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Analysis of the deformation characteristics of the surrounding rock mass a deep tunnel during excavation through a fracture zone

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Abstract: Aiming at geological disasters triggered by fracture zones in surrounding rocks during the excavation of deep railway tunnels, the research investigated deformation and failure of surrounding rocks triggered by sudden changes of rock quality encountered in a tunnel excavation project. The research started from analysis of a field case: the Daliang Tunnel on the Gansu-Qinghai section of the Lanzhou-Urumuqi second double-track railway in China. The deformation profiles of surrounding rocks at different distances from the fracture zone was evaluated. The deformation of surrounding rocks in and around the fracture zone was studied by combining in-situ measurement, theoretical research, and numerical simulation. In addition, relationships of deformation of surrounding rocks in the section of the fracture zone with the materials and length of advanced support as well as the excavation footage behind the fracture zone were discussed. Analysis of the results showed that there is an inflection point on the deformation curves of surrounding rocks far ahead of the fracture zone; however, it is difficult to observe the inflection point on deformation curves of surrounding rocks in engineering practice as the rocks there have undergone substantial deformation before being exposed. A combination of material properties and length of advanced support is conducive to controlling the deformation of surrounding rocks in the fracture zone. Taking a rate of deformation of surrounding rocks in the fracture zone as less than 0.1 mm/d as the stability criterion, the excavated length of that rock mass behind the zone at a footage of 1 m/d is about two thirds of that at 2 m/d and half of that at 3 m/d when the deformation stabilizes.

Keywords: deep tunnel, fracture zone, rock excavation, surrounding rock deformation, excavation footage.

1 Introduction

With the vigorous development of infrastructure in China in recent years, tunnels running through mountains need to be built for highways and railways. In the context, it has become a general trend for future cavern excavation to form underground tunnels by fully-mechanised construction ^[1], in which the full-face excavation is key, whereas, it has become increasingly common to encounter adverse geological conditions such as fault fracture zones during tunnel construction. Owing to being characterised by low strength, ease of deformation due to looseness, and poor self-stabilisation capacity ^[2-4], these fracture zones pose a great threat to the safety and long-term stability of underground tunnel construction projects ^[5]. Considering this, the stress state on the section of the fracture zone must be understood before excavating to this position. Using the finite element method, Zeng *et al.* ^[6] and Zhou *et al.* ^[7] simulated the full-face excavation and support process when tunnelling through a fault fracture zone. In addition, they analysed the stress redistribution within the surrounding rock after full-face excavation. Moussaei *et al.* ^[8] designed and constructed a set of physical models to simulate the full-face excavation process of a circular tunnel and the volume loss and change induced by the excavation. Using a continuous cross-section monitoring system, Li *et al.* ^[9] monitored the convergence of a tunnel, to avoid excessive deformations thereof triggering a collapse.

If disasters such as collapses, rockbursts, and water in-rush occur in the surrounding rocks during the fully-mechanised, full-face excavation in the section of the fracture zone, the tunnel boring machine (TBM) may be deformed upon extrusion, becoming jammed, and even damaged ^[10, 11]. All these problems are caused by the large deformation after excavation unloading of the fracture zone, making it necessary to study the deformation of the rock mass in the fracture zone under conditions involving excavation disturbance. Zheng *et al.* ^[12] and Ortlepp *et al.* ^[13] state that excavation unloading using the TBM breaks the stress equilibrium state and subsequent stress redistribution may lead to the cracking, grooving, spalling, and collapse of rocks on the excavation face or tunnel walls. Lu *et al.* ^[14] analysed the stability of surrounding rocks in a fault fracture zone and investigated the changes in, and distribution of, surface and deep displacement of surrounding rocks. Chen *et al.* ^[15] explored the extrusion and failure mechanisms of a deep tunnel constructed in broken carbonaceous phyllite under high geostatic stresses. Through theoretical analysis and numerical simulation, Zhan *et al.* ^[16] conducted multi-factor coupling analysis on the

failure mechanism of soft-rock roadways, considering a convergence trend in all directions of high rheology and large deformation as the essential feature of the deformation of deep soft-rock roadways. Under the effects of fluid-mechanical coupling, Wang *et al.* ^[17], Ding *et al.* ^[18], and Barla ^[19] conducted triaxial unloading mechanical tests on sandstone at different initial confining pressures and unloading rates. In this way, they studied the evolution of the energy in sandstone specimens during damage induced by unloading and discussed the influences of the initial confining pressure and rate of unloading on the deformation of sandstone.

