Study on construction mechanics of in-situ expansion of ultra-large flat tunnel in collapse section of an existing tunnel

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Abstract. To meet the needs of the transportation, a tunnel in Fujian Province was expanded in situ to form an ultra-large flat tunnel. In some sections, a large-scale collapse has occurred during the original tunnel construction, which brings a huge risk for the expansion construction. Based on the above background, the theoretical formula is used to calculate the spatial distribution of the collapse, and the dynamic construction mechanical characteristics of the CRD method are studied through numerical analysis and field tests. The results show that: (1) curves of the surface settlement are presented small in both ends while big in the middle, and the settlement center appears above the left side of the tunnel centerline, and the surface settlement caused by the excavation of the upper bench is the largest; (2) the surrounding rock disturbance range is 0.5 times of the tunnel diameter in front and rear of the tunnel. The deformation of the left spandrel of the tunnel is the largest; (3) there are two "V" shaped shear plastic areas in the arch of the tunnel, which is deep into the surrounding rock. During the tunnel construction, the monitoring results of surface settlement, surrounding rock, and support system deformation are in good agreement with the numerical simulation results, which verifies the reliability of the numerical simulation. The research results can provide a reference for the further study of the stability of surrounding rock during the expansion construction of the ultra-flat section tunnel and provide some reference for similar projects in the future.

1. Introduction
With the development of China's economy, the traffic capacity of some existing highway tunnels tends to be saturated, which cannot meet the increasing traffic demand, so it is urgent to expand the tunnels.
For the tunnels limited by topography, in-situ expansion based on existing tunnels is the best scheme \(^1\)-\(^2\). During the in-situ expansion construction of the tunnel, the existing tunnel lining will be removed, and the surrounding rock around the expanded excavation will break the original "rock-lining" balance system and the stress redistribution of surrounding rock is complex \(^3\). Furthermore, some existing tunnels have suffered from large-scale collapse, water outburst and mud outburst during construction, as well as the deterioration of lining materials, cavity behind lining and other diseases, also other factors such as operation period, construction technique, climate, and environment, all of which bring huge safety risks to the expansion construction \(^4\)-\(^6\). The stability of surrounding rock is still an outstanding problem in complex situations such as an in-situ expansion of small section tunnel into ultra-large flat section tunnel.

At present, the research on the new four-lane super flat tunnel at home and abroad has been relatively mature \(^7\)-\(^10\), while the engineering cases of in-situ two expansion four tunnel are relatively few, some scholars have done some research on the in-situ expansion of tunnels and achieved some results. With the help of numerical simulation, Gao et al.\(^{[12]}\) comprehensively compared the structural stability, surrounding rock stress, deformation, and other characteristics of the four expansion forms, and concluded that one side expansion excavation is the best scheme. Based on the background of Yuzhou Tunnel and the characteristics of construction mechanics, Zhu et al.\(^{[13]}\) consider that one side expansion excavation is the best among the three expansion excavation methods. Lin et al.\(^{[14]}\) studied the stability of weak surrounding rock during the excavation process of the CD method through numerical simulation in Damaoshan Tunnel. Through the comparative analysis of numerical calculation and field tests, the rationality of the calculation results is proved, and the optimization suggestions of bolt support parameters are put forward. Based on the expansion project of Houci Tunnel, Sun, et al.\(^{[11]}\) established a three-dimensional finite element model, calculated and analyzed the ground settlement, vault settlement, tunnel surrounding displacement, and the stress change rule of arch foot and vault, and proposed the formula for the spatial change rule of displacement of the expansion tunnel. Some scholars also put forward the theoretical analysis method, which simplifies the cross-section of the side expansion tunnel into the problem of the double connected area by the equal generation circle method, and deduces the complex function expression of the stress and displacement of the surrounding rock by combining the complex function theory Schwarz Alternating method, to determine the optimal single expansion width. Wu et al.\(^{[16]}\) studied the width, stress, and displacement characteristics of the surrounding rock of the single excavation of the layer by the Peeling method through theoretical analysis, and verified the calculation accuracy by numerical simulation. Peng\(^{[3]}\) used the theoretical analysis method to solve the Peeling method and Bench method, obtained the displacement and stress distribution characteristics and rules of the surrounding rock of the in-situ expansion tunnel, and determined the single expansion width. The above research on in-situ expansion tunnel is mainly divided into two categories. The references \(^{[12]}\) and \(^{[13]}\) compare the expansion forms (new tunnel, in-situ expansion tunnel) and expansion methods (one side, two sides, surrounding expansion excavation). The research results suggest that one side expansion and in-situ expansion should be adopted, but the specific construction methods are not discussed in depth. The references \(^{[11]}\), \(^{[14]}\) ~ \(^{[17]}\) mainly analyze the displacement and mechanical characteristics of the expanded tunnel under the specific construction method, but there is still a gap in the research and application of the construction method of in-situ expansion tunnel.
crossing the existing collapse section. In this paper, based on a tunnel expansion project in Fujian Province, the dynamic construction mechanical characteristics of the CRD method are studied by numerical simulation and combined with the field monitoring data, it is proved that the method can meet the safety requirements of the expansion construction.

