Physical model experiment research on evolution process of water inrush hazard in a deep-buried tunnel containing the filling fault

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Abstract
In deep tunnel engineering, water inrush disasters caused by filling faults occur frequently, which has generated wide interest in the fields of rock mechanics and fluid mechanics. The rock mass similar material was prepared with river sand as aggregate, cement as a binder, and clay as a regulator, and the similar material of the fault was composed of river sand and gravel, which laid a good foundation for the development of physical model experiment. Then, using the self-designed visualization test system of the two-dimensional model of a deep-buried tunnel filling fault water inrush, four physical models were laid by changing the fault width, fault cross distance, and fault cross angle to study the influence of different hydraulic pressures. In addition, the evolution process of water inrush disaster and the distribution characteristics of seepage weakening failure zone, hydraulic buckling failure zone, and excavation disturbance failure zone were analyzed and discussed. Furthermore, the justification classifications of tunnel risk were established to characterize the process of water-inrush of different schemes for different loading water pressure. The research results further reveal the evolution characteristics of rock fissures, connection, and formation of water inrush channels, and provide an important basis for reducing and controlling the occurrence of such tunnel water inrush disasters.

Keywords Physical model · Filling fault · Water inrush · Evolution process · Deep buried tunnel

Introduction
In underground engineering, rock mass, a kind of natural medium, is often the main bearing structure (Xing et al. 2019; Ju et al. 2020). However, due to the crustal movement, there are various defects in the rock mass, such as joints, fissures, and faults. A large number of research and practice of geotechnical engineering show that the instability characteristics and failure modes of underground rock mass engineering are closely related to joints, fissures, and faults (Qian et al. 2017a, b; Su et al. 2017; Qian et al. 2017a, b; Wang et al. 2019a, b; Deng et al. 2017, 2018; Rehbock-Sander and Jesel 2018). It is of great significance to deeply study the mechanical behavior of rock mass with defects for the stability control of underground rock mass engineering. In particular, the fault is a common geological structure in strata. Due to affected by the fault structure, the rock mass near the fault is usually accompanied by a series of serious engineering geological problems, such as joint and fissure development, weathering, and groundwater development. Under the influence of these adverse factors, there will inevitably be a weak fracture zone with low overall strength and poor stability near the fault, which leads to the difficulty of excavation and support in nearby projects, and has become a prevailing topic for scholars (Zhang et al. 2019; Deng et al. 2018; Dong and Luo 2020; Jiang et al. 2018). Filling fault, a complex fault structure, is a common and dangerous geological body that must be effectively treated in the construction process of rock mass engineering because of its complex
geological origin and the fact that the rock mass inside the fault is generally weak, broken, and easily deformed.

In the world, railways, highways, and other tunnel projects are gradually developing toward large, long and deep. Deep and long tunnels are essential to the construction of infrastructure projects, such as transportation, water conservancy, and hydropower, involving many engineering fields, such as highways, railways, hydropower stations, and cross-basin water diversion. In the construction process of a deep and long tunnel, it is inevitable to encounter the filling fault geological structure which is widely distributed in the stratum (Scibek 2020; Ishibashi et al. 2016). The filling fault, a kind of complex geological structure, possesses a strong hydraulic connection with surface water and underground river due to its loose and broken internal structure, as shown in Fig. 1. Especially when continuous rainfall occurs in the tunnel site area, it is apt to suffer major water inrush accidents with the strong sudden occurrence and large water inrush. Water inrush disaster caused by filling fault has become the main engineering geological hazard encountered in tunnel engineering, which generates wide interest in scholars.

Many experts and scholars have made a series of beneficial achievements in theoretical analysis and numerical simulation (Ren et al. 2020; Wang et al. 2018, 2020; Chen et al. 2015; Xing et al. 2018; Shrestha et al. 2014; Arka et al. 2018; Sainoki et al. 2021). The similar physical model experiment, a method based on a similar theory, is to study the mechanical behaviors and other related characteristics of the corresponding prototype by expanding or reducing the physical model. Gradually, it becomes an important means to solve complex engineering problems. However, there are few physical model experiments on water-inrush of deep-buried tunnel filling fault. In the meanwhile, there is often constant physical and chemical interaction between water and rock mass in the filling fault structure, which could result in constant changes in the mechanical and permeability properties of the rock mass. During the catastrophic evolution of water inrush for deep-buried tunnel filling fault, the occurrence factors of faults and their relative positions to the excavation section have certain effects on the stress field and seepage field of tunnel surrounding rock.

