	Uniaxial compressive strength test/blocks
1	D2K584+868	Mud-bearing limestone	8/9	4
2	D2K584+872	Limestone	13/15	4
3	D2K584+872	Intraclastic limestone	12/15	4
4	D2K854+887	Limestone	10/14	4
5	D2K854+890	Limestone	11/12	4
6	D2K584+900	Limestone	7/8	2
7	D2K854+904	Mud-bearing limestone	10/10	4
8	D2K854+904	Mud-bearing limestone	11/13	5

Note: The numbers before and after the "/" represent the number of valid trials and the total number of samples, respectively.

During the point load test (Fig 1), irregularly shaped block or cylindrical specimens (Fig 2) that meet the specification requirements were used. The prepared specimens were placed on the loading points of the point load instrument, and then the jack was used to apply continuous and uniform loading while controlling the time of specimen destruction, which was completed within 10s to 60s. After the specimen was destroyed, a vernier caliper was used to measure the average width W of the specimen in the vertical loading direction and the distance D between the two loading points. The test was considered valid only if the fracture surface extended through the entire rock sample between the two loading points; otherwise, it was discarded. Finally, the point load strength value of the specimen was calculated. In the uniaxial compressive strength test (Fig 3), the corresponding rock blocks from the point load test were cut into cylindrical specimens with a height-to-diameter ratio of 1:1 (5cm height, 5cm diameter) or 2:1 (10cm height, 5cm diameter) (Fig 4). Before the test, vaseline was applied to the top and bottom surfaces of the specimen, which was then placed at the center of the compression plate to avoid eccentricity-induced errors. The specimens were then loaded at a constant rate of 0.5MPa/s until failure, and the final failure load and the characteristics of the rock sample's failure were recorded.

Fig .1 Point Load Test Equipment

Fig.2 Point Load Test Specimen

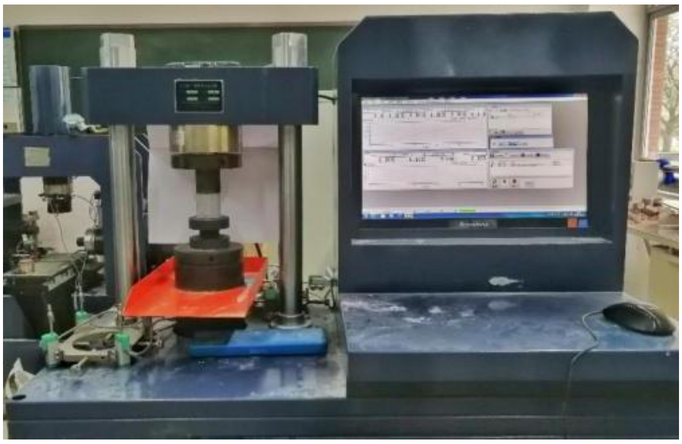


Fig.3 Uniaxial compressive strength testing machine

a Aspect ratio 1:1 *b Aspect ratio 2:1*

Fig.4 Cylindrical samples

2.2 Experimental Results

The rock types in the experimental area are mainly hard rocks, with saturated uniaxial compressive strengths ranging from30MPa to 60MPa, and saturated point load strengths ranging from 1.5MPa to 4MPa. In uniaxial saturated compression experiment, the experimental data showed good stability. However, by increasing the number of samples and adjusting the data using appropriate statistical methods, reliability of the point load test can be effectively improved. Finally, Table 3-2 shows the test results obtained through the above steps. Except for experimental group2 (Is(50)=4.01 MPa, Rc=46.38 MPa) which showed abnormal results, the point load and saturated uniaxial compressive strengths of the other 7 experimental groups showed a significant positive correlation.

