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ORIGINAL ARTICLE

The influence of seepage characteristics on the reliability of a tunnel roof under dynamic disturbances

Tianjiao Yang · A. P. S. Selvadurai · Shuhong Wang

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Abstract Tunnels in use are often affected by dynamic effects. The reliability method is adopted in this paper to comprehensively examine the uncertainty parameters of the structure and the fractures within the rock mass. A damage variable is included in the permeability expression to analyze the changes in the safety to the tunnel roof with fractures, the width of the fractures, and the changes in permeability under dynamic disturbances. It is found that a single dynamic disturbance has little impact on the reliability of the structure, but there is a considerable potential safety hazard to the structure after multiple dynamic disturbances. When a disturbance occurs several times, the failure probability of the structure can increase sharply and the structure can experience rapid failure. In considering seepage in the rock mass, initially, the permeability of rock can be neglected comparison to that that of a fracture. However, when structural damage occurs, the permeability of the fracture increases exponentially, and the permeability of rock also changes dynamically.

T. Yang · S. Wang (✉)
Dept. of Resources and Civil Engineering, Northeastern University, Shenyang 110819, China
e-mail: shwang@mail.neu.edu.cn

T. Yang
e-mail: yangtianjiaols@126.com

A. P. S. Selvadurai
Dept. of Civil Engineering and Applied Mechanics, McGill University, 817 Sherbrooke Street West, Montreal, QC H3A 0C3, Canada
e-mail: patrick.selvadurai@mcgill.ca

Article Highlights In order to study the damage characteristics of a tunnel roof under dynamic disturbance based on the reliability theory, three points are proposed in this paper:

- First, a single dynamic disturbance has little influence on the tunnel reliability. During the disturbance, the stress and reliability of the structure go through a rising—stable—falling process. However, due to the influence of damage, the reliability of the structure cannot be restored to its initial state after the disturbance ends. After the disturbance reaches a certain number of times, the failure probability of the structure will suddenly increase intensely. Therefore, in a structural safety assessment, the damage to the structure should not be ignored just because the damage due to a single disturbance is low.
- Second, the permeability of the fracture is not constant but changes during structural damage. The fracture permeability was seen to increase by three times after multiple disturbances. Therefore, it is necessary to take the dynamic permeability characteristics of the rock mass into consideration in the research and analysis of rock mass permeability. Moreover, the permeability of the rock itself can be neglected compared with that of the fracture.

- Third, as the damage increases, the fracture width will expand continuously, but this expansion can be approximately regarded as a linear expansion. At the same time, in the process of decreasing structural safety, the difference between the structural safety analysis with and without fractures is also larger. Therefore, it is particularly important to consider the influence of fractures in the reliability safety assessment of tunnel structures.

Keywords Tunnel roof · Permeability · Fractures · Damage · Reliability · Dynamic disturbance

1 Introduction

There is a complex interaction and a mutual constraint between the seepage field and the deformation field of a rock mass. The mechanical properties of rock masses will change dramatically during long-term loading (Deru et al. 2007; Gudmundsson et al. 2010; Jiang et al. 2004), which can seriously affect the permeability of rock mass (Yin et al. 2014).The permeability of the rock mass is a vital indicator for rock mass stability, which has a controlling effect on the project construction and safety throughout the operation. During the construction and operation of various rock engineering projects, especially in regions of high geostatic stress states, high temperature and high water pressure, under the influence of factors such as stress concentration, unloading and seepage caused by excavation, the original micro-cracks in the rock mass can develop and expand into macro cracks, which result in macro-crack formation and rock fragmentation. Compared with intact rock mass, the permeability of fractured rock mass is significantly higher; this can cause a sudden change in the seepage of rock mass during engineering endeavours and cause major disasters (Jiang et al. 2008). In terms of the seepage characteristics of a rock mass, much relevant research has been done over the past five decades. Tsang and Witherspoon (1981) proposed a protrusion-depression combination model; that is, the protrusion and bulge on the fracture surface control the deformation and seepage characteristics of the fracture, respectively. Wu et al. (2019a, b) studied safety assessment methods under different conditions in tunnel engineering. In their studies, if the tunnel is disturbed due to dynamics, the stress of the tunnel structure gradually increased from the original rock stress to a peak stress and then the stress was rapidly relieved. In this process, the tunnel roof gradually changed from its original geometry to one that was damaged and fragmented. As fractures propagate, the permeability increases rapidly (Selvadurai 2004a, b; Selvadurai and Ichikawa 2013; Selvadurai et al. 2011), which can be several orders of magnitude greater than the initial value (Huang et al. 2018; Zhang et al. 2017). Therefore, the effects of the damage caused by dynamic disturbance on permeability must be considered in any safety assessment exercise. Souley et al. (2001) described the excavation damage-induced increase in permeability in granite of the tunnel from the Canadian Shield, which can be up to four orders of magnitude. The research by Massart and Selvadurai (2012) developed a multi-scale computational homogenization approach to relate the isotropic and the deviatoric stress-induced permeability evolution, where the micro-mechanical processes at the grain-scale incorporate both Coulomb-friction and dilatant friction.