2. Engineering background
A tunnel in Fuzhou was built in 1987. It is a bidirectional four-lane highway tunnel with a span of 10 m and both tunnels are 970m long. During the construction, a large area of collapse occurred. After more than 20 years of operation, the existing tunnels are suffering from leakage, lining cracking, and other diseases, and the traffic capacity has been unable to meet the actual demand. Now it will be expanded into an eight-lane tunnel, with a clear distance of 22.16m between the two tunnels, a tunnel excavation span of 19.03m, an excavation height of 13.5m, and an excavation section area of 208.5m$^2$. The section layout is shown in Fig. 2.

![Fig.1 Longitudinal profile of the tunnel](image1)
![Fig.2 Diagram of the tunnel section (unit: m)](image2)

According to engineering geological exploration results, the grade V surrounding rock section (YK17 + 651 ~ YK17 + 827) of the right tunnel once suffered from such disasters as arch cracking and large-area tunnel collapse. The surrounding rock in this section is medium to slightly weathered tuff,
which is partially revealed by fragmentary strong weathering, and it is a loose structure of breccia and gravel, with a relatively thin surrounding rock on the arch. The construction risk of this section is great, which is the control section of the whole expansion project. Based on the grade V surrounding rock section of the right tunnel, this paper uses the theoretical formula to judge the spatial scope of the collapse body, uses the numerical calculation method to study the dynamic construction mechanics of the four-lane highway tunnel in situ expansion of the collapse section, and finally verify the numerical simulation results by field test.

3. The scope of collapse and the determination of its engineering characteristics
For the grade V surrounding rock section (YK17 + 651 ~ YK17 + 827) of the right tunnel passing through the existing collapse section, the overall shape and spatial distribution of the collapse mass are estimated by theoretical formula. Regarding the code for the design of road tunnels[20], the tunnel is determined as a shallow-buried tunnel and the simplified mechanical model of collapse is shown in Fig. 3.

In Eq 1, calculated friction angle of surrounding rock \( \phi_c \) is 40° and the friction angle of both sides of the roof geotechnical column \( \theta \) is 24°. The angle between the fracture surface and the horizon, that is, the fracture angle \( \beta \) is 77.9° and the width of the fracture surface is 27.24 m according to Eq 2.

\[
\beta = \arctan(\tan \phi + \sqrt{(\tan^2 \phi + 1) \tan \phi \tan \theta})
\]

\[
B = B_c + 2 \times (H + h) \times \tan(90° - \beta)
\]

where \( \phi_c \) is the calculated friction angle of surrounding rock; \( \theta \) is the friction angle of both sides of the roof geotechnical column; \( B \) is the width of fracture surface; \( B_c \) is the span of the tunnel; \( H \) is the depth of the tunnel; \( h \) is the height of the excavation;

![Fig.3 Simplified mechanical model of collapse](image)

4. Numerical calculation and analysis of four-lane highway tunnel expanded in situ in collapse section

4.1 Establishment of a three-dimensional numerical model
The FDM software FLAC$^{3D}$ is used for calculation. According to the engineering geological report and design data of a tunnel in Fujian Province, YK17 + 745 ~ YK17 + 815 with a buried depth of 26m is selected for the model. The overall size is 220m × 100m × 70m. Among them, the collapse section of the right tunnel is located at YK17 + 775 ~ 785, with a total length of 10m. The upper part of the model is taken as a free boundary, the bottom and sides are taken as normal constraint boundary, only considering the self-weight stress field, a total of 280770 solid elements, and 358494 element nodes. The 3D model is shown in Fig. 4 and Fig. 5.