Given this fact, further research on the evolution characteristics of rock fissure formation, extension, and connection for water inrush channels can provide an essential theoretical basis for reducing and controlling the occurrence frequency of tunnel water-inrush disasters. In this article, aiming at filling fault deeply buried tunnel engineering, the physical model test on the water inrush of different width, fault staggered distance, and different fault staggered angle with filling fault has carried on the comparative analysis, through the independent design of deeply buried tunnel filling fault system. In addition, the water-inrush mechanism was discussed to further reveal its characteristics in deeply buried tunnels with filling faults.

**Methodology**

**Experimental preparation**

The self-designed and developed two-dimension model test system for water inrush disasters in deep tunnels was adopted, as shown in Fig. 2. The system is mainly composed of a mechanical loading system (in-situ stress loading system and hydraulic pressure loading system) and data monitoring system, which can realize the loading of models with the size of 1000 × 1000 × 300 mm. Moreover, tests on the water inrush process and progressive failure of tunnels under different overburden loads and water pressures can be carried out, which is significant to reveal the disaster evolution characteristics of water inrush in deep tunnels with filling faults. To accurately measure the pore water pressure and surrounding rock stress at different times and locations in the model, a miniature earth pressure chamber with a diameter and thickness of 1.2–0.8 cm was selected to measure the surrounding rock stress, and a miniature osmometer with a measuring range of 0–100 kPa and a diameter of 5 mm was adopted to measure the pore water pressure.

It is obvious that the selection of similar materials is especially crucial to the success of the model test in the disaster model test of deep tunnel filling fault. Second, similar materials should satisfy both the basic geometric similarity condition of the model test (Eq. 1) and the fluid–structure coupling similarity condition (Eq. 2). This paper takes the physical and mechanical properties of soft surrounding rock of a tunnel in the Qiyue Mountain tunnel engineering geological survey report as the reference object, and obtains mechanical parameters of similar materials in the physical model according to the geometric
similarity principle and fluid–structure coupling similarity theory, as shown in Table 1:

\[
\begin{align*}
C_e & = C_q = 1 \\
C_u & = C_t C_1 \\
C_\sigma & = C_c = C_E = C_r C_1 \\
C_k & = \sqrt{C_l} / C_t
\end{align*}
\] (1)

where \( C_t \) is geometric similarity ratio, \( C_r \) is bulk density similarity ratio, \( C_e \) is strain similarity ratio, \( C_q \) is internal friction angle similarity ratio, \( C_c \) is cohesion similarity ratio, \( C_E \) is modulus similarity ratio, \( C_\sigma \) is stress similarity ratio, \( C_u \) is displacement similarity ratio:

\[
C_k = \sqrt{C_l} / C_t
\] (2)

where \( C_k \) is permeability coefficient similarity ratio.

Considering the previous research results, the river sand, clay, cement, and water were adopted to prepare similar materials for deeply buried tunnel rock mass, in which the river sand as aggregate, cement as the cementing agent, clay as the regulator of fluid–structure interaction similar material. The tests on the mechanical properties and the water property of materials are conducted under the different proportions of clay to accord with the test requirements of a physical model experiment for similar material. To screen out the impurities in the river sand and avoid its influence, ten target quasi screens were used to screen the river sand used in the test. The cement selected in the test was ordinary composite Portland cement with a strength grade of 32.5. The similar material of the fault is composed of river sand and gravel, of which the river sand is selected from the similar material of surrounding rock, and the particle size of gravel ranges from 1.5 to 2 cm. Based on previous similar material configuration experiences, the ratio coefficient of river sand, cement, and water was finally set to be 11:0.7:1.1, and the ratio of similar materials was finally determined by changing the specific gravity of clay and conducting a large number of tests. The preparation process of samples is roughly shown in Fig. 3.