Compared with the aforementioned model, it is found that the geological data and geotechnical model of an underground structure cannot be completely and perfectly obtained; natural uncertain factors cannot be accurately expressed by deterministic models. Therefore, the reliability analysis method has become a reasonable approach when considering the influence of engineering uncertain factors (Ching et al. 2009). In recent years, more and more attention has been paid to reliability analysis of underground structures. Mollon et al. (2009, 2013) gave accurate results of the failure probability of a circular tunnel face based on the limit analysis theory with first-order second-moment method and response surface methodology. Hoek (1998) and Hoek and Marinos (2000) studied the failure probability of a rock mass properties by using the Monte Carlo theory. Zeng (2014) analyzed the reliability of a circular tunnel driven by a pressurized shield by the Hoek–Brown non-linear failure criterion using the first order reliability method (FORM), response surface methodology and an importance sampling method. Li and Low (2010) used the first order reliability method and the Monte-Carlo simulation to calculate the reliability index of a circular tunnel subjected to a hydrostatic pressure field. Low and Einstein (2013) studied the reliability of a symmetric roof wedge of a circular tunnel and rock forces using the first-order reliability method (FORM) and the second-order reliability method (SORM).Wu et al. (2019a, b) used the idea of reliability method to predicted the reliability of a tunnel roof under blasting disturbances. These methods can evaluate the characteristics of the tunnel structure to a certain extent, but also have their own limitations. At present, studies devoted to the analysis of fluid–solid coupling in a fractured rock mass often ignore the influence of uncertainty caused by complex parameters of the fractured rock mass, while studies focusing on reliability correlation of structural randomness do not incorporate analysis of the impact of the fractures themselves.

In this paper, damage variables are included in the permeability expression, and the permeability model with damage characteristics is considered. A reliability method is adopted in this paper to comprehensively consider the parameter uncertainties of the structure and the fracture factors inside the rock mass, the difference between the reliability changes in the tunnel roof with and without cracks, the width of the fractures, and the changes in the permeability under conditions of dynamic disturbances.

2 Vibration damage model of rock mass under dynamic disturbance

Dynamic disturbance is an important factor that must be considered when examining the functional requirements of a tunnel structure. Dynamic disturbances can cause certain vibrations in the rock mass; The intensity of the vibration is influenced by the mechanical properties of the rock mass. However, the dynamic damage has always been evaluated in terms of over-excavation rather than considering the actual characteristics of the damage (Raina 2000). Saiang (2004) pointed out that the key to quantification of blasting damage is to determine the depth and range of the damage, instead of considering the deterioration of mechanical properties such as the strength and the stiffness. Jafari (2003) evaluated the degradation law of rock joints subjected to cyclic shear load through experiments. Crawford and Curran (1982) conducted dynamic experiments on a rock mass and found that the friction coefficient of the rock mass changes dynamically with the shear rate. The experiment of Singh (2011) examined the effect of shear velocity on the friction characteristics of the rock mass surface. Dynamic disturbance is a dynamic process as well as incorporates significant randomness. The safety state of the rock mass structure cannot be accurately and effectively reflected only by the parameters of a specific state. The mechanical property of a tunnel roof is random and influenced by dynamic actions, which requires changing ideas for the study of the reliability of the blasting process (Chaudhuri and Chakraborty, 2006; Liu et al. 2011; Thirukumaran et al. 2015). In the dynamic disturbance, the strength of the rock mass will be degraded and reduced under the cyclic action of the vibration loading; this form of degradation process itself is a dynamic form. The degradation coefficient $D(t)$, which changes with time, is used to represent the dynamic strength attenuation of a rock mass under dynamic disturbance. The shear strength of the rock mass structural surface at any moment $\tau(t)$ can be expressed as

$$\tau(t) = \tau_0 \cdot D(t) \tag{1}$$

where $D(t)$ represents the degradation coefficient, τ_0 represents the initial shear strength of the rock mass structural surface, which is determined from the Mohr–Coulomb failure criterion

$$\tau_0 = \sigma(t) \cdot \tan\varphi_0 + c_0 \tag{2}$$

In (2), φ_0 represents the initial friction angle, c_0 represents the initial cohesion, and $\sigma(t)$ represents the normal stress applied to the structural surface at any time. Therefore, the strength parameters of the rock mass structural surface $\tau(t)$ can be determined from (3) at any time of the dynamic action.

$$\begin{aligned} \tau(t) = \sigma(t) \cdot \tan\varphi_0 \cdot D(t) + c_0 \cdot D(t) \\ \varphi(t) = \arctan[\tan\varphi_0 \cdot D(t)] \\ c(t) = c_0 \cdot D(t) \end{aligned} \tag{3}$$

where $\varphi(t)$ represents the friction angle at any moment, $c(t)$ represents the cohesion at any moment. In (3), it can be obtained that the shear strength parameters of the structural surface at any time of a blasting sequence can be obtained as long as the expression of the degradation coefficient $D(t)$ is obtained. The influence on the strength parameter is expressed as the relative velocity influence coefficient $\gamma(t)$ and the vibration wear influence coefficient $\eta(t)$. Assuming that both factors are mutually independent, the vibration degradation coefficient is defined as (Wu et al. 2019a, b)

$$D(t) = \gamma(t) \cdot \eta(t) \tag{4}$$

In the process of rock mass deterioration, its permeability k will also change (Selvadurai 2004a, b; Selvadurai and Shirazi 2004)

$$k = k_0\left[1 + (1 - D(t))^2\right] \tag{5}$$

where k_0 represents the initial permeability.