Scholars have conducted much research into means of controlling the deformation of, and damage to surrounding rocks in the fracture zone ^[20, 21]. Aiming at control over deformation of surrounding rocks in deep underground caverns, Xie *et al.* ^[22] proposed a set of comprehensive treatment methods and elaborated a support mechanism based on the unloading of the rock surrounding such caverns. This enriches the theory and technology behind control of the deformation of surrounding rocks of deep roadways. By combining the finite element and discrete element methods, Huo *et al.* ^[23], Su *et al.* ^[24], and Huang *et al.* ^[25] studied the deformation and failure mechanisms of surrounding rocks of caverns and proposed corresponding techniques with which to control the surrounding rock. To solve problems pertaining to the support of deep roadways under high geostress and in the presence of fracture zones, Li *et al.* ^[26] studied the deformation of roadways and modes of failure of supporting structures. On this basis, they proposed a new support scheme combining multi-level anchorage and concrete filled steel tubes. By constructing tunnel models containing a regional fracture zone, Xie *et al.* ^[27] and Cheng *et al.* ^[28] simulated and analysed the excavation, to determine an optimal support scheme for the rock surrounding the tunnel and provide a basis for *in-situ* construction. Lyu *et al.* ^[29] studied the strengthening effect of full-face anchorage on a roadway constructed in thick, soft, rock, and transformed the longitudinal confinement effect of the excavation face to the circumferential virtual internal support force for the first time. In this way, they proposed a two-dimensional (2-d) model for the full-face anchorage of the roadway constructed in thick, soft, rock. Du *et al.* ^[30] built a mechanical model for the boundary curves of plastic zones and the distribution of the principal stress of surrounding rocks of a circular tunnel under the effect of non-tectonic stress. By analysing the influences of different factors on the shapes of plastic zones and principal stress in the surrounding rock, they believed that stability of the rock surrounding a circular tunnel can be

improved by a series of measures. These include inhibiting the harmful expansion of plastic zones, improving the principal stress regime, and allowing uniform distribution of plastic zones within the range controllable by the support system.

Taking excavation of the fracture zone in the Daliang Tunnel in China as the engineering background, the present research investigated the method for predicting the fracture zone according to the deformation of the rock surrounding the tunnel. Moreover, the control effects of the length and material properties of advanced support on deformation of surrounding rocks in, and around, the fracture zone were also studied, and the severity of the damage to the excavation footage and length of the tunnel behind the fracture zone to regional support structures in the fracture zone was investigated. Finally, the appropriate length and material properties of advanced support as well as excavation footage and length of the tunnel behind the fracture zone were determined.

2 Practical problems during excavation through the fracture zone in the Daliang Tunnel

2.1 Project background

Table 1 Classification of surrounding rocks and designed excavation methods of the Daliang Tunnel ^[31]

Starting and ending mileage	Surrounding rocks	Geological features
DK328+820~DK329+810	V	Gravel soil, Stone, Joint fissure development
DK329+810~DK330+130	IV	Sandstone slate, Joint development, Rock fragmentation
DK330+130~DK330+700	III	Sandstone slate, Good lithology
DK330+700~DK330+980	IV	Sandstone slate, Joint development, Rock fragmentation
DK330+980~DK331+440	III	Sandstone slate, Good lithology
DK331+440~DK332+040	IV	Sandstone slate, Joint fissure development
DK332+040~DK334+790	III	Limestone slate, Large buried depth, Good lithology
DK334+790~DK334+840	IV	F5 fault influence zone, Limestone slate, Fracture development
DK334+840~DK335+040	V	F5 fault fracture zone, Fault hornstone
DK335+040~DK335+370	V	Affected by structure, Rock fragmentation

The Daliang Tunnel at LXS-7 in the Gansu–Qinghai section of the newly constructed Lanzhou-Urumuqi second double-track railway in China is in Menyuan County, Qinghai Province.

This is a mid to high-mountain area in Qilian Mountains with an average elevation of 3600 to 4200 m (4430 m at its peak). From chainage DK328+820 to DK335+370, the tunnel has a total length of 6550 m and runs through sandstone, limestone, slate, and the F5 fault fracture zone. The classification and geological features of surrounding rocks in different mileage ranges are listed in Table 1.

2.2 Excavation process and analysis of monitoring data

When excavating the tunnel to DK334+241, a collapse occurred at a position 24 to 34 m from the tunnel face, and the specific location is shown in Fig. 1. The steel frame for primary support fell off under compression and about 185 m³ of broken loose rocks became destabilised and collapsed into the tunnel. The resulting collapse cavity was 3.5 m deep, 6 m high, and 7 m long (Fig. 2). Surrounding rocks in the collapse zone contained fragmented carbonaceous slate interlayers and were significantly affected by the tectonic action. They collapsed constantly and were in a state of failure. Within the 10 m around the collapse zone, the primary shotcrete of surrounding rocks was found to contain numerous cracks, shotcrete on the vault showed severe spalling, and the maximum convergence of the sidewalls reached 1 m. The monitoring and measurement of the surrounding rocks showed that their collapse led to large deformations in and around that location. At 5 m from the collapse zone, the subsidence and convergence on the day of collapse reached 36.4 mm and 57.6 mm, respectively.

