

STABILITY IMPLICATIONS OF NON-PERSISTENT JOINTS IN LARGE-SECTION TUNNEL SURROUNDING ROCK

Jianfeng Chen, Xiamen R&B BAICHENG Co.,Ltd, Xiamen, China,
e-mail: chenjianfengt@163.com

Abstract

In the construction of large-span tunnels, the degree of joint development within the surrounding rock demonstrates heightened sensitivity and exerts a critical influence on the stability of the rock mass. To gain deeper insight into the influence of non-persistent joints on the stability of surrounding rock in large-span tunnels, a reliable discrete element numerical model was established based on the referenced tunnel project. By adopting the area of shear slip zones as a stability criterion, the safety factor of the surrounding rock was evaluated, and the effects of joint trace length, rock bridge ratio, and relative rock bridge dip angle on surrounding rock stability were further analysed. The results indicate that the safety factor first increases and then decreases with increasing joint trace length. Variations in the rock bridge ratio lead to changes in crack propagation modes: when the rock bridge length is smaller than the joint spacing, the stability of the surrounding rock is only slightly affected. The influence of the relative rock bridge dip angle on tunnel stability is comparatively minor. These findings may provide valuable reference for the design and construction of large-span tunnels.

Keywords: large-section tunnel; non-persistent joint; rock mass stability; DEM; numerical simulation.

1. INTRODUCTION

Owing to the steady growth in transportation demand, tunnel construction faces growing challenges from deep overburden, enlarged cross sections, and highly complex geological settings^[1]. Full-scale cutting tests in composite soft–hard rock strata have also shown that geological heterogeneity can produce pronounced variations in excavation-induced force responses, further increasing the complexity of tunnel construction under heterogeneous geological conditions

[2]. The continuous increase in tunnel cross sectional area usually leads to a greater number of joints within both the excavation surface and the surrounding rock mass^[3]. Beyond the inherent requirement of long self-stabilisation time and the difficulty of deformation control in large span tunnels, their behaviour is strongly influenced by joint properties^[4]. This situation inevitably compromises both the safety of construction processes and the economic rationality of support structure design^[5].

As a fundamental structural feature of natural rock masses, joints tend to reduce their overall strength and impart significant anisotropy^[6]. Joint location can significantly affect rock deformation, crack propagation, failure mode, and strength^[7]. To elucidate the influence of joints on the mechanical properties and failure patterns of rock masses, extensive investigations have been conducted by numerous researchers through laboratory experiments. Results from direct shear tests on rock mass specimens demonstrate that the mechanical behaviour of rock masses containing through-going joints is directly governed by the shear characteristics of the joints^[8]. For single-jointed rock specimens, failure is predominantly governed by surface abrasion along the joint plane^[9]. In contrast, for specimens containing double parallel joints, stress concentrations are mainly distributed within the intervening rock between the joints, and the failure of this interlayer plays a critical role in controlling the shear failure of the rock mass^[10]. Monotonic compression tests on jointed rock masses reveal that the inclination angle of joints is the dominant factor governing their mechanical behaviour. With increasing joint inclination, both the strength and elastic modulus of the rock mass exhibit a “V-shaped” trend, initially decreasing and subsequently increasing^[11-12]. The study by Yang et al. (2017) further revealed that rock mass strength increases with increasing joint spacing, and is also enhanced as the degree of joint overlap rises^[13]. Chen et al. (2012) reported that with increasing joint persistence, both the ductility index of the rock and its ultimate peak strain increase^[14]. Jia et al. (2025) found that the influence of joint density on the strength and elastic modulus of rock masses follows a power-law decay relationship^[15]. Moreover, variations in joint inclination induce significant changes in the form of the stress–strain response and the eventual failure mode of the rock mass^[16-17].