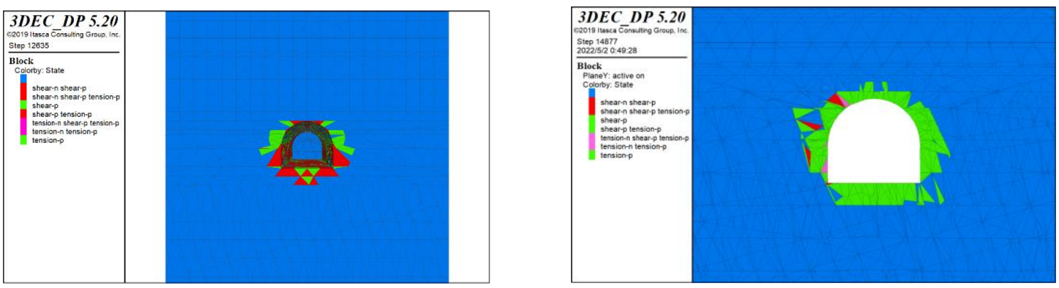
3. Tunnel Plasticity Calculation Model

3.1 Geological Parameters

The Yuanling tunnel is selected as the research object with a total length of 13117m and a maximum burial depth of about 2258m. The tunnel excavation height is 4.36m and the span is 16.80m. Some parts are high-stress areas, with geological formations of limestone and sandy. The limestone is in medium-thick layers, with a bedding plane attitude of 182°∠12°~18°. The rock relatively hard, slightly weathered, and the rock mass integrity is good, belonging to Class III surrounding rock. The joints and fissures are generally well-developed mainly with one set of joints, with an attitude of 178°~189°∠32°~45°, a spacing of1~6m, and an extension of 1~9m. The joint surfaces are slightly open, dry and rough, without seepage, and with calcite. On-site analysis shows that the surrounding rock is basically stable. In the sandy shale layer, two sets of large joints (J1 and J) are developed. The rock mass is more fragmented and the integrity is poor. The surrounding rock is rated as Class IV soft rock, with poor stability. The of J1 joint in sandy shale is 80°~100°∠60°~75°, and the attitude of J2 joint 345°~5°∠55°~70°, with a spacing of 0.2~0.4m. The sets of joints form an "X" shape (Figure 3-1). The bedding plane attitude of the rock layer in the calculation model is 18°∠15°, and the dip directions of the joints are 90°, 100°, and 180°, with angles of 65°, 75°, and 40°, and joint spacings of 4m, 4m, and3m. As shown in Figure 5-2, the size of the three-dimensional model is X×Y×Z=80m×80×80m, with a tunnel model excavation height of 12.23m, a span of 14.7m, and a length 80m.

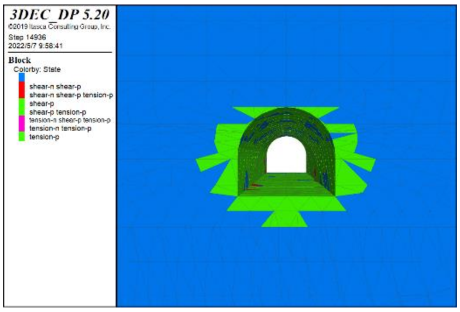
3.2 Plastic Zone Failure

Analysis The tunnel model is simulated for full-section cyclic excavation without support. As the tunnel is excavated some shear and tensile areas gradually appear on the tunnel body, mainly concentrated on both sides of the tunnel. It is found that when the tunnel is excavated 80m, the plastic failure zone of the surrounding rock is mainly concentrated in the latter half of the tunnel, i.e., the section passing through the shale formation. The closer to the face, the more obvious the plastic failure is. At a distance of 18m from the face, the characteristics of failure are quite significant (Fig 5), with some shear and tensile failure areas appearing on the left arch shoulder and arch waist.