Fig. 1 The longitudinal profile of the collapse zone in the Daliang Tunnel (unit: m)

Fig. 2 Collapse of the fracture zone

The long-term cumulative subsidence and convergence of surrounding rocks in the tunnel section and the zone around were observed (Fig. 3). It was found that the cumulative deformation of surrounding rocks in the section was far larger than that in the surrounding zone, and even larger than that in other excavated tunnel sections and locations that had been monitored over a longer time. Therefore, accurately judging types of surrounding rocks ahead and identifying the potential fracture zone is of important practical significance when selecting a rational excavation scheme, implementing advanced support measures, and actively controlling deformation and stabilizing the surrounding rock.

Fig. 3 Monitoring results of cumulative deformation during the excavation

It remains difficult to identify small fracture zones or areas of suddenly-changing quality of surrounding rocks in the design stage due to difficulty in exploration and the high cost incurred; besides, the geological forecast can only explore a limited area in a single time, is costly, and even disturbs the normal field excavation process, therefore, it is impractical to completely rely on the geological forecast to judge whether a small fracture zone lies ahead or not. The deformation profiles of homogeneous rocks generally show a certain regularity, while the presence of a fracture zone influences the deformation characteristics of surrounding rocks within a certain distance, that is, characteristics of deformation curves of surrounding rocks, to some extent, can reflect changes in the nature of the surrounding rocks ahead.

The measured data from a tunnel section (DK330+029~DK330+115) with slight changes in the quality of surrounding rocks and without fracture zones around were analyzed. Relationships between the monitoring data and cumulative subsidence at monitoring points at different positions are plotted (Fig. 4), there is an inflection point on the curve fitted between the cumulative subsidence of the vault and number of days of monitoring.

Fig. 4 Statistical analysis of cumulative subsidence measured at different points

The further to analyse the position of the inflection point on the deformation profile, a cubic polynomial is used to fit the measured data such that:

$$Y = -0.0012x^3 + 0.1117x^2 + 0.1866x \quad (1)$$

The correlation coefficient is: $R=98.6\%$.

Let the second derivative of $Y(x)$ be 0, it can be calculated that the inflection point is at $X_{\text{拐}} \approx 31 d$, that is, the rate of deformation is found to change from increasing to decreasing on the 31st day.

3 Loosening of rock in the fault fracture zone under excavation disturbance during tunnelling

3.1 Stress redistribution in surrounding rocks caused by tunnel excavation

As the tunnel passes through the fault fracture zone, the surrounding rocks in the fault fracture zone of the tunnel tend to change from an elastic state to a plastic state and form a plastic broken rock zone; with increasing distance from the cavern wall, the radial stress σ_r

in the rock surrounding the cavern grows from zero and the stress state in the surrounding rock changes from an axial to a biaxial state. As a result, the plastic zone in the surrounding rocks is transformed to an elastic zone. Due to occurrence of the plastic broken-rock zone, the stress within a certain range of the zone decreases because of the concomitant stress relief therein. In addition, the locus of maximum stress concentration shifts from the cavern wall to the boundary of the plastic and elastic zones, increasing the stress in the elastic zone.

As shown in Fig 5, suppose that a circular tunnel with the radius of R_0 is excavated in a homogeneous, continuous, isotropic rock, and the radius of the plastic broken rock zone formed after excavation is R_p . Then, the circular cavern is in an isotropic ($\sigma_0 = \gamma Z$) hydrostatic stress field and the strength of the surrounding rocks conforms to the Mohr–Coulomb linear strength criterion. A unit in the plastic zone at distance r from the centre of the tunnel is selected. When not considering the body force of the unit itself, it satisfies the symmetric condition and the shear stress on the elemental area is 0. Under these conditions, the stress is only related to the variable r while remaining independent of θ , therefore, the radial stress σ_{rp} and the tangential stress $\sigma_{\theta p}$ are both principal stresses (major and minor, respectively).