Despite the relatively comprehensive body of laboratory-scale studies on jointed rock masses, it is important to recognise that the joint orientations and distributions present in actual engineering rock masses are considerably more complex than those assumed in controlled experiments^[18]. In addition, the inherent scale effect between laboratory specimens and field-scale rock masses makes the straightforward extrapolation of laboratory-derived laws to engineering practice unrealistic^[19]. Owing to the significant progress in computational

technology, numerical simulation has become extensively employed in the design, construction, and research of underground engineering projects [20-22]. By employing finite element method (FEM) analysis of the displacement field surrounding tunnels, Abbas et al. (2024) revealed that the existence of an individual joint serves as a concentration point for stress release, which in turn heightens the risk of abrupt tunnel failure^[23]. Dong et al. (2023) employed the discrete element method (DEM) and revealed that when the joint inclination is relatively small, the failure of the surrounding rock is predominantly characterised by bending at the tunnel crown^[24]. In contrast, when the inclination exceeds 60°, shear slip becomes the dominant failure mode. Moreover, variations in joint inclination and position lead to a pronounced asymmetry in tunnel deformation^[18]. With the increase in joint spacing, the bearing capacity of the tunnel initially decreases, then increases, and ultimately approaches that of intact surrounding rock^[24-25]. An increase in joint length leads to a reduction in the bearing capacity of the tunnel. With the progressive increase in in-situ stress, the existence of joints significantly enhances the likelihood of rockburst occurrence^[26].

In summary, it is noteworthy that existing studies on the influence of joints on tunnel surrounding rock have largely concentrated on through-going joints, whereas the mechanical and stability implications of joints arranged in a spatially intersecting three-dimensional network remain insufficiently addressed. In practical tunnel excavations, rock masses frequently contain numerous discontinuous joints^[27]. The mechanical response associated with discontinuous joints is considerably more complex than that observed in through-going joints^[1]. To this end, this study is based on the widening section of a port-access tunnel in Xiamen, Fujian Province, China. Using the discrete element code 3DEC, this study systematically investigates the effects of joint trace length, rock bridge ratio, and relative rock bridge dip angle on the stability of large-span tunnel surrounding rock. The main contributions are the identification of the scale-dependent relationship between the peak safety-factor increment and tunnel span, the quantitative comparison of the sensitivities of the three joint parameters, and the clarification of the associated rock-bridge failure mechanisms. These findings provide a reference for stability assessment and construction design of similar large-span tunnels in jointed rock masses.

2. DEM NUMERICAL SIMULATION

Originally developed by Cundall and Strack (1979), DEM conceptualises the simulated material as an assembly of discrete elements, with inter-element interactions characterised by force–displacement relationships^[28]. The motion of

each element is governed by Newtonian mechanics, thereby ensuring equilibrium of both forces and moments throughout the system. In practical geotechnical engineering, DEM enables more accurate modelling of jointed rock masses, effectively capturing the mechanical properties and deformation characteristics of joints. It provides a more realistic representation of rock and joint surface failure, as well as the nonlinear deformation behaviour observed under field conditions^[29]. Therefore, 3DEC is employed to conduct modelling and analysis of the FC5 widening section.

2.1 Calibration of Rock Mechanics Parameters

The accuracy of DEM numerical results is strongly governed by the calibration procedure^[21]. Rock cores were collected on site and prepared into standard cylindrical specimens with a diameter (D) of 50 mm and a height (H) of 100 mm in accordance with the guidelines of the International Society for Rock Mechanics and Rock Engineering. Monotonic loading tests were conducted under confining pressures (σ_3) of 0 MPa, 5 MPa, and 10 MPa at an axial loading rate of 0.02 mm/min to obtain the rock mechanical parameters. Subsequently, following the standards of laboratory triaxial testing, a cylindrical numerical specimen with D = 50mm and H = 100mm was constructed. The specimen was discretised into approximately 6000 tetrahedral block elements. The block material behaviour was represented by a Mohr–Coulomb elastoplastic constitutive model, while the contacts were characterised using a Coulomb slip model^[30]. The specific details and theoretical basis of the constitutive models are comprehensively described in the manual and will not be reiterated here^[31]. Loading was applied via rigid loading plates, with the lower plate fixed and the upper plate displaced at a rate consistent with that used in the laboratory tests. To facilitate measurement and calculation, monitoring points were placed on both the upper and lower ends as well as at the mid-height of the specimen, allowing for the determination of axial and radial stresses and strains^[32]. The numerical model for specimen loading and the distribution of measurement points are illustrated in Figure 1(a). The mechanical parameters of the blocks were set in accordance with the results of laboratory triaxial tests, whereas the contact parameters between blocks were the focus of the present calibration. The calibration of these parameters was conducted via back analysis, consistent with strategies described in the existing literature^[11,15].