Considering that the basic mechanical properties and hydraulic properties of materials play an important role in the test, they should be tested in detail. Meanwhile, to determine the optimal ratio of similar materials, the basic mechanical properties of similar materials are tested including uniaxial compressive strength, elastic modulus, tensile strength, cohesion, and internal friction angle, and the hydrological properties of materials are tested including permeability coefficient. Basic mechanical parameters such as uniaxial compressive strength, tensile strength, elastic modulus, cohesion, and internal friction angle of the sample were measured using DNS100 electronic universal testing machine developed by Changchun Institute of Mechanical

**Table 1** Physical and mechanical parameters of prototype materials and ideal similar materials

| Material type       | \( P \) (kN/m\(^3\)) | \( \sigma_c \) (MPa) | \( E \) (GPa) | \( \sigma_t \) (MPa) | \( C \) (MPa) | \( \Phi \) (°) | Permeability coefficient (cm/s) |
|---------------------|---------------------|--------------------|--------------|-------------------|-------------|------------|-----------------------------|
| Prototype material  | 26.00               | 38.51              | 4.11         | 5.66              | 8.72        | 32.54      | 4.13 \\times 10^{-5} 5.05 \times 10^{-4} |
| Ideal similar material | 17.81              | 0.2964             | 0.03163      | 0.04354           | 0.06711     | 32.54      | 6.39 \times 10^{-6} 7.82 \times 10^{-5} |

![Overall appearance of the test system](image)

**Fig. 2** Overall appearance of the test system
Science Co., LTD and Variable angle shear instrument, as shown in Fig. 4a, b. The permeability of the sample was tested by the variable head test method with the permeameter, as shown in Fig. 4c. The test results are summarized in Table 2.

As can be seen from Table 2, when the ratio of river sand, clay, cement, and water is 11:0.9:0.7:1.1, the different range of various performance parameters of surrounding rock similar materials and those of ideal conditions is 0.11–7.04%. It shows that the similar material of surrounding rock can satisfy the requirement of the model test.

![Fig. 3 Process of specimen production](image)

![Fig. 4 Experimental devices adopted in obtaining experimental parameters](image)

![Table 2 Performance parameters of similar materials](image)
well. While, when the river sediment proportion of fault material is 25%, that is, the ratio of river sand to gravel is 1:3, the permeability coefficient of similar fault material is only 5.24% different from that of ideal fault material, indicating that similar fault material under this ratio can also well meet the requirements of the model test.

**Experimental schemes**

To compare the water-inrush characteristics of deep-buried tunnel filling faults with different fault widths, different fault intersecting distances, different fault dips, and different water pressure, the basic parameters of each test scheme are shown in Table 3, in which the length and thickness of the fault layer in all test schemes are 35–5 cm, respectively.

During the test, to ensure that the water pressure was uniformly applied to the fault, a gravel sublayer with a thickness of 20 mm and the same length and width as the fault size was laid at the outlet device. The arrangement of pore water pressure and stress measuring points inside the model is shown in Fig. 5. Monitoring sections were set along the axial direction of the tunnel, and stress sensors and seepage sensors were arranged. It should be noted that the 4# seepage sensor in Schemes 1, 2, and 4 and the 5# seepage sensor in Scheme 3 are set to test the attenuation of water pressure in gravel sublayer.

**Experimental process**

The test process is divided into model pouring and model loading.