Experiments have shown that (Jafari 2003; Lee 2001):

(i) When the cyclic shear frequency is increased, the strength of the structural surface will decrease.
(ii) The rate of strength decrease is faster in the early stage and gradually slowed.
(iii) When the cyclic shear amplitude is increased, the degree of the strength degradation of the structural surface will increase.
(iv) When the cyclic shear amplitude is increased, the final convergence of the structural strength will decrease.
(v) The wear influence coefficient of the structural surface strength is influenced by the number of cyclic shears and the cyclic shear amplitude.

The wear influence coefficient of vibration of the structural surface $\eta(t)$ can be expressed by a negative exponential function (Zeng et al. 2014)

$$\eta(t) = \delta(t) + [1 - \delta(t)]e^{-a \cdot K(t)} \tag{6}$$

where $\delta(t)$ represents the convergence value of the wear influence coefficient, $K(t)$ represents the number of cyclic shears, a represents the undetermined coefficient that can be determined by fitting experimental data.

When the cyclic shear amplitude is increased, the convergence value $\delta(t)$ of the wear influence coefficient will decrease, which is consistent with the negative exponential decay law. It can be expressed as

$$\delta(t) = R_0 + (1 - R_0)e^{-b \cdot J(t)} \tag{7}$$

where $J(t)$ represents the cyclic shear amplitude, R_0 represents the minimum value of the convergence value $\delta(t)$ with the degradation of the cyclic shear amplitude, which can be determined by the test. b represents the undetermined coefficient that can be obtained through experimental data.

Substituting (7) into (6) we obtain the expression for wear influence coefficient of the vibration

$$\eta(t) = R_0 + (1 - R_0)e^{-b \cdot J(t)} + (1 - R_0)\left[1 - e^{-b \cdot J(t)}\right]e^{-a \cdot K(t)} \tag{8}$$

Experiments have shown that (Singh et al. 2011):

(i) When the relative speed increases, the friction coefficient of the rock structure surface will decrease. When the relative speed is at a low level, the friction coefficient will decrease rapidly; if the relative speed is at a high level, the friction coefficient will slowly decrease at this time.
(ii) The final friction coefficient will be close to the set value. Therefore, there is a negative exponential decay law between the strength of the structural surface and the relative velocity between the blocks, which is

$$\gamma(t) = P_0 + (1 - P_0)e^{-mg \cdot |v(t)|} \tag{9}$$

Where $v(t)$ represents the relative velocity between the blocks at any time; P_0 represents the convergence value of the relative speed influence coefficient that can be obtained through experimental data. m is the coefficient that can be obtained through experimental data.

Therefore, the vibration degradation coefficient of the structural is related to three factors: relative velocity, the number of cyclic shears and cyclic shear amplitude. It is a dynamic variable during the disturbance process, and its expression is

$$D(t) = \left[P_0 + (1 - P_0)e^{-mg \cdot |v(t)|}\right] \cdot \left\{R_0 + (1 - R_0)e^{-b \cdot J(t)} + (1 - R_0)\left[1 - e^{-b \cdot J(t)}\right]e^{-b \cdot K(t)}\right\} \tag{10}$$

3 Establishment of a fracture model for the tunnel roof

In this study, the tunnel was subjected to dynamic disturbances during the service life, and the disturbance produced a vibration effect on the tunnel. The acceleration amplitude of the vibration changes through three stages: gradually increasing stage, stationary stage and gradual degradation stage. It is a non-stationary random process. It is very difficult to calculate it as a general non-stationary process. At present, the more useful method is to modify a stationary random process by multiplying it by a non-stationary function. Therefore, the modulated random process is obtained to represent the non-stationarity of the vibration. The nonstationary model of uniform modulation is (Yan et al. 2003; Zhao et al.2012)

$$\ddot{u}_g(\mathrm{t}) = \mathrm{g(t)} \cdot \mathrm{A}_g(t) \tag{11}$$

where $\ddot{u}_g$(t) represents the acceleration of non-stationary vibration, g(t) represents the non-stationary intensity function, which is a deterministic function and $\mathrm{A}_g(t)$ represents a random process of stationary vibration.

The vibration intensity envelope function is expressed as (Zhao et al. 2012)

$$g(t) = \begin{cases} \left(\frac{t}{T_1}\right)^2 & amp; 0 \le t \le T_1 \\ 1 & amp; T_1 \le t \le T_2 \\ e^{-C(t-T_2)} & amp; t \ge T_2 \end{cases} \tag{12}$$

where T_1 represents the end of the rising phase of the vibration acceleration, T_2 represents the end of stationary phase of the vibration acceleration, C is the amplitude degradation coefficient.