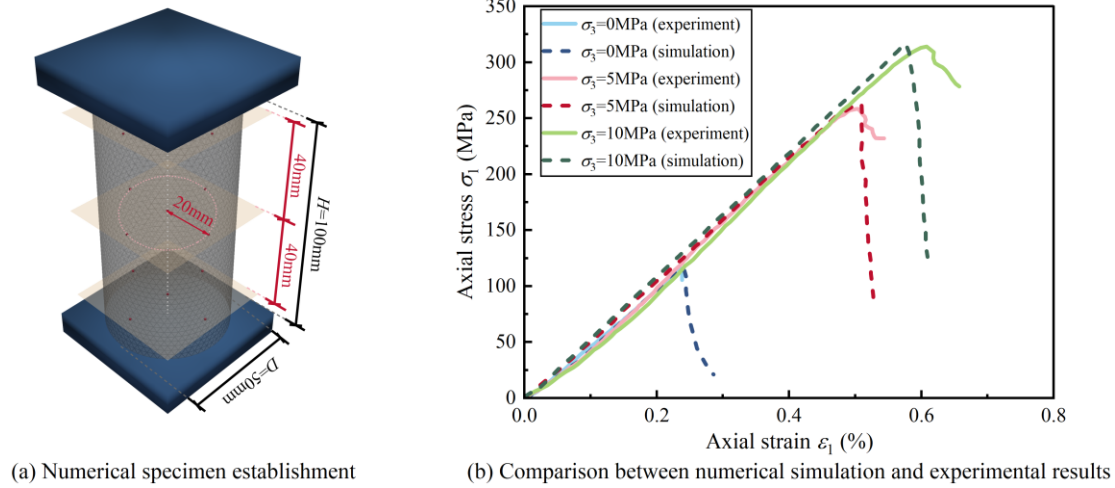


Figure 1. DEM numerical specimen generation and corresponding calibration results.

Following calibration, the mechanical parameters of the blocks and joints are summarised in Table 1 and Table 2, respectively. The stress–strain relationship of the numerical specimen compared with that obtained from laboratory tests is illustrated in Figure 1(b). It can be observed that the calibrated model exhibits a deviation of 0.86% in peak stress, -5.49% in peak strain, and 6.59% in maximum elastic modulus relative to the experimental results. These results indicate that the calibrated parameters for this rock specimen are reasonably accurate.

Table 1. Calibrated mechanical parameters of rock blocks.

Density	Elastic modulus	Poisson's ratio	Cohesion	Friction angle
ρ ($\text{kg}\cdot\text{m}^{-3}$)	E (GPa)	μ	c_R (MPa)	φ_R ($^\circ$)
2643	51	0.23	23.4	59

Table 2. Calibrated mechanical parameters of rock specimen contacts.

Normal stiffness	Shear stiffness	Tensile strength	Cohesion	Friction angle
k_{nC} ($\text{MPa}\cdot\text{m}^{-1}$)	k_{sC} ($\text{MPa}\cdot\text{m}^{-1}$)	σ_{tC} (kPa)	c_C (MPa)	φ_C ($^\circ$)
3.3×10^4	7.6×10^3	14.4	57	19

2.2 Calibration of joint mechanical parameters

The spatial distribution of rock mass joints forms a fundamental basis for evaluating rock mass strength and quality, as well as for conducting discontinuous deformation analyses. Comprehensive data acquisition and statistical characterisation of joint distribution and related details are essential to ensure that joint modelling of the rock mass is consistent with in situ conditions. Accordingly, field measurements of exposed joint surfaces in the surrounding rock were carried out using geological sketching, a compass, and a hammer. Joint development data from the tunnel face and sidewalls were subsequently obtained and represented in CAD for descriptive and analytical purposes. Based on this analysis, the joint characteristics of the FC5 widening section were summarised, providing essential input for model construction and for the calibration of joint mechanical parameters. The principal joints at the tunnel face and those along the sidewalls of the surrounding rock are illustrated in Figure 2. The dominant joint sets identified from the field survey are summarised in Table 3.