Fig. 5 Analysis of the stress in the rock surrounding the plastic zone

Let $\sum \sigma_{rp} = 0$, the equilibrium equation along the r direction is obtained as follows:

$$\left(\sigma_{rp} + \frac{\partial \sigma_{rp}}{\partial r} dr \right) (r + dr) d\theta - \sigma_{rp} r d\theta - 2\sigma_{\theta p} dr \sin \frac{d\theta}{2} = 0 \quad (2)$$

If $d\theta$ is very low, then $\sin \frac{d\theta}{2} \cong \frac{d\theta}{2}$, the following can be obtained after expanding the

above equation and ignoring the high-order small quantity:

$$\frac{\partial \sigma_{rp}}{\partial r} + \frac{\sigma_{rp} - \sigma_{\theta p}}{r} = 0 \quad (3)$$

For weak and broken rocks, their shear failure can be evaluated using Mohr–Coulomb strength theory:

$$\sin \varphi = \frac{\sigma_{\theta p} - \sigma_{rp}}{\sigma_{\theta p} + \sigma_{rp} + 2c \cot \varphi} \quad (4)$$

Simultaneous equations are then constructed: when using the boundary condition $r=R_0$, then $\sigma_{rp}=p_i$, where p_i represents the support resistance.

The radial stress is given by $\sigma_{rp} = (p_i + c \cot \varphi) \left(\frac{r}{R_0} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - c \cdot \cot \varphi$

The tangential stress is given by $\sigma_{\theta p} = \frac{1 + \sin \varphi}{1 - \sin \varphi} (p_i + c \cot \varphi) \left(\frac{r}{R_0} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - c \cdot \cot \varphi$

It can be seen from this that the redistributed stress in surrounding rocks in the plastic zone is independent of the natural stress on the rock. Instead, it is dependent on the support resistance p_i and the indicators of shear strength of rocks (c and φ).

According to the theory of elasticity, when the lateral pressure coefficient of rocks is $\lambda=1$, the stress in rock mass in the elastic zone after tunnel excavation meets the following conditions:

$$\begin{cases} \sigma_{re} = \sigma_0 \left(1 - \frac{R_0^2}{r^2} \right) + p_i \frac{R_0^2}{r^2} \\ \sigma_{\theta e} = \sigma_0 \left(1 + \frac{R_0^2}{r^2} \right) - p_i \frac{R_0^2}{r^2} \\ \tau_{r\theta e} = 0 \end{cases} \quad (5)$$

On the boundary between the elastic and plastic zones, $\sigma_{\theta p}$ and σ_{rp} can be calculated either using the stress formula for the elastic zone or that for the plastic zone, giving $\sigma_{re} = \sigma_{rp}$, $\sigma_{\theta e} = \sigma_{\theta p}$, $\sigma_{re} + \sigma_{\theta e} = \sigma_{rp} + \sigma_{\theta p}$

By substituting the stress predicted by formula applicable to the elastic and plastic zones into the above formula, it can be obtained that.

$$\sigma_{re} + \sigma_{\theta e} = 2\sigma_0 \quad (6)$$

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$$\sigma_p + \sigma_{\theta p} = \frac{2}{1 - \sin \varphi} (p_i + c \cot \varphi) \left(\frac{r}{R_0} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - 2c \cdot \cot \varphi \quad (7)$$

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$$\text{Then } 2\sigma_0 = \frac{2}{1 - \sin \varphi} (p_i + c \cot \varphi) \left(\frac{R_p}{R_0} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - 2c \cdot \cot \varphi$$

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Therefore, the radius of the plastic zone is given by

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$$R_p = R_0 \left[\frac{\sigma_0 (1 - \sin \varphi) - c \cdot \cos \varphi + c \cdot \cot \varphi}{p_i + c \cot \varphi} \right]^{\left(\frac{1 - \sin \varphi}{2 \sin \varphi} \right)} \quad (8)$$

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3.2 Distribution characteristics of deformation of surrounding rocks during tunnel excavation

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Previous research on the distribution of displacement induced by blasting excavation of

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tunnels focuses more on the magnitude of displacement on the transverse section of the tunnel. As

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shown in Fig 6, ignoring the dilation of the surrounding rocks, the displacement u^* at the

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boundary of the plastic zone and the displacement u of the tunnel wall can be expressed by

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Formula (9) and (10), where, according to Formula (8):

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$$u^* = \frac{1 + \mu}{E} R_p (\sigma_0 \sin \varphi + c \cdot \cos \varphi) \quad (9)$$

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$$u = \frac{R_p}{R_0} u^* + \frac{K(R_p^2 - R_0^2)}{2R_0} \quad (10)$$

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where R_0 and R_p separately refer to the radii of the tunnel and the plastic zone; σ_0 , P_i , φ ,

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μ , E , and K denote the initial geostress, support resistance, internal friction angle, Poisson's ratio,

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elastic modulus, and bulk modulus, respectively.

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Fig. 6 Radial displacement of surrounding rocks

The displacement of the tunnel wall is approximately directly proportional to the square of the radius of the plastic zone, and the surrounding rocks are significantly deformed under conditions of a low internal friction angle or high geostress. During the normal excavation of the tunnel, when encountering the fracture zone, that is, when the quality of the surrounding rock changes significantly on the longitudinal section, the displacement shows an abrupt increase.