The pouring process of the model is shown in Fig. 6, and the specific steps are as follows. According to the selected proportion of similar materials, the weights of various raw materials required for each test were weighed, which were 440 kg of river sand, 36 kg of clay, 28 kg of cement, and 44 kg of water. The weighed river sand, clay, and cement were first put into the mixer for stirring, the stirring time was about 10–15 min. After the three solid raw materials are evenly mixed, liquid water is added, and the mixing time is about 20–25 min. The mixed similar materials are spread layer by layer from bottom to top, and roughening treatment is carried out on the contact surface of two batches of similar materials to reduce the influence of layered pouring on the integrity of the model. When similar materials are poured into the specified locations, the burying of test elements and fault structures is initiated. Precise measurements and planning of the burying locations are required before the burying. The fault structure is made using the method of embedded mold. When the mold is taken out, the fault filling, gravel sub-layer, and water outlet device are filled in turn. Continue pouring similar materials until the whole model box is covered, and then pound and scrape the top surface of the model, and then lift and load the uniform beam, jack, and reaction beam in turn. Finally, it was loaded to the initial stress step by step, and the pressure was stabilized for 24 h.

The loading process can be divided into three stages: the initial in-situ stress field application stage, excavation stage, and hydraulic loading stage. In the in-situ stress field application stage, the model is loaded to the in-situ stress state before

| Schemes | Fault width (cm) | Fault intersecting distance (cm) | Fault inclination angle (°) |
|---------|-----------------|---------------------------------|---------------------------|
| One     | 10              | 11                             | 45                        |
| Two     | 20              | 11                             | 45                        |
| Three   | 10              | 16                             | 45                        |
| Four    | 10              | 11                             | 30                        |
excavation. The purpose is to simulate the actual underground engineering process of loading before excavation. The bottom and side of the model are displacement-constrained boundary conditions, and the vertical load is applied to the top of the model. To make the in-situ stress field applied evenly and avoid the damage of embedded test elements caused by the sudden increase of model deformation caused by the one-time application, the initial stress field was applied by the step loading method. After a certain amount of load was applied each time, the data changes of each test element were observed, that is, the stress is completely transferred and balanced in the model, and then the next level of load is applied until the in-situ field is reached.

When the initial ground stress field of the model is applied, the tunnel is excavated. The excavation method is full-face excavation, forming at one time. When the monitoring data of each sensor on the acquisition system are stable, that is, the stress is completely transferred and balanced in the model, and then the next level of load is applied until the in-situ field is reached.

When the initial ground stress field of the model is applied, the tunnel is excavated. The excavation method is full-face excavation, forming at one time. When the monitoring data of each sensor on the acquisition system are stable, the next stage of the operation is carried out. After the completion of the excavation stage, the water pressure loading stage will be started. The loading water pressure will be increased by 5 kPa in each stage. When the monitoring data of each sensor on the acquisition system are stable, the water pressure will be increased to the next level. When the pore water pressure of each measuring point drops obviously and tends to a stable value, the water pressure will continue to increase, and the pore water pressure will remain unchanged, which means that the tunnel water inrush test is over.

**Results and analysis**

**Characteristics of rock mass stress during excavation**

In different test schemes, the stress change characteristics of rock mass in the process of model excavation can be roughly divided into three stages, namely, the stress redistribution stage, the stress adjustment stage and the stress rebalance stage. However, due to the different working conditions of each test scheme, the stress values of rock mass in each stage are different. Taking Scheme 1 as an example, the stress change curve of surrounding rock during excavation is shown in Fig. 7.

As can be seen from Fig. 7, the radial stress of surrounding rock decreases due to excavation. The stress release rate of surrounding rock ranges from 55.58% to 65.89%, followed by that of surrounding rock at two times the diameter of the tunnel, which ranges from 11.76% to 19.94%, and that of surrounding rock at three times the
diameter of the tunnel is the weakest, which ranges from 6.87% to 16.77%. It shows that with the increase of the distance from the tunnel face, the influence of excavation on the change of surrounding rock stress is gradually weakened. Contrary to the variation of radial stress, the tangential stress of surrounding rock increases due to excavation, and the increased amplitude decreases with the increase of the distance from the tunnel section. The tangential stress increase rates of surrounding rock around the tunnel, two times tunnel diameter and three times tunnel diameter are 3.69%, 2.09%, and 2.01%, respectively. Therefore, compared with the tangential stress, the excavation has the most obvious effect on the radial stress of the surrounding rock.