a) Excavation advance 70m

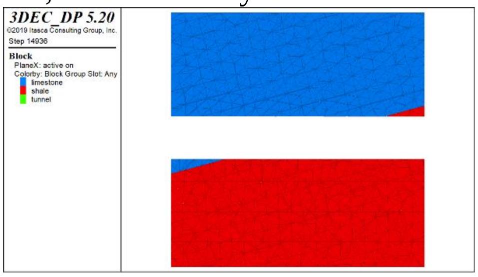
b) 18m away from the face

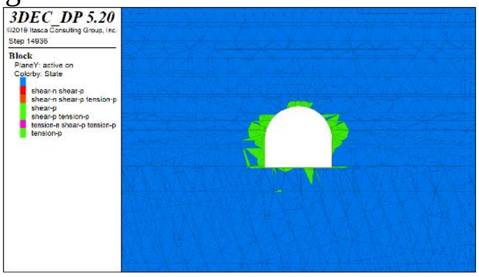


c) Excavation advance 80

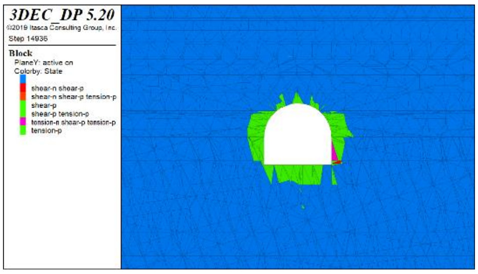
Fig.5 Characteristics of plastic failure in unsupported tunnel excavation

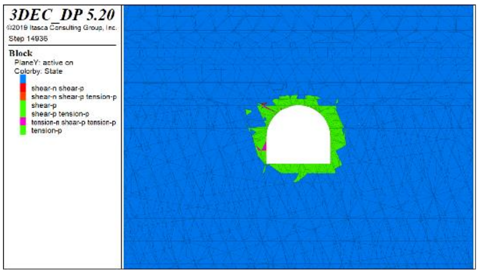
An analysis of the tunnel was conducted, with plastic zone analysis performed at tunnel sections 57 m 13 m, and 40 m away from the tunnel face Fig.6.





a)Excavation advance 80m tunnel surrounding rock characteristics b) 57m from the face





c) 40m the face *d) 13m from the face*

Fig.6 Plastic failure characteristics of tunnel surrounding rock interface

The analysis reveals that when the tunnel excavation reaches 80 meters, at a distance of 57 meters from the tunnel face, the tunnel passes through the limestone, and there is no obvious plastic deformation zone around the tunnel. At a distance of 40 meters from the tunnel face, the tunnel through both limestone and sandy shale, with some tensile zones appearing on the right side of the arch and some shear zones at the arch foot. At a distance 13 meters from the tunnel face, the tunnel completely passes through the sandy shale, with some shear zones appearing on the left side of the arch and some and tensile zones on the left side of the arch. It can be seen that when the tunnel excavation reaches the sandy shale layer, the area of plastic deformation the tunnel is more obvious than when it passes through the limestone.

4. Analysis of Full Section

Excavation with Tunnel Support From the previous discussion, it is known that when the model is excavated without support, a certain amount of displacement and plastic deformation occurs around the tunnel. To prevent further damage to the tunnel's surrounding rock, appropriate measures need to be implemented.

4.1 Selection of Support Material Properties

According to the People's Republic of China's "Technical Specifications for Anchor Rod Sprayed Support" (Jianbiao [2001] No. 158), for a tunnel with a span of 10m < ≤ 15m in Class III surrounding rock, 100~150mm thick steel mesh sprayed concrete should be set, with anchor rods ranging 3.0mm to 4.0m in length. For Class IV surrounding rock, 150~200mm thick steel mesh concrete should be set, with anchor rods ranging from 3.0mm to 4.0m in length. Considering these standards, a full section of200mm thick steel mesh sprayed concrete will be set, with a concrete strength grade of C25[17]. The tunnel is excavated in stages, with excavation advance being 5m. After each excavation, at 1m behind the working face, five anchor rods are installed at the arch top, arch shoulders and arch waist. The anchor rod length is 3m. Additionally, concrete lining is set on the tunnel walls. The material properties of the lining and the of the anchor rod structural units are as follows:

Table 2. Physical and Mechanical Parameters of Anchor Rods

Support Type	Elastic Modulus/GPa	Cross-sectional Area/mm ²	Length/m	Tensile Yield Strength/MPa	Cement Mortar Bond/MPa	Cement Mortar Bond Stiffness/MPa
Mortar Anchor Bar	200	380	3	500	0.24	17.5

Table 3. Physical and mechanical parameters of the lining

Support Type	Strength Grade	Elastic Modulus/GPa	Thickness/m	Cohesion /MPa	Tensile Strength /MPa	Density/(kg.m3)	Friction Angle/°	Poissons Ratio
Sprayed concrete	C25	23	0.2	2.5	1.3	2200	45	0.2

4.2 Displacement Characteristic

Analysis During the excavation process, the tunnel model was supported until the excavation was completed. By analyzing the displacement contour of the lining, it was found that in the latter half of the tunnel, where the surrounding rock is sandy shale, the displacement of the lining is larger, at the same time, the axial tension of the anchor rod is also larger (Fig 7).

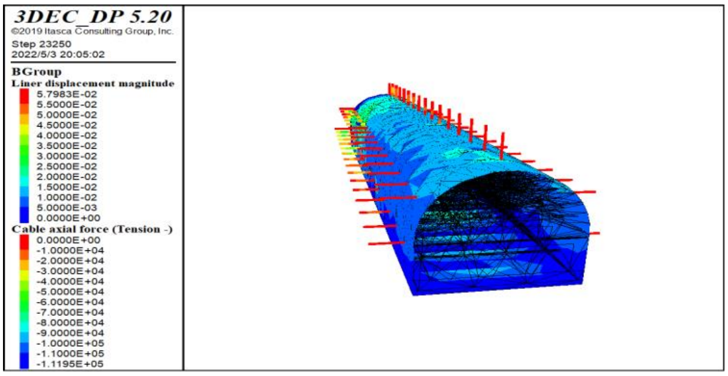
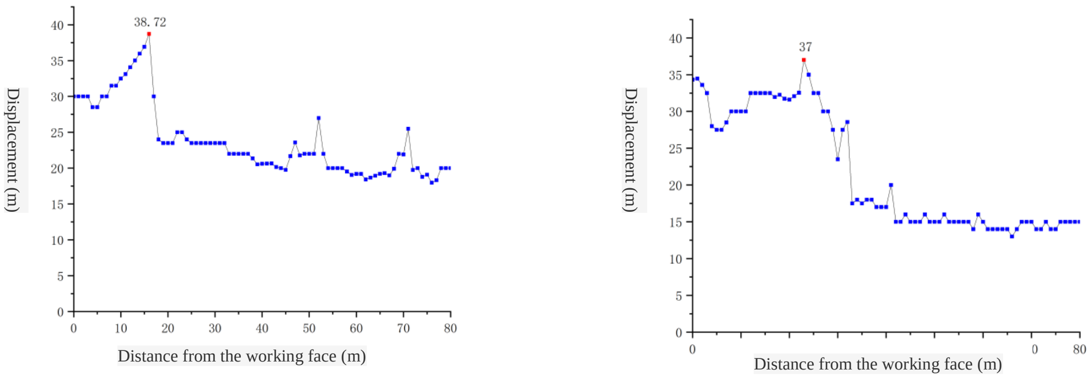
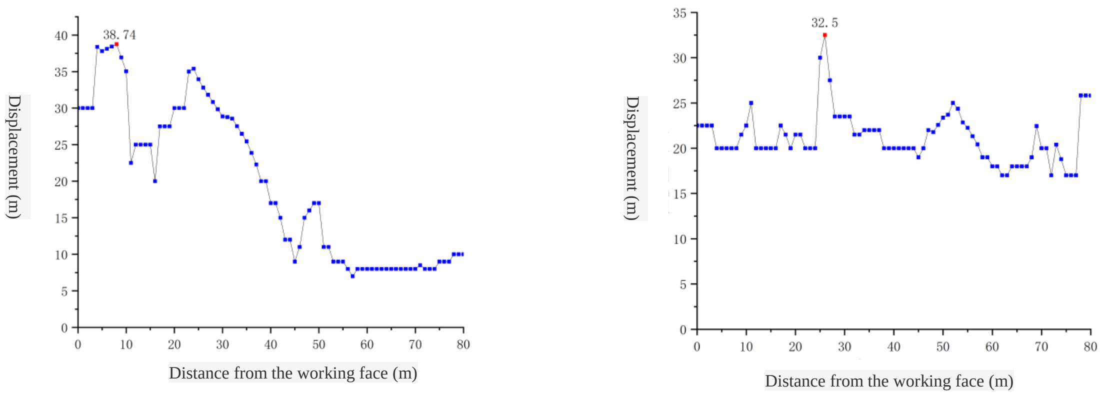