The research into deformation profiles on the longitudinal section is mainly limited by the three-dimensional (3-d) nature of the problem. Previous research on convergence and deformation of formations due to tunnel excavation also mainly takes the radial displacement of the tunnel as the research object [32]. Under the assumption of linear elasticity, the radial deformation of rock surrounding a circular tunnel can be expressed in polar coordinates:

$$U_r = \eta \frac{\sigma_1 R^2 (1 + \mu)}{2Er} \left\{ 1 + \lambda + (1 - \lambda) \left[4(1 - \mu) - \frac{R^2}{r} \right] \cos(2\theta) \right\} \quad (11)$$

where λ , μ , σ_1 , and η separately represent the lateral pressure coefficient, Poisson's ratio, maximum overburden pressure, and constraint loss.

Fig. 7 Numerical model of the profile of radial displacement along the tunnel axis

After considering the elastic and plastic deformation of rocks, empirical formulae were used to describe the ratio of the radial deformation to the maximum deformation of the surrounding rock at different distances from the tunnel face. It is supposed that the far-field stress on the rocks is homogeneous, so the profile of radial displacement along the tunnel axis can be calculated using the numerical model for the problem shown in the Fig.7, and the profile represents the longitudinal section of the unlined tunnel with a radius of R near the tunnel face. The radial displacement is U_r at distance X from the tunnel face. If X is large, the radial displacement reaches its maximum. For negative values of X (i.e., for positions ahead of the tunnel face), the

radial displacement decreases, and the displacement at a shorter distance ahead of the tunnel face is practically 0, for example,

Panet ^[33] proposed that

$$\frac{U_r}{U_{\max}} = \alpha_0 + (1 - \alpha_0) \left[1 - \left(\frac{m}{m + Z/R} \right)^2 \right] \quad (12)$$

Hoek ^[34] and Carranza-Torres ^[35] point outed that

$$\frac{U_r}{U_{\max}} = \left[1 + \exp \left(\frac{-Z}{1.1R} \right) \right]^{-1.7} \quad (13)$$

Most empirical formulae used to describe the deformation at longitudinal sections are summarised based on numerous items of *in-situ* monitoring data pertaining to deformation and can intuitively reflect the changes in radial displacement of the unsupported tunnel wall. Whereas, after considering the influence of the fault fracture zone ahead of the tunnel face, the deformation distribution on the longitudinal section of the tunnel becomes more complicated. In the case, it needs to be analysed from the 3-d perspective using numerical calculation ^[36-38].

4 Numerical simulation and verification of the excavation process

The rock samples collected from the fracture zone in the field were analysed by conducting the laboratory test ^[39]. Besides, the HJC constitutive model was used for numerical calculation. The material parameters of the model are shown in Table 2. Based on the field test, the tested data of effective geostress were obtained at depths of 15.0, 18.5, 22.0, and 27.0 m in the main auxiliary tunnel of the inclined shaft (Table 3).

Table 2 Material parameters of the HJC numerical model

Type	$\rho /$ kg/m ³	$G /$ GPa	A	B	C	N	f_c MPa	$T /$ MPa	D	P_c MPa	μ_c	P_l GPa	μ_l	k_1 GPa	k_2 GPa	k_3 GPa
Surrounding rock	2620	15.2	0.88	1.9	0.009	0.77	56	4.54	0.05	17.3	0.001	0.88	0.1	90	-197	228
Fracture zone	1890	9.9	0.53	1.2	0.004	0.42	33	2.86	0.024	13	0.001	0.78	0.1	63	-142	169

Notes: ρ , G , A , B , C , N , f_c , T , and D represent the density, shear modulus, normalised cohesive strength, normalised pressure hardening coefficient, strain-rate hardening exponent, pressure hardening exponent, quasi-static compressive strength, maximum tensile stress of materials, and damage parameter, respectively; P_c , μ_c , P_l , and μ_l denote the hydrostatic pressure, volumetric strain, hydrostatic pressure under compaction, and volumetric strain corresponding to the hydrostatic pressure under compaction; k_1 , k_2 , and k_3 are pressure constants.