The shear response of tunnel roof at any time is (Li et al. 1993; Yan et al. 2005)

$$Q(t) = \int_0^t h_Q(t-\tau)\ddot{u}_g(\tau)d\tau \tag{13}$$

where h(t) represents the impulse response distribution function, given as follows (Yan et al. 2005)

$$h(t-\tau) = \begin{cases} \frac{1}{\varpi} \cdot e^{[-\delta_j \omega(t-\tau)] \cdot \sin[\varpi(t-\tau)]} & t \ge \tau \\ 0 & t \le \tau \end{cases} \tag{14}$$

where δ_j represents the system damping ratio, ω represents the system self-resonant frequency, ϖ represents the system damping frequency, and $\varpi = \omega\sqrt{1-\delta_j^2}$. The shear response expectation of a tunnel at any time can be expressed as

$$\begin{aligned} \mathrm{u}_Q(\mathrm{t}) &= \mathrm{E[Q(t)]} \\ &= \int_0^t E[h_Q(t-\tau)] \cdot E[\ddot{u}_g(\tau)]d\tau \\ &= \int_0^t E[h_Q(t-\tau)] \cdot E[g(\tau)] \cdot E[A_g(\tau)]d\tau \end{aligned} \tag{15}$$

The shear response variance of a tunnel at any time can be expressed as

$$D_Q(t) = E\{[Q(t) - \mu_Q(t)]^2\} \tag{16}$$

Substituting (12) into (15) gives

$$\begin{aligned} D_Q(t) = \int_{-\infty}^{\infty}\int_{-\infty}^{\infty} & h_Q(h-\delta) \cdot h_Q(t-\eta) \cdot g(\delta) \cdot g(\eta) \\ & \cdot R_A(\delta-\eta)d\delta d\eta \end{aligned} \tag{17}$$

where $R_A(\delta - \eta)$ represents the autocorrelation function of $\mathrm{A}_g(t)$.

The bearing capacity limit equation of a tunnel roof during dynamic disturbance can be expressed as

$$Z = R_S \cdot D(t) - \int_0^t h_Q(t-\tau)\ddot{u}_g(\tau)d\tau \tag{18}$$

Fissures will cause groundwater migration due to the influence of mining disturbances, which will eventually affect the stability of the entire tunnel rock mass. The seepage characteristics existing in the fracture can be simulated by the cubic law, which assumes that the fractures surfaces are modelled as two parallel plates. The parallel plate model, developed by the application of the Navier–Stokes equation for laminar incompressible flow between two parallel smooth plates, is applied to calculate the permeability k of the joint (Benjelloun 1991)

$$k = e_h^2/12 \tag{19}$$

In the above formula, e_h represents the hydraulic aperture of the fracture.

Since natural fractures are completely different from idealized parallel plates, the hydraulic aperture of the joint is different from its mechanical aperture (Nguyen and Selvadurai 1998). Elliot et al. (1985) and Witherspoon et al. (1979) gave a linear relationship between the hydraulic aperture and the mechanical aperture:

$$e_h = e_{h0} + f\Delta e_m \tag{20}$$

where e_{h0} and Δe_m respectively represent the initial hydraulic aperture and mechanical aperture, and f represents a proportionality factor. Benjelloun (1991) experimentally verified the validity of Eq. (20) and found that f changes between 0.5 and 1. This factor comes from the roughness of the joint surfaces. when $f = 1$, it is suitable for the limiting ideal situation of parallel smooth plates; this situation prevails only when the joint is relatively open. The aperture is in mm. For most other situations, $f < 1$. At the same time, f is affected by the flow path geometry. For rectilinear laminar flow, f is close to 0.8; for radial flow, f is close to 0.5.

The basic assumptions adopted in this paper when establishing the model are as follows:

1. The rock matrix is a homogeneous and isotropic linear elastic medium, and its deformation belongs to the category of small deformations;
2. No crack propagation occurs in the fracture matrix during the infiltration process;
3. The compressibility of fluid and the thermal effect of the fluid flowing in the fractures is ignored.

This paper refers to the tunnel model and related parameters given in the paper by Wu et al. (2019b). Figure 1a shows the tunnel model of a fractured rock mass under dynamic disturbance. In this model, the external rock block size is 5 × 5 m, and the internal tunnel size is 1 × 1 m to eliminate any boundary effects. There are 4 fractures with a width of 0.01 m near the tunnel roof. The boundary conditions and initial conditions of the model are as follows: the rock block is surrounded by a fixed boundary, and the tunnel is surrounded by a free boundary. The distance between the position of dynamic disturbance and the roof of the tunnel is 0.95 m. Tunnel excavation can cause changes in both the seepage field and water level. In this study, the initial water head of the unexcavated model (initial model) is equal to the surface, with Z = 5 m, while the base surface of the model is chosen as Z = 0. The top surface of the model is a free surface, and the water head is 5 m. The finite element mesh is generated by COMSOL™ Multiphysics finite element software. A free triangular grid is adopted in the finite element method. The predefined element size is set as an adaptive mesh feature, and grid refinement near the fractures guarantees the stability of the calculation. The resulting mesh has 6269 elements, and 25,310 degrees of freedom and is shown in Fig. 1b.

Table 1 shows the correlation coefficient of the rock damage function (Wu et al. 2019a, b).

Table 2 gives the parameters of the fractured rock mass medium used in this study (Chen et al. 2018).