The CRD method is adopted in the study section, and the circular footage of the tunnel is 2m. This paper mainly studies the dynamic construction mechanical characteristics of the right tunnel, without considering the influence of the excavation of the left tunnel on the right tunnel. The layout of tunnel excavation construction is shown in Fig. 6 and Fig. 7.

The construction process of the CRD method is as follows: (1) backfill at the bottom of the existing tunnel; (2) demolition of existing tunnel arch lining and expansion excavation of left upper heading; (3) construction of primary support, anchor bolt, temporary middle partition and temporary invert of left upper heading; (4) demolition of the lower lining of existing tunnel and expansion excavation of left lower heading; (5) construction of primary support, anchor bolt and temporary middle partition of left lower heading (6) expanding excavation of right upper heading (7) construction of initial support, anchor
bolt and temporary inverted arch of right upper heading (8) expanding excavation of right lower heading (9) construction of primary support and anchor bolt of right lower heading (10) removal of temporary middle partition and temporary inverted arch (11) construction of secondary lining.

4.2 Physical and mechanical parameters

According to the design data of this tunnel project, in the grade V surrounding rock section, the primary support consists of 30cm thick C25 shotcrete, an I22b steel frame with a longitudinal spacing of 60cm, 15cm × 15cm Φ8 steel mesh and 80cm × 60cm Φ25 hollow grouting bolt with a design length of 5m. The secondary lining is made of 70 cm thick concrete.

The surrounding rock is simulated by the solid element, and the constitutive model is the Mohr-Coulomb elastic-plastic model. Physical and mechanical parameters of surrounding rock are determined according to geological survey data and laboratory test results, as shown in Table 1, and the representative core of this section of drilling is shown in Fig. 8. The tunnel lining structure is simulated by solid elements, and the constitutive model is an isotropic elastic model. Among them, according to the principle of equivalent bending stiffness and Eq 3, the elastic modulus of steel arch is converted into the primary support. A total of 23240 bolts were simulated by cable elements. Support structure parameters are shown in Table 2.

Table 1. Physical and mechanical indexes of the surrounding rock

| Name            | Elastic modulus E(GPa) | Density ρ(kg/m³) | Poisson’s ratio ν | Cohesion C(MPa) | Friction angle φ (°) |
|-----------------|------------------------|------------------|-------------------|-----------------|----------------------|
| Surrounding rock| 0.9                    | 1860             | 0.40              | 0.20            | 23                   |
| Collapse strata | 0.3                    | 1225             | 0.48              | 0.05            | 15                   |
| Backfill        | 0.5                    | 1640             | 0.45              | 0.10            | 18                   |

\[ E_c = E_0 + \frac{I_c}{I_s} \]

Where \( E_c \) is the equivalent elastic modulus after conversion; \( E_0 \) is the elastic modulus of shotcrete; \( E_s \) is the elastic modulus of steel arch; \( I_c \) is the moment of inertia of shotcrete; \( I_s \) is the moment of inertia of steel arch.

Table 2. Physical and mechanical indexes of solid element support structure materials

| Name             | Elastic modulus E(GPa) | Density ρ(kg/m³) | Poisson’s ratio ν |
|------------------|------------------------|------------------|-------------------|
| Primary support  | 26.2                   | 2350             | 0.20              |
| Secondary lining | 26.2                   | 2450             | 0.20              |
4.3 Calculation results and analysis

To explore the construction mechanical characteristics of the in-situ expansion tunnel across the collapse section, two sections I-I' and II-II' are selected along the tunnel axis, of which section II-II' is located in the collapse section, as shown in Fig. 9.

![Diagram of critical sections and monitoring points of the tunnel](image)

**Fig. 9** Diagram of critical sections and monitoring points of the tunnel (unit: m)

4.3.1 Surface settlement analysis. For shallow tunnels, the surface settlement value and its development trend can be used as an important basis to judge the stability of surrounding rock. Fig. 10 is the surface settlement curve of sections I-I' and II-II'.