Table 3 The hydrofracturing results from vertical boreholes

Geostress direction	Measuring depth of position / m				Geostress of Calculation Model
	15	18.5	22	27	
Maximum horizontal principal stress / MPa	23.55	25.14	23.04	24.81	24
Minimum horizontal principal stress / MPa	12.95	13.45	13.77	13.37	13.4
Perpendicular stress / MPa	12	12.09	12.18	12.3	12

Based on the above test results, the physico-mechanical parameters and geostress of the rock mass were input to the calculation model. By using the ANSYS/LS-DYNA numerical simulation software, a 3-d calculation model for the excavation of the deep tunnel crossing the fault zone was built. For the convenience of obtaining deformation characteristics of surrounding rocks of the tunnel, a one-half scale model was used for calculation. The model measures $200\text{ m} \times 50\text{ m} \times 100\text{ m}$ (length $\times$ width $\times$ height), as shown in Fig. 8. It is supposed that the excavated length of the tunnel model is 120 m, the cross-section of the tunnel is that of a circular arch, and the tunnel excavation has a span of 15.0 m and height of 13.0 m. The width of the fault fracture zone is 10 m according to that of the collapse zone. For convenience of subsequent calculation and analysis, the dip angle of the fault fracture zone is set to 90° .

Fig. 8 Dimensions of the numerical model (unit: m)

The numerical model is built using eight-node hexahedron elements, including 187,880 elements and 200,850 nodes (Fig. 9). Full-face excavation is performed from left to right at the rate of 1 m/d by referring to the practical excavation process.

Fig. 9 Finite-element mechanical model

Fig. 10 Data measured in the tunnel and numerically calculated deformation profiles for surrounding rocks

Comparison of the calculated and measured subsidence at monitoring points in the vault 38 m from the fracture zone (Fig. 10) shows that there is certain deviation between the calculated and measured results. On the one hand, an obvious increase in the deformation rate is observed on the calculated deformation profile one day after excavation at the monitoring point, that is, there is an apparent inflection point for the deformation; however, no such inflection point is observed on the measured profile. Considering that it is a high-risk task to measure deformation in the close vicinity of the tunnel face immediately after excavation of the monitoring point in engineering

practice, the actual monitoring is generally carried out one, or even several days after excavation [40]. Therefore, considering the lag of the practical monitoring, the deformation profile obtained through calculation is translated leftward, to obtain a new curve. Comparison shows that the calculated and measured deformation curves considering the lag in the monitoring match in terms of the trend in deformation or specific values thereof.

5 Deformation characteristics of rock surrounding the fault fracture zone under full-face excavation

5.1 Numerical simulation to predict likely fracture zones using the deformation profile of the rock surrounding the tunnel

The tunnel was excavated at a footage of 1 m/d from the position where the tunnel face was 55 m from the fracture zone, until the fracture zone was just exposed (Fig. 11). Before tunnelling through the fracture zone, deformation curves at elements in the vault 50, 45, 40, 35, 30, 25, 20, 15, 10, 5, and 1 m from the fracture zone with the day of excavation were obtained. On this basis, the relationship between deformation of the surrounding rock and the fracture zone ahead was ascertained.

Fig. 11 Layout of monitoring points in the rock surrounding the tunnel

The excavation is stopped when the tunnel face reaches the position where the fracture zone is exposed. In this way, cumulative deformation in 70 d at various monitoring points is obtained (Fig. 12). It is found that subsidence and deformation of surrounding rocks differ with distance from the tunnel face as it advances towards the fracture zone. In the tunnel section far from the fracture zone, the change rate of deformation of surrounding rocks first increases, then gradually decreases with further excavation; when the tunnel face is less than 20 m from the fracture zone, the rocks in the region around the fracture zone were deformed substantially, and the total

cumulative deformation energy is larger. Therefore, within a certain range, the closer the monitoring points are to the fracture zone, the larger the deformation recorded before excavation thereat. According to the stress analysis, this happens because of the low rigidity of the fracture zone, so surrounding rocks closer to the fracture zone are weakly confined, which allows their greater deformation in the early stages of the excavation.

Fig. 12 Calculated cumulative subsidence at different positions

Considering the ideal monitoring in the practice, that is, the deformation monitoring starts at the same time as the excavation reaches the monitoring point, then the calculated cumulative subsidence after exposure of monitoring points at different points is as shown in Fig. 13: the monitoring point 50 m from the fracture zone shows the maximum cumulative subsidence at 20 d after its exposure, while the monitoring point at 20 m from the fracture zone shows the minimum subsidence. By analysing the deformation profiles of different monitoring points after exposure, it is evident that an inflection point is present on the curves at some monitoring points far from the fracture zone. The inflection point functions as an indicator of the presence of a fracture zone ahead of the tunnel. This indicates that the numerical model can be used to simulate and analyse the deformation of exposed rock surrounding such a tunnel in engineering practice.