Taking the tunnel roof as a whole and due to the randomness of the dynamic disturbances and the spatial heterogeneity of the underground rock mass, not all the parameters can be uniquely determined. Also the stress state in the tunnel roof is not uniformly distributed. So some parameters are regarded as random variables, as shown in Table 3 (Wu et al. 2019a, b).

The vibration waves caused by the dynamic disturbance showed three distinct stages: rising, stationary and decaying. The disturbance process is divided into 10 periods with each period lasting 0.1 s. The time varying parameters in the different periods as follows (Table 4).

The unit of block relative velocity is cm/s and the unit of cyclic shear amplitude is mm. The fluid–solid coupling model of a rock tunnel under excavation disturbance is established by COMSOL™, and the

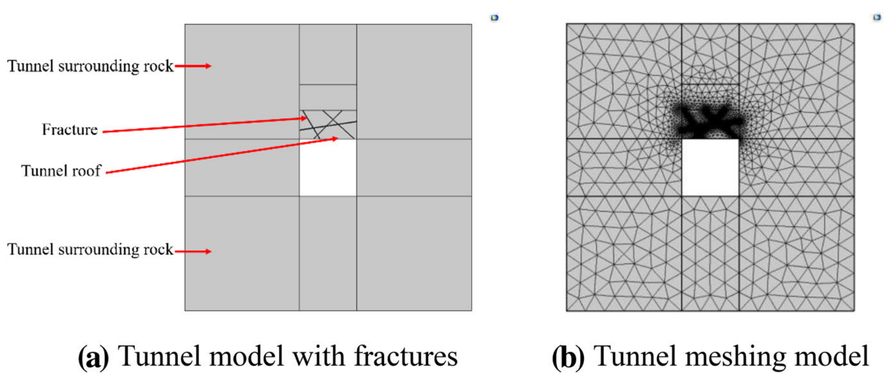

Fig. 1 Tunnel model of fractured rock mass under dynamic disturbance. **a** Tunnel model with fractures, **b** Tunnel meshing model

Table 1 Deterministic parameters

a	b	R_0	m	P_0
0.76	0.15	0.77	0.01	0.76
T_1	T_2	ω_g	δ_g	C
0.25	0.85	16.5	0.8	0.5

The unit of T is s, and the unit of ω_g is rad/s.

Table2 Model calculation physical parameters

Value	Parameter	Unit
ρ	3070	kg/m^3
μ	0.01	$Pa \cdot s$
E	3.658E3	MPa
v	0.3	1
k_R	8E − 10	m^2

damage effect on permeability is considered, which gives the stress changes in the rock tunnel under disturbance loads. Figure 2 shows the stress variation rule of the tunnel structure at different times for a single disturbance. It can clearly be seen that the stress effect generated by a single disturbance goes through the process of rising-stable-attenuation, and the stress change in the tunnel roof position is 0.0297 N/m^2 (0.1 s)-0.0733 N/m^2(0.3 s)-0.0746 N/m^2(0.5 s)-0.0745 N/m^2(0.7 s). At the same time, the stress change corresponding to the disturbed position is 0.0560 N/m^2(0.1 s)-0.1380 N/m^2(0.3 s)-0.1405 N/m^2(0.5 s)-0.1403 N/m^2(0.7 s). Therefore, the stress on the tunnel roof and the disturbed area has gone through three periods of rising-stable-attenuation. At the initial stage of the disturbance, the stress of the tunnel roof and the disturbed area of the tunnel

Table 4 Time-varying parameters

Time period	1	2	3	4	5
The number of loop cuts	10	20	30	40	50
Loop shear amplitude	6	7	9	10	14
Block relative speed	19	23	26	32	37
Time period	6	7	8	9	10
The number of loop cuts	60	70	80	90	100
Loop shear amplitude	15	12	7	5	2
Block relative speed	40	35	20	13	3

changes greatly, and then remains relatively stable in the later stage.

4 Structural reliability analysis

Each reliability index is calculated by the Monte Carlo method (MC method) in this paper. MC method has the characteristics of generality, high precision and intuitionism, and can effectively solve the high-order nonlinear problems (Do et al. 2020; Feng and Zhang 2020; Liu et al. 2020). The main idea of MC method is to conduct multiple random sampling and determine the failure probability and reliability index through a large number of sample values.

When a single disturbance is applied to the tunnel roof, its reliability changes are shown in Figs. 3, 4, 5.

It can be seen from these figures that the structural reliability also experienced a descending—stationary—recovery process during the disturbance process of rising- stationary -decay. However, due to the damage to the structure caused by the disturbance process, the reliability of the structure itself cannot be restored to the initial state because of the effect of disturbance, and the single disturbance has little

Table 3 Uncertainty parameters

Variable	μ	σ	Variable coefficient	Distribution function
a_m	1.01	1.831	0.181	Gaussian distribution
c	2.1	0.56	0.267	Gaussian distribution
φ	30	6.5	0.217	Gaussian distribution

Here, a_m is the peak acceleration with units cm/s^2. The unit of c is MPa.