![Surface settlement curve of each section of the tunnel](image)

**Fig. 10** Surface settlement curve of each section of the tunnel
As shown in Fig. 10, the settlement curve of each section presents the form of "single peak" with small in both ends while big in the middle, and the maximum settlement is mainly concentrated above the central axis of the expansion tunnel, approximately in line with the normal distribution. Due to the asymmetry of the construction, the settlement curve is not completely symmetrical, and the maximum settlement is shifted with the excavation of the construction section. When the upper left bench of the tunnel reaches the critical section, the maximum settlement is to the left of the central axis. When the upper right bench of the tunnel reaches the critical section, the maximum settlement point begins to move to the right side of the expansion tunnel axis. For section I-I’, when the tunnel excavation is completed, the maximum settlement will move to the axis of the extended tunnel. However, the maximum settlement of section II-II’ is located on the left side of the axis of the expanded tunnel. This is because section II-II’ is located in the collapse section, and the collapse body is located directly above the existing tunnel. The existing tunnel is expanded to the right side, so the collapse body is on the left side of the expanded tunnel relatively, resulting in the load asymmetry of the expended tunnel, that is, the geological bias. Therefore, the surface settlement of section II-II’ is affected by both construction bias and geological bias. Compared with the excavation of the lower bench, the excavation of the upper bench is the main cause of the surface settlement. Therefore, the control of the surface settlement should be paid attention to in the excavation and removal of the temporary support of the upper bench.

4.3.2 Deformation analysis of primary support. The vertical displacement and horizontal displacement of section I-I’ and section II-II’ are monitored respectively, and the displacement time-history curve is shown in Fig. 11 and Fig. 12.

![Fig. 11 The time-history curve of horizontal displacement around the tunnel](image-url)
Fig. 12 The characteristic curve of vertical displacement around each critical section of tunnel

As shown in Fig. 11, the horizontal displacement of all parts around the tunnel increases first and then becomes stable. The horizontal displacement of each point around the tunnel gradually increases within the 5 construction steps before the upper left bench heading face reaches the critical section (within 10 m in front of the tunnel face, i.e. 0.5D). When the heading face passes through each section, the displacement curve changes suddenly, and the growth rate of the horizontal displacement around the tunnel reaches the maximum.

As for the tipping point of horizontal displacement around the tunnel, each section presents a relatively uniform rule: when each heading passes through each section, the stress redistribution is caused by the excavation and unloading of the tunnel, and the growth rate of the horizontal displacement of the corresponding part reaches the maximum. Besides, in the process of tunnel excavation, the surrounding rock within 0.5D behind the tunnel face is greatly affected, and the horizontal displacement growth rate of the surrounding rock is the largest.

As for the horizontal displacement after stabilization, the maximum horizontal displacement of section I-I' occurs at the right arch haunch, and the maximum displacement is 1.129mm. However, the maximum displacement of section II-II' occurs at the left spandrel, the maximum displacement is 2.625mm, and the displacement of section II-II' at the center of the collapse section is the largest because for the whole expansion tunnel, the collapse body is located at the left side of the tunnel, there is geological bias, and the left side of the tunnel bears greater surrounding rock pressure, so the horizontal displacement of the left spandrel is the largest. Therefore, in the construction process, special attention should be paid to the horizontal displacement control of the left spandrel of the collapse section.

As shown in Fig. 12, the development rule of the vertical displacement around each critical section of the tunnel with the construction process is almost consistent with the horizontal displacement. As for the direction of displacement, the vault has downward settlement displacement, while the arch bottom has upward uplift displacement.

In terms of the vertical displacement after stabilization, the maximum vertical displacement of section I-I' occurring at the invert is 4.113mm. However, the maximum displacement of section II-II' occurring at the vault, is 6.840 mm. Besides, the settlement of the left spandrel of section II-II' is 5.483mm, and the deformation is relatively prominent because the left side of the expanded tunnel bears geological bias. In the process of tunnel excavation, the surrounding rock within 0.5D behind the tunnel face is disturbed the most, and the vertical displacement growth rate of the surrounding rock is the largest. The final displacement contours of section II-II' are shown in Fig. 13.
4.3.3 Stress response analysis of surrounding rock. Because the whole section is divided into several parts for expanding excavation, and the multiple heading is pushed forward and closed in a ring one after another, and the construction step length and the staggering distance between the heading faces of each heading are short, the influence of tunnel excavation on the surrounding rock stress is very complex, so it is necessary to conduct a study on the influence of tunnel excavation on the surrounding rock stress. Select the section II-II’ and draw the curves of the first principal stress $\sigma_1$ and the stress intensity $\sigma_1 - \sigma_3$ with the construction steps, as shown in Fig. 14.