Fig.7 Characteristics of tunnel support



a)Maximum displacement of the arch top at different distances from the face

b) Maximum displacement of the arch shoulder at different distances from the face



c) Maximum displacement of the arch waist at different distances from the face

d) Maximum displacement of the invert at different distances from the face

Fig.8 Maximum displacements at various locations of the tunnel with supported excavation for 80 meters

4.3 Analysis of Plastic Zone

Failure After setting up the concrete lining and mortar-anchored support, as the tunnel excavation, parts of the tunnel body gradually develop shear and tensile regions, mainly concentrated on both sides of the tunnel. When the entire tunnel section is excavated, is observed and analyzed that the plastic failure zones in the surrounding rock are mainly concentrated in the first half of the tunnel, i.e., the section passing through the strata, and the further from the face, the more obvious the plastic failure. At a distance of 78 meters from the face, the shear failure are more significant, mainly located at the left and right arch waist of the tunnel; at a distance of 40 meters from the face, the tensile characteristics are more significant, mainly located at the right arch waist. When the tunnel is 57 meters away from the face, it completely passes through the limestone and at this time, there is no obvious plastic failure zone in the surrounding rock of the tunnel; when the tunnel is 13 meters away from the face it completely passes through the sandy shale, and at this time, there is no obvious plastic failure zone in the surrounding rock of the tunnel (Figure 8). By comparing the un-supported tunnel model with the tunnel model that has been set up with concrete lining and mortar-anchored support, it is found that the tunnel is excavated for 80 meters, in the un-supported tunnel model, the closer to the face, the more obvious the plastic failure of the rock, mainly concentrated in the second half of the tunnel, i.e., the section passing through the sandy shale; in the tunnel model that has been set up concrete lining and mortar-anchored support, the further from the face, the more obvious the plastic failure of the surrounding rock, mainly concentrated in the first half the tunnel, i.e., the section passing through the limestone. After the support is set up, the plastic failure zone at the interface between the limestone and sandy is significantly reduced.

5. Conclusion

The tunnel passes through multiple geological formations and various rock types, creating a complex geological environment. There are no obvious of active tectonics in the tunnel area, but due to the influence of terrain and other factors, several types of unfavorable geology have developed in the, which affect the construction of the project to varying degrees.

Under the full-face excavation method, without support, the closer the tunnel excavation section is the face, the more obvious the plastic deformation; after excavation, the plastic deformation of the surrounding rock is mainly concentrated in the sandy shale formation; during excavation, further away from the face, the greater the displacement of the arch top; the areas with larger displacement before and after support are all concentrated in the rear part of tunnel, i.e., the part passing through the sandy shale.

After support, the displacement and plastic deformation at various parts of the face are significantly.

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