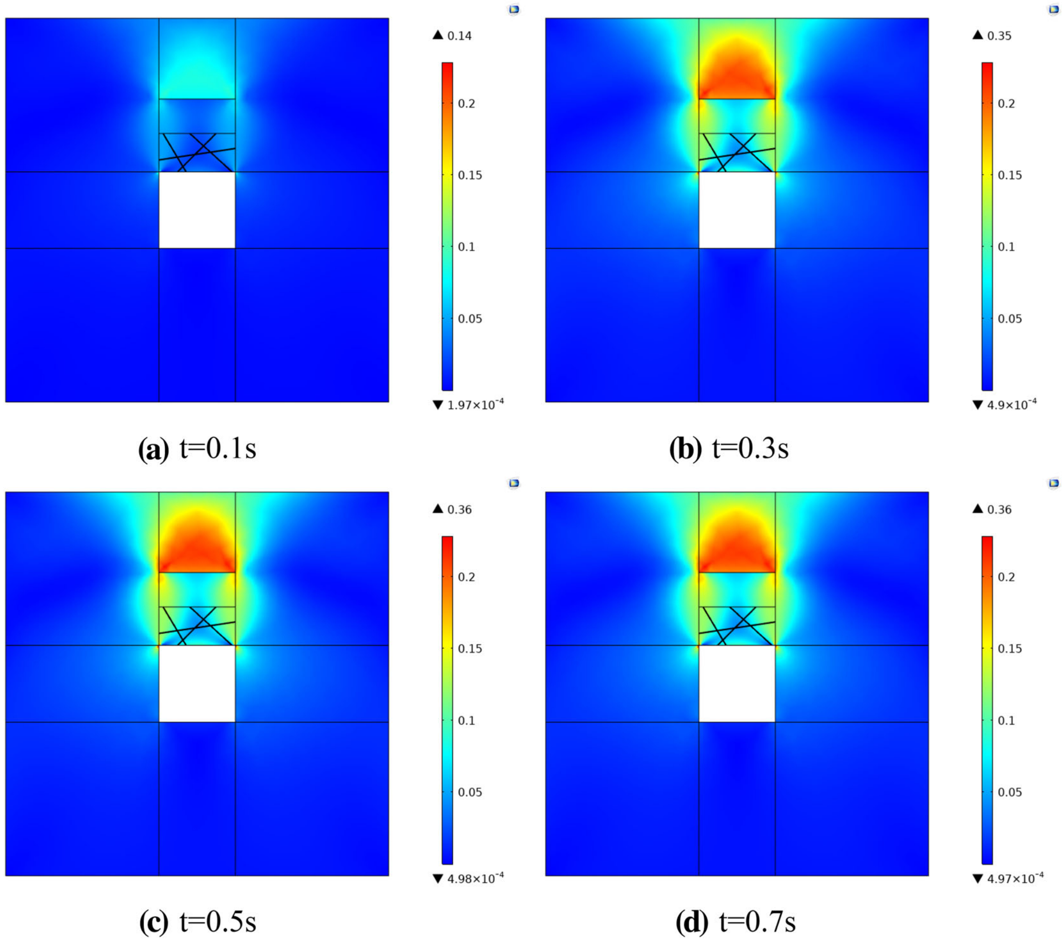

Fig. 2 Stress variation diagram of the tunnel structure (N/m^2). **a** t = 0.1 s, **b** t = 0.3 s, **c** t = 0.5 s, **d** t = 0.7 s