Characteristics of pore water pressure during the hydraulic loading process

To study the variation characteristics of pore water pressure and rock mass stress during hydraulic loading,
representative measurement points in each test scheme were selected for analysis. In each scheme, the selected representative measurement points are all located 5 cm away from the tunnel circumference of each model. The specific values and variation characteristics of each measurement point are shown in Fig. 8.

In Scheme 1, the peak values of pore water pressure and stress are 14.71–72.66 kPa, respectively. In Scheme 2, the peak values of pore water pressure and stress are 11.04–67.43 kPa, respectively. In Scheme 3, the peak values of pore water pressure and stress are 16.62–73.55 kPa, respectively. In Scheme 4, the peak values of pore water pressure and stress are 9.12–62.41 kPa, respectively. Compared with Scheme 1, the peak values of pore water pressure and rock mass stress in Scheme 2 are reduced by 24.95–7.20%, respectively, indicating that the larger the fault width is, the smaller the peak values of pore water pressure and stress are. Compared with Scheme 1, the peak values of pore water pressure and rock mass stress in Scheme 3 increase by 12.98–1.22%, respectively, which indicates that the greater the staggering distance is, the greater the peak values of pore water pressure and stress will be. Compared with Scheme 1, the peak values of pore water pressure and rock mass stress in Scheme 4 are reduced by 38.00–14.11%, respectively, indicating that the smaller the cross angle is, the smaller the peak values of pore water pressure and stress are.

With the increase of loading water pressure, the pore water pressure and surrounding rock stress in each scheme increased first and then decreased. However, in the growth section of pore water pressure, the pore water pressure decreases gradually with the increase of pore water pressure between every two stages of loading water pressure. Taking Scheme 2 as an example, when the loading water pressure...
Pressure increases from 5 to 20 kPa, the increased range of pore water pressure is 180.99%, 95.74%, and 41.36%, respectively. The reason may be that the seepage of water makes the water channel inside the surrounding rock expand continuously, resulting in the permeability of surrounding rock material gradually increasing, and then the increased range of pore water pressure gradually decreasing. Different from the variation of pore water pressure, the stress of surrounding rock increases first and then decreases with the increased amplitude of stress between every two stages of water pressure in the growth section of surrounding rock stress. Taking Scheme 3 as an example, when the loading water pressure increases from 20 to 35 kPa, the increased range of surrounding rock stress is 2.96%, 3.85%, and 2.81%, respectively. The reason may be due to the stress concentration in the process of the continuous expansion of the water channel inside the surrounding rock. In addition, the pore water pressure and stress drop rapidly when the water inrush channel is formed and the filling material gushes out.

**Discussion**

**Characteristics of pore water pressure in outburst prevention rock mass**

This section mainly analyzes the variation characteristics of pore water pressure in the test process before the drop of pore water pressure at each measurement point, which is of great significance for the prevention and control of water inrush disasters.

It can be seen from Fig. 9, with the increase of the loading water pressure, the pore water pressure of each measuring point in the model increases gradually in each scheme, and the increase of pore water pressure increases with the
increase of the distance from the upper boundary of the fault. In Scheme 1, when the loading water pressure increases by 150% (from 10 to 25 kPa), the pore water pressure of 1–4# measuring points increases by 1413.33% (from 0.650 to 9.836 kPa), 881.56% (from 1.498 to 14.702 kPa), 261.27% (from 6.156 to 22.237 kPa) and 166.99% (from 9.205 to 24.586 kPa), respectively. It is worth noting that the measuring point 4# in the model is located at the junction of the gravel sublayer and fault, and its growth rate is only 16.99% different from that of the water outlet device, namely, the loading water pressure. Therefore, the method of embedding gravel sublayer at the water outlet device can not only play the role of water outlet pressure of the water outlet device but also play the role of water outlet pressure of the water outlet device. In addition, the attenuation of the outlet pressure of the outlet device is relatively small. Similarly, in Scheme 4, measuring point 4# is located at the junction of the gravel sub-layer and fault. When the loading water pressure increases from 5 to 15 kPa (the increase rate is 200.00%), the pore water pressure of 4# measuring point increases from 4.87 to 14.821 kPa, the increase rate is 204.08%, and the increase rate is only 4.08% different from that of water outlet device. Therefore, the method of embedding gravel sub-layer in the outlet device achieves the same effect as Scheme 1.