From the characteristics of time-history curve, with the advance of each heading face, the first principal stress $\sigma_1$ and the stress intensity $\sigma_1 - \sigma_3$ of each critical part show an overall growth trend. When the heading face passes through the critical section, the surrounding rock is affected by the degradation effect of loading and unloading, and the stress indexes change abruptly. Finally, each stress index tends to be stable, which shows that the surrounding rock support balance system is gradually formed, and shotcrete, rigid support, and anchor bolt are gradually exerting a role. The stress fluctuation of the surrounding rock parts of the critical section tunnel mainly occurs from the 3 construction steps in front of the left upper heading through the critical section to the three construction steps after the removal of the temporary support, a total of 24 construction steps. From the perspective of the influence scope of each heading, the influence scope of the excavation advance of each heading face on the stress indexes...
is about 10 m (0.5D) in front and rear of the critical section, the stress influence scope is about 1 time of the tunnel diameter.

4.3.4 Analysis of plastic zone of the surrounding rock. The distribution of plastic zone provides an important basis for judging the stress state of surrounding rock and analyzing the failure mechanism of surrounding rock, which is one of the most intuitive data reflecting the stability of surrounding rock. The distribution of the plastic zone of the surrounding rock after the completion of the tunnel excavation is shown in Fig. 15.

Notably, the above sections are mainly shear failure. The plastic area of section I-I' is mainly located in the shallow layer around the tunnel, while the plastic area of section II-II' is larger, which formed two "V" shaped shear plastic zones deep into the surrounding rock along the fracture surface at the upper left of the expansion tunnel. The distribution of plastic zones is corresponding to the asymmetric deformation. The “V” shaped plastic zones in the left spandrel are much larger than that of right, which brings about larger deformation in the left spandrel. In the practical project, the idea of asymmetric coupling support in critical parts, and different length of bolts to implement coupling support in the "V" shaped plastic zones should be adopted, to suspend the unstable rock mass to the deep stable rock stratum, to make full use of the strength of the deep surrounding rock.

5. Field test

To verify the reliability of numerical simulation in the process of in-situ expansion construction by the CRD method in the collapse section, the monitoring of surface settlement and deformation of the support structure in the tunnel is carried out. In this paper, the monitoring data of YK17 + 780 in the central section of the collapse section are selected for analysis.

5.1 monitoring results and analysis of surface settlement

The maximum settlement occurs on the left side of the middle line of the tunnel and the value obtained from the test is 2.4mm, 5m to the left of the middle line of the tunnel, which is 35.2% larger than the calculated value. The difference between the calculated and the measured value is gradually small when the distance is 12m away from the central line on both sides.
Compared with the field test results, the error of numerical simulation is mainly caused by the selection of parameters and the simplified assumptions before the analysis, such as the complexity of the collapse fill, the uncertainty of the practical construction. Although there are errors between the numerical simulation and the field test, the distribution of the settlement curve is almost the same. Many phenomena revealed by the numerical simulation are shown in the analysis of the measured data, and the maximum settlement value meets the safety requirements.

5.2 Monitoring results and analysis of primary support deformation

The monitoring items of the primary support deformation are mainly the settlement of the left and right vault and spandrel. The layout of the deformation monitoring points is shown in Fig. 17, and the cumulative settlement time-history curve is shown in Fig. 18.

From the trend of curve change, the settlement deformation of each monitoring point can be divided into three stages: acceleration deformation stage, oscillation stage, and convergence stage. With the release of surrounding rock load, the settlement value of each monitoring point increases rapidly within 5 days after the monitoring. With the surrounding heading face gradually approaching the monitoring section, the existing monitoring points are disturbed frequently, and the settlement curve enters the stage...
of oscillation. With each heading face gradually leaving the monitoring section, the settlement value of each monitoring point tends to converge. From the distribution of deformation, the settlement of the left spandrel is larger than that of the right spandrel, which indicates that there is bias pressure on the left side of the tunnel, and it is consistent with the numerical calculation results.

6. Conclusion

(1) With the theoretical formula, the spatial scope and scale of the collapse are estimated.

(2) The three-dimensional dynamic construction mechanical properties of the CRD method in the collapse section are studied. The results show that: The maximum surface settlement is 2.5m to the left of the middle line of the tunnel, which is mainly caused by the excavation of the upper bench. In the process of construction, the disturbance range in front and rear of the tunnel is 0.5 times of the tunnel diameter, and the maximum deformation of the surrounding rock occurs at the left spandrel. There are two "V" type shear plastic zones occurred in the larger deformation part of the surrounding rock arch.

(3) The results of field tests are almost consistent with the numerical calculation results, which verifies the reliability of the numerical simulation.

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