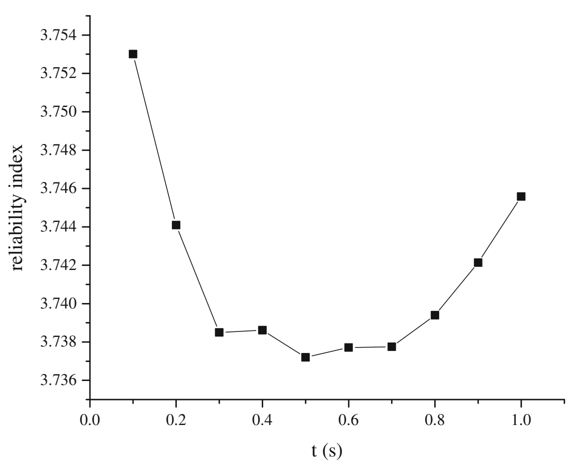

Fig.3 Reliability index of the tunnel roof due to a single disturbance

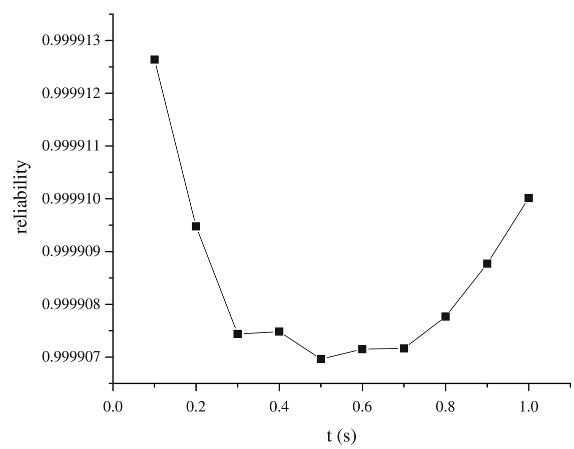

Fig.4 Reliability of the tunnel roof due to a single disturbance

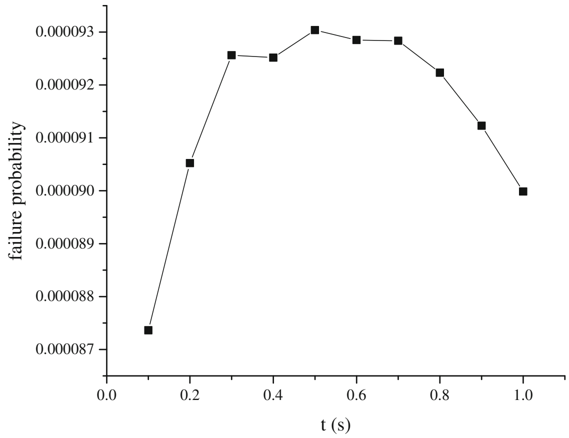

Fig.5 Failure probability of the tunnel roof due to a single disturbance

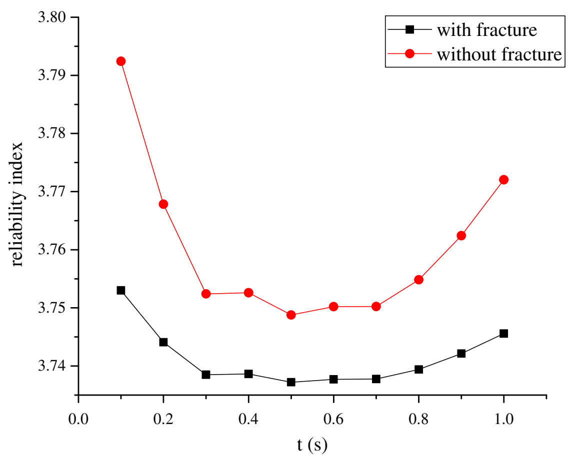

Fig. 6 Changes in structural reliability indexes with and without fractures

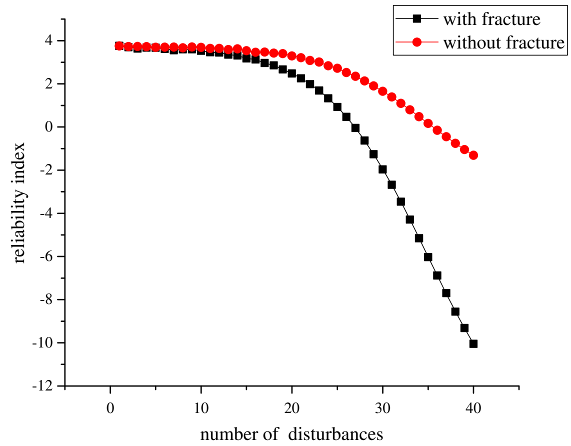

Fig.7 Reliability index of a tunnel roof due to multiple disturbances

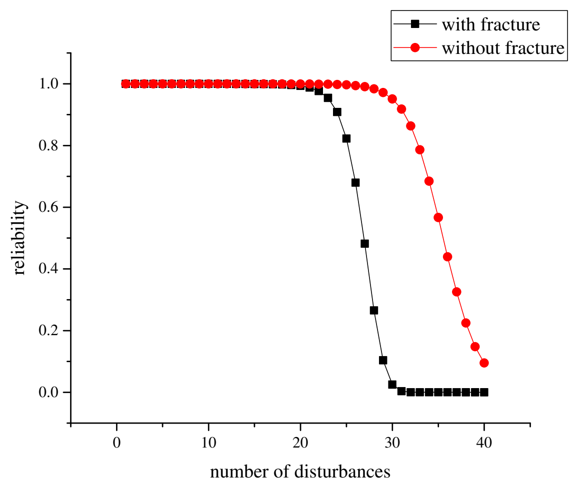

Fig.8 Reliability of a tunnel roof due to multiple disturbances

impact on the reliability of the structure. This article mainly studies and analyzes the impact of cracks, and the reliability index of the comparison between fractures and non-fractures is shown in Fig. 6.

According to the content in Fig. 6, we can know that the reliability of the rock mass will decrease within a certain range due to the influence of cracks. Ignoring the influence of fractures in the rock mass will make the analyzed result higher than the actual situation and put the structure in jeopardy.

During the tunnel use, it will experience more than one disturbance. Even though a single disturbance does not necessarily compromise tunnel safety, it does not mean that the influence of the disturbance on the safety of the structure can be ignored. The reliability changes of the structure due to multiple disturbances are shown in Figs. 7, 8, 9.

It can be seen from these figures that the influence of a disturbance on the safety of a structure increases as the number of disturbances increases. The damage due to a single disturbance to the structure cannot be neglected because it has little impact on the structure's safety; if the necessary repair and reinforcement measures have not been taken, it may lead to a conventional disturbance that causes the structure experience rapid damage. This paper also compares the difference between the analysis with and without considering that the effect of fractures on the structural safety becomes larger as the structural safety decreases. Therefore, it is particularly important to

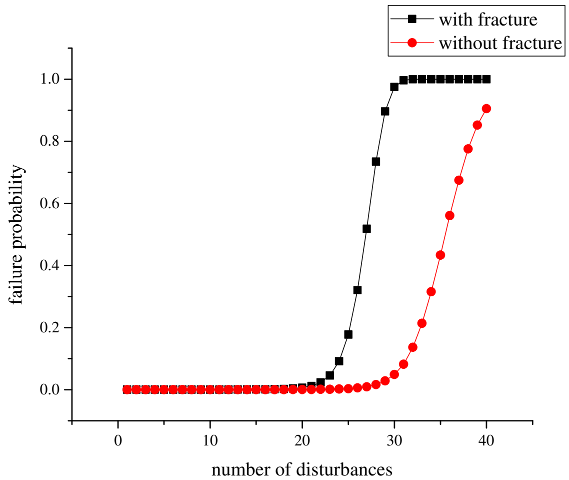

Fig.9 Failure probability of a tunnel roof due to multiple disturbances

consider the impact of fractures when making a safety assessment of tunnel structures.