In addition, under the same loading water pressure, with the increase of the distance from the upper surface of the fault, the pore water pressure of each measuring point shows a gradual attenuation trend, and the attenuation amplitude gradually decreases with the increase of the loading water pressure. The reason may be that with the increase of the loading water pressure, the seepage and damage effect of water flow on the rock mass continuously leads to the softening of the internal structure of the rock mass, and some loose and fine surrounding rock particles constantly gush out under the action of water migration, so that the attenuation range of pore water pressure from 4# measuring point gradually decreases.

Figure 10 shows the relationship between pore water pressure and loading water pressure. To study the water inrush characteristics of deep-buried tunnels with different fault widths, the test results of Scheme 1 and Scheme 2 were compared and analyzed. In Scheme 1 with 10 cm fault width, when the loading water pressure increases from 10 to 20 kPa, the pore water pressure increases by 836.65%, 667.30%, 179.24%, and 113.36% at measuring points 1–4#, respectively. While in Scheme 2 with 20 cm fault width, the pore water pressure increases by 223.33%, 176.44%, 127.55%, and 107.85% at measuring points 1–4#, respectively. Compared with Scheme 1, the increase rates are reduced by 5.50%, 51.69%, 490.86%, and 613.32%, respectively. It can be seen that when the loading water pressure increases in the same range, the larger the fault width of the model, the smaller the growth range of pore water pressure of the measuring point at the same relative position with the tunnel. Moreover, the width of the fault has a greater influence on the growing range of the measuring point farther away from the water outlet device.

To study the water inrush characteristics of deep-buried tunnels with different fault crossing angles, the test results of Scheme 1 and Scheme 3 were compared and analyzed. In Scheme 1 with an 11 cm fault crossing distance, when the loading water pressure increases from 20 to 25 kPa, the pore water pressure increases by 61.72%, 27.88%, and 28.66% at measuring points 1–3#, respectively. In Scheme 3 with a 16 cm fault crossing distance, the pore water pressure increases by 95.83%, 51.36%, and 32.51% at measuring points 2–4#, respectively. Compared with Scheme 1, the increase rates are increased by 10%, 12%, and 15%, respectively. It can be seen that when the loading water pressure increases in the same range, the larger the fault intersecting distance is, the larger the growth range of pore water pressure is at the same measuring point relative to the tunnel.

To study the water inrush characteristics of deep-buried tunnels with different fault crossing angles, the test results of Scheme 1 and Scheme 4 were compared and analyzed. In Scheme 4 with a 30 ° fault crossing angle, when the loading water pressure increases from 10 to 15 kPa, the pore water pressure increases by 126.12%, 104.53%, and 95.83% at measuring points 1–3#, respectively. While in Scheme 1 with a 45 ° fault crossing angle, the pore water pressure increases by 344.31%, 424.03%, and 96.79% at measuring points 1–3#, respectively. Compared with Scheme 4, the increase rates are increased by 218.19%, 319.50%, and 0.95%, respectively. It can be seen that when the loading water pressure increases in the same range, the larger the fault intersecting angle is, the greater the growth range of pore water pressure at the same measuring point relative to the tunnel will be.

**Evolution process of water inrush disaster**

The representative water inrush process in Scheme 2 is analyzed. In the process of water pressure loading, according to the real-time observation of tunnel water inrush, the test phenomenon of each model is shown in Table 4.