Fig. 13 Calculated cumulative subsidence after exposure of monitoring points at different positions

5.2 Deformation of surrounding rocks under different forms of advanced support

Fig. 14 Surrounding rocks in, and around, the fracture zone before excavation

Fig. 15 Surrounding rocks in, and around, the fracture zone after excavation

The tunnel was excavated from a position 10 m ahead of the fracture zone to one 10 m behind it. Elements in the fracture zone and within 10 m ahead and behind the fracture zone were selected to analyse the displacement. The model before and after excavation and the selected analysis elements are shown in Figs 14 and 15. The analysis was conducted on elements with an interval of 1 m. The elements of monitoring points in the vault of the tunnel were labelled as E1 to E30, while those in the sidewalls were labelled F1 to F30. On this basis, the influences of different material properties and lengths of advanced support on the subsidence and convergence of surrounding rock in, and around, the fracture zone were investigated.

5.2.1 Different materials of advanced support

To analyse the influences of different materials used in the advanced support system on the deformation of the rock surrounding the fracture zone after tunnel excavation, the length of the advanced support is set to 12 m, and L is set to 1 m beyond the fracture zone at each end. The HJC constitutive model is also used for the support materials. Table 4 lists the main mechanical parameters of the six groups of support materials. Figs 16 and 17 compare the deformation (subsidence and convergence) curves of surrounding rocks in, and around, the fracture zone when using different support-material parameters.

Table 4 Material parameter used to model the advanced support in the HJC model

No.	$\rho /$ kg/m ³	$G /$ GPa	A	B	C	N	f_c MPa	T/ MPa	D_1	P_c MPa	μ_c	P_l GPa	μ_l	k_1 GPa	k_2 GPa	k_3 GPa
1	2700	15.6	0.93	2.0	0.009	0.80	62	4.87	0.053	18	0.001	0.91	0.1	96	-208	240
2	2800	15.8	0.98	2.13	0.0097	0.86	66.7	5.16	0.057	18.67	0.001	0.94	0.1	99.67	-220	251
3	2900	16.1	1.02	2.27	0.0103	0.93	71.3	5.45	0.062	19.33	0.001	0.98	0.1	103.33	-233	261
4	3000	16.3	1.07	2.40	0.0110	0.99	76.0	5.74	0.066	20.00	0.001	1.01	0.1	107.00	-245	272
5	3100	16.6	1.12	2.53	0.0117	1.05	80.7	6.03	0.070	20.67	0.001	1.04	0.1	110.67	-257	283
6	3200	16.8	1.16	2.67	0.0123	1.12	85.3	6.32	0.075	21.33	0.001	1.08	0.1	114.33	-270	293

As shown, the surrounding rocks around the fracture zone exhibit slight differences in deformation; while those in the fracture zone show significantly different deformations, with the maximum subsidence reducing from 107.8 to 64.5 mm and maximum convergence decreasing from 78.2 to 48.4 mm. When increasing the strength of material used in the advanced support, deformation in the surrounding rock decreased significantly. In particular, the subsidence and convergence of surrounding rocks show large changes when using materials 1 to 4 in the advanced

support, however, when improving strength of the materials further, the control effect of advanced support on deformation of surrounding rocks of the tunnel gradually weakens and the support becomes harder to build and install.

Fig. 16 Calculated subsidence of surrounding rocks in the vault under advanced support with materials of different properties

Fig. 17 Calculated convergence of surrounding rocks under advanced support with materials of different properties

At the same time, it is found that the deformation of surrounding rocks remained relatively large, which indicates that the advanced support implemented only in the section of the fracture

zone fails to give the full play to the strength of the materials used in the advanced support. Therefore, the following section further discusses the control effect of the length of advanced support on deformation of surrounding rocks in, and around, the fracture zone.

5.2.2 Different lengths of advanced support

Fig. 18 Calculated subsidence of surrounding rocks in the vault under different lengths of advanced support

Fig. 19 Calculated convergence of surrounding rocks in the sidewalls under different lengths of advanced support

Taking the parameters of material No. 4 for the advanced support (Section 5.2.1) as the

benchmark, the influences of the length of advanced support on the deformation of the fracture zone were further analysed. Here, the lengths of the advanced support for the fracture zone are set to 14, 16, 18, 20, 22, 24, and 26 m, which are $L = 2, 3, 4, 5, 6, 7$, and 8 m beyond the two ends of the fracture zone, respectively. Under those conditions, the changes in the subsidence and convergence of surrounding rocks in, and around, the fracture zone under different lengths of advanced support were studied.