During the process of damage to the structure caused by disturbances, the fractured state inside the rock mass is not a constant layer. As damage increases, the fracture width expands continuously, as shown in Fig. 10.

From Fig. 10 it is evident that, even if the damage state and stress state of the rock mass are nonlinear, the fracture width changes and can be approximately regarded as a linear opening, and there will be no sudden changes such as failure probability. When the crack width is constantly changing, the permeability of the rock mass will also change accordingly, as shown in Fig. 11.

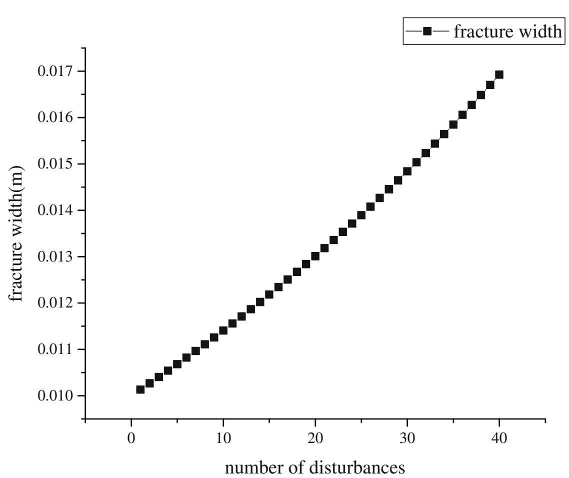

Fig.10 Fracture width due to multiple disturbances

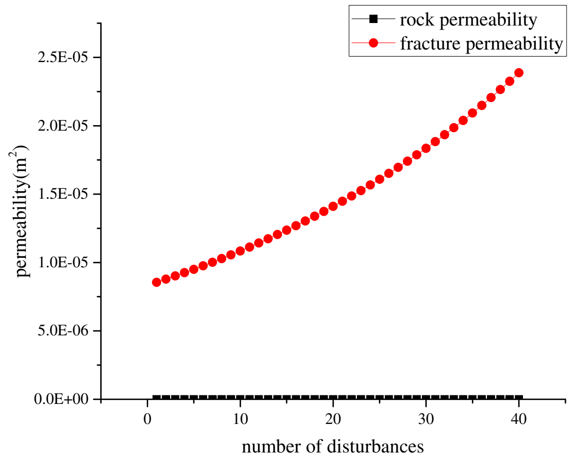

Fig.11 Permeability changes due to multiple disturbances

It can be seen from this figure that the fracture permeability increases continuously as damage to the rock mass proceeds. The fracture permeability increases by about 3 times after multiple disturbances. Thus, it is inaccurate to regard the permeability of the system as a constant value in the study. In the seepage process of the rock mass, the fracture is the primary seepage channel. Compared with the fracture permeability, the permeability of the rock itself increases slowly and can be ignored. Therefore, in the process of studying the permeability characteristics of rock masses, we need to pay attention to the permeability changes caused by fracture damage.

5 Conclusion

This paper studies the damage characteristics of a tunnel roof under dynamic disturbance based on the reliability theory. Get the following conclusion.

(1) A single dynamic disturbance has little influence on the tunnel reliability. During the disturbance, the stress and reliability of the structure go through a rising—stable—falling process. However, due to the influence of damage, the reliability of the structure cannot be restored to its initial state after the disturbance ends.

(2) Although the single disturbance has little influence on the tunnel reliability, the structure will still have a huge potential safety hazard after multiple disturbances during its service life.

When the disturbance reaches a certain level, the failure probability of the structure will suddenly increase sharply. Therefore, in a structural safety assessment, the damage to the structure should not be ignored just because the damage due to a single disturbance is low.

(3) In the seepage of rock mass, the permeability of the rock itself is basically unchanged, and can be neglected compared with that of the fracture. Moreover, the permeability of the fracture is not constant but changes during structural damage. The fracture permeability was seen to increase by three times after multiple disturbances. Therefore, the dynamic permeability characteristics of a rock mass should be considered when analyzing the permeability of the rock mass.

(4) The reliability of a rock mass will be reduced to a certain extent under the influence of fractures. At the same time, with the decrease of structural safety, the difference between the structural safety analysis with and without fractures is also larger. As the damage increases, the fracture width will expand continuously, but this expansion can be approximately regarded as a linear expansion. Therefore, it is particularly important to consider the influence of fractures in the reliability safety assessment of tunnel structures.

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Declarations

Conflict of interest The authors declared that they have no conflict of interest.

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