After the end of tunnel excavation, there was no significant change in the tunnel, as shown in Fig. 11a. When the loading water pressure increases from 5 to 15 kPa, a very small amount of water seepage begins to appear on the right shoulder of the tunnel. Then, when the loading water pressure increases from 15 to 20 kPa, a small amount of water seepage continues to appear on the right shoulder of the tunnel, but a small amount of water seepage begins to flow out of the tunnel mouth and becomes turbid, as shown in Fig. 11b. When the loading water pressure increases from 20 to 25 kPa, the seepage water flowing out of the tunnel mouth...
increases continuously, accompanied by a small amount of fine sand mixture flowing out, which indicates that the water inrush channel is gradually and slowly expanding, as shown in Fig. 11c. When the loading water pressure increases from 20 to 25 kPa, the seepage water flowing out of the tunnel mouth increases continuously, accompanied by a small amount of fine sand mixture flowing out, which indicates that the water inrush channel is gradually and slowly expanding, as shown in Fig. 11c. When loading water pressure increase from 25 to 30 kPa, as the sudden water continues to increase, starting from the tunnel portal has a lot of fine sand and gravel, indicates that water inrush channel at this time is fast, the water flushing action of internal fault fillings, make fillings began to constantly spurt from the tunnel, as shown in Fig. 11d. When the loading water pressure increases from 30 to 35 kPa, the water inflow continues to increase, but the water flow color starts to become lighter but still shows light yellow, indicating that the water inflow channel has developed to a new state, as shown in Fig. 11e. As the loading water pressure continues to increase, naked-eye observation shows that the water flowing out of the tunnel mouth is clear in color and stable in flow rate, indicating that the water-inrushing channel between the tunnel and the water-bearing structure of the fault has been completely and thoroughly connected, as shown in Fig. 11f.

Through evolution process analysis of water-inrush disaster of tunnel modeling, we can find that the failure of the tunnel is of significantly gradual change. And then, the justification classifications of tunnel risk ($R$) were established to characterize the process of water-inrush of different schemes.
for different loading water pressure. As shown in Fig. 12, the values (1–6) are corresponding to the stabilization stage after excavation, little water seepage stage, stream water outflow stage, the outflow stage of water and mud mixture, the outflow stage of water, mud, and gravel, and massive water inrush stage, respectively. When \( R \) is less than or equal to two, the tunnel is in a stable stage. In addition, when \( R \) is among three and five, the tunnel is in the position of inrush failure stage, as the water carries mud and gravel out of the tunnel. When \( R \) is equal to six, the path of water inrush is formed in the surrounding of the tunnel. Thus, the justification classifications of tunnel risk, summarized through physical modeling, could qualitatively conduct the engineering construction.

As we can see from Fig. 12, when loading water pressure is 25 kPa, compare the \( R \) of Model-1 and Model-2. The \( R \) values of Model-1 and Model-2 are two and three, respectively. In terms of experiment results, the failure pressure of Model-2 inrush is less than that of Model-1, which concludes that the larger the width of fault is, the easier inrush occurs. Comparing the \( R \) values between Model-1 and Model-3, \( R3 \) is less than \( R1 \), which indicates that the greater the fault cross distance is, the higher the water pressure of inrush is. Similarly, at the same loading water pressure, \( R4 \) is obviously greater than \( R1 \), which means that the smaller the fault cross radiant is, the easier the inrush occurs.

According to the model tests under the four schemes, the water inrush phenomenon is the same, but due to the different working conditions of each model, the loading water pressure and the final failure mode of tunnel surrounding rock are different when the water inrush occurs. The failure modes of the four scheme models are shown in Fig. 13. It can be seen from Fig. 13 that in four schemes, the failure mode of the outburst prevention rock mass can be roughly divided into three parts, namely, the seepage weakening failure area, the hydraulic buckling failure area, and the excavation disturbance failure area. Among them, the excavation disturbance failure area is mainly affected by the excavation disturbance, the range of hydraulic buckling failure zone is mainly affected by water pressure erosion, and the seepage weakening failure zone is mainly affected by water weakening. The statistics of damaged area for each model are shown in Fig. 14.

Figure 13 gives the failure area range of each model. In the final failure modes of different scheme models, the scope of seepage weakening failure zone excavation disturbance failure zone and hydraulic buckling failure zone decreases successively. Taking Scheme two as an example, the damage ranges of seepage weakening failure area are 127.28, 203.12, 224.85, and 168.13 cm² respectively. Compared with the fault activation zone, the excavation disturbance failure zone is reduced by 66.60%, and the hydraulic buckling zone is reduced by 45.10% compared to the excavation disturbance zone.