As seen from Figs 18 and 19, deformation at elements E1 to E10 and E21 to E30 in the vault in the vicinity of the fracture zone, as well as at elements F1 to F10 and F21 to F30 in the sidewalls all decrease slightly with certain regularity for given support parameters. In comparison, an apparent attenuation is seen in the deformation of elements E11 to E20 in the vault and F11 to F20 in the sidewalls in the section of the fracture zone: both the subsidence and convergence of surrounding rocks are much reduced as the length of advanced support increases from 14 m to 20 m; with the further increase of the length from 20 m to 26 m, the subsidence and convergence of surrounding rocks change only marginally. This indicates that for a fracture zone with a width of 10 m, it is recommended to set the length of advanced support to 20 m by comprehensively considering the safety and stability of surrounding rocks and the construction efficiency (*i.e.*, it is best to use a length of advanced support that is twice the width of the fracture zone).

5.3 Damage to rock surrounding the fracture zone under excavation footages behind the zone

Fig. 20 Excavation of rock mass behind the fracture zone

Taking the parameters of material No. 4 for the advanced support (Section 5.2.1) and a length of advanced support of 20 m as the benchmark, the tunnel is excavated uninterruptedly for 50 m

from the position 5 m behind the fracture zone (Fig. 20). The numerical calculation and analysis were conducted at excavation footages of 1, 2, and 3 m/d, respectively, to study deformations of elements E15 in the vault and F15 in the sidewall (both within the fracture zone), under different excavation footages.

Fig. 21 Calculated subsidence of surrounding rocks in the vault under different excavation footages

Fig. 22 Calculated convergence of surrounding rocks in the sidewalls under different excavation footages

As seen from Figs 21 and 22, as the excavation zone of the tunnel moves further from the fracture zone, the deformation around the tunnel decreases constantly. When taking a daily

deformation of surrounding rocks of less than 0.1 mm as a criterion for judging stability, during the excavation at 1 m/d, the rate of subsidence of element E15 falls to below 0.1 mm/d after excavation over a length of 19 to 20 m; while the rate of convergence of element F15 is less than 0.1 mm/d after excavation over 15 to 16 m. If the tunnel is excavated at 2 m/d, the rate of subsidence of element E15 and the rate of convergence of F15 fell to a value below 0.1 mm/d separately after excavation over 30 to 32 m and 26 to 28 m. At an excavation footage of 3 m/d, the rate of subsidence of element E15 fell to less than 0.1 mm/d after excavation over 39 to 42 m, while the rate of convergence of element F15 fell to this level after excavation over 36 to 39 m. In general, the subsidence is larger than the convergence of the surrounding rocks, and the latter is about 0.7 times the former, therefore, the subsidence should be used to judge the stability of the surrounding rocks, according to which the excavation footage can be changed. When the tunnel is excavated over 30 m, the excavation footage can be increased from 1 m/d to 2 m/d, and furthermore to 3 m/d after excavation over 39 m.

6 Conclusion

This research demonstrates an engineering case-study in which the excavation of a deep tunnel through a fracture zone induced deformation and failure of surrounding rocks. Numerical simulation and analysis were conducted throughout the excavation of the tunnel through the fracture zone as well to control the deformations therein. The following conclusions can be drawn:

(1) If there is a fracture zone on the excavation path of a tunnel ahead, the rate of deformation of the rocks surrounding the tunnel section ahead of the fracture zone tends to increase at first, then decrease, with an inflection point in the deformation curve. This inflection point can only be observed on the deformation curves of exposed rock surrounding a tunnel section far from the fracture zone, however, as the rock surrounding the tunnel near the fracture zone have undergone large deformation early in the process, no inflection point is observed after exposure of that rock. The deformation profiles of surrounding rocks at different locations can also be used to estimate whether a fracture zone lies ahead or not.

(2) When the strength of materials used to form advanced supports is 1.15 times that of the rock surrounding the tunnel and the length of advanced support is twice the width of the fracture zone, the deformation of surrounding rocks can be safely controlled. In addition, the convergence is about two-thirds to three-quarters of the subsidence of the surrounding rock.

(3) With the increase in the excavation length behind the fracture zone, the final deformation of surrounding rocks in the fracture zone at the excavation rate of 1 m/d is about two-thirds that at 2 m/d and half that at 3 m/d. In addition, as the deformation of the surrounding rock stabilizes, the excavated length at 1 m/d is also about two-thirds that at 2 m/d and half that at 3 m/d.

Author Contributions statement

Junhong Huang: Conceptualization, Formal analysis, Writing - review & editing, Funding acquisition. Guang Zhang: Data curation, Methodology. Yi Luo: Writing - original draft, Project administration, Formal analysis, Funding acquisition. Shaohua Hu: Investigation, Supervision. Hangli Gong: Prepared figures 8-22. Xinpeng Li: Prepared figures 1-7, Funding acquisition. Xin Liu: Validation.

Competing Interests statement

We declare that the authors have no competing interests as defined by Nature Research, or other interests that might be perceived to influence the results and/or discussion reported in this paper.

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Figures

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