In the final failure mode of different models, the permeability of model three is the largest, followed by model two and model four, while model one is the smallest. Compared with Scheme 1, Scheme 2, Scheme 3, and Scheme 4, the scope of seepage weakening failure zone area are 127.28, 203.12, 224.85, and 168.13 cm² respectively. Compared with Scheme 1, the scope of the seepage weakening failure zone of Scheme 2 is increased by 59.58%, that of Scheme 3 is increased by 76.66%, and the scope of the

| Water pressure (kPa) | Scheme 1 | Scheme 2 | Scheme 3 | Scheme 4 |
|----------------------|----------|----------|----------|----------|
| 5                    | No obvious phenomenon | No obvious phenomenon | No obvious phenomenon | The outflow of a small amount of water |
| 10                   | No obvious phenomenon | No obvious phenomenon | No obvious phenomenon | Stream water outflow |
| 15                   | Very little water seepage | The outflow of a small amount of water | No obvious phenomenon | The outflow of water and mud mixture |
| 20                   | The outflow of a small amount of water | Stream water outflow | Very little water seepage | The outflow of water, mud, and gravel |
| 25                   | Stream water outflow | The outflow of water and mud mixture | The outflow of very little water | Massive water inrush |
| 30                   | The outflow of water and mud mixture | The outflow of water, mud, and gravel | Stream water outflow | – |
| 35                   | The outflow of water, mud, and gravel | Massive inrush | The outflow of water and mud mixture | – |
| 40                   | Massive inrush | – | The outflow of water, mud, and gravel | – |
| 45                   | – | – | Massive water inrush | – |
seepage weakening failure area of Scheme 4 is increased by 32.09%.

The range of excavation disturbance failure zone and hydraulic buckling failure zone in different models is the largest of model four, followed by model two, model one, and model three is the smallest. The range of excavation disturbance and hydraulic buckling failure zone of model 1, 2, 3, and 4 are 27.93 and 25.28 cm², 67.85 and 37.25 cm², 17.30 and 13.75 cm², 74.08 and 48.17 cm², respectively. Compared with model one, the range of excavation disturbance and hydraulic buckling failure zone of model two is increased by 142.84% and 47.35%, decreased by 38.08% and 45.61% in Scheme 3, and increased by 165.14% and 90.55% in Scheme 4.

Conclusions

The deep-buried tunnel with filling fault was taken as the research object, and the physical model tests were used to conduct a comparative study on the water inrushing disaster evolution characteristics of a deep-buried tunnel under different fault widths, different fault crossing distances, and different fault crossing angles. The main research results are as follows:
1. During the excavation process, the tangential and normal stress changes of surrounding rock undergo three stages of stress redistribution, stress adjustment, and stress rebalancing, and the excavation results in the reduction of radial stress of surrounding rock, while the tangential stress of surrounding rock increases.

2. In the process of water pressure loading, the pore water pressure and stress of surrounding rock changes are the trend of first increases and then decreases, and the width of the fault model of the pore water pressure and the greater the stress peak value is smaller, staggered distance model of the pore water pressure and the greater the peak stress, the greater the crisscross Angle, the smaller the model of the pore water pressure, and the stress peak value is smaller.

3. In the process of water pressure loading, the final failure mode of rock mass in the physical model is composed of three parts: infiltration weakening failure zone, hydraulic buckling failure zone, and excavation disturbance failure zone, and is different due to the influence of fault layout.

4. The justification classifications of tunnel risk ($R$), summarized through physical modeling, could qualitatively conduct the engineering construction. The greater the $R$ value is, the easier the inrush of the tunnel occurs.

5. Under the same loading water pressure, the pore water pressure and the decrease of the amplitude in the same point of four models are negatively related fault width and thickness of the fault, the positive correlation between fault staggered distance and fault staggered angle and relationship, and the model of the diffusion distance as far as the width and thickness of fault and fault were positively related, and negatively correlated with fault staggered distance and fault staggered angle.
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Declarations

Conflict of interest The authors have not disclosed any competing interests.

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