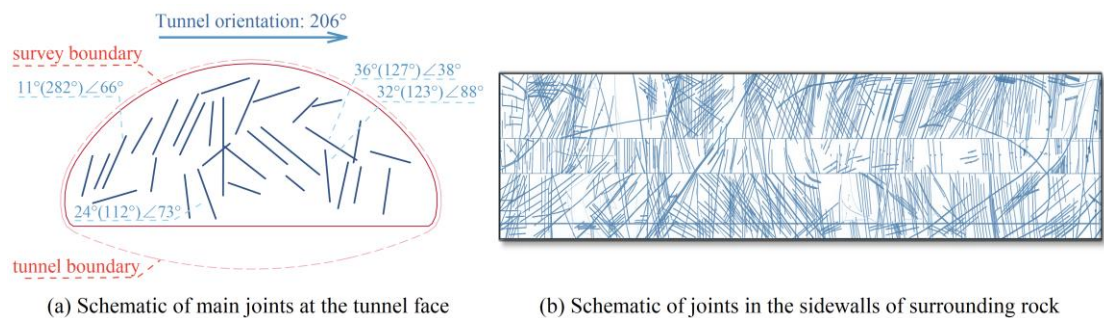


Figure 2. Schematic illustration of joints in the FC5 tunnel section.

Table 3. Summary of field-determined dominant joints.

Group ID	Dominant orientation	Strike variation range	Dip variation range
I	123°∠88°	92° ~ 156°	38° ~ 90°
II	295°∠76°	272° ~ 350°	72° ~ 90°
III	210°∠87°	191° ~ 233°	79° ~ 90°

Figure 3 illustrates the probability distributions of the visible trace lengths of the principal joints and their spacing. According to the probability distributions in Figure 3, joints exhibiting longer trace lengths may be reasonably simplified for modelling purposes, with certain traces treated as through-going joints. The average spacing of joints was found to be about 1.36 m, and a spacing of 1.5 m was therefore selected to facilitate numerical simulation. The consolidated joint data offer a robust reference for subsequent model construction, thereby improving the reliability of the simulation results.

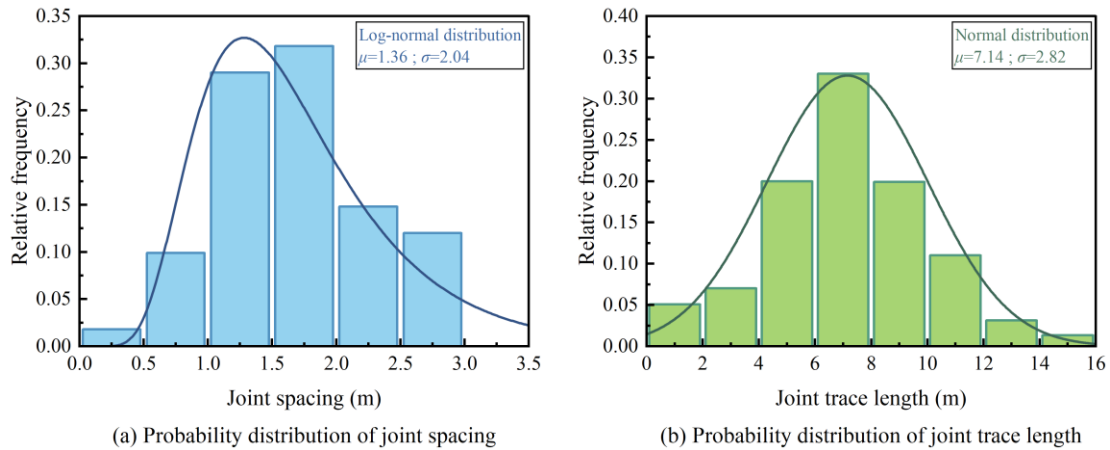


Figure 3. Probability distributions of joint statistics.

Based on the actual engineering conditions of the referenced tunnel project, the established numerical model represents a flat, large-span highway tunnel with a span of 30 m, a height of 17 m, and a flatness ratio of approximately 0.57. The tunnel is buried at a depth of 80 m, with a total excavation length of 50 m, and an excavation step of 5 m. The lateral and bottom boundaries were set at three times the tunnel span^[33], resulting in a computational model with dimensions of 210 m × 200 m × 50 m (width × height × depth). The maximum mesh size was restricted to 3 m, producing approximately two million elements. For the boundary conditions, the model top was treated as a free surface to represent the ground surface, so only gravitational loading was considered in the simulation, while the lateral and bottom boundaries were constrained by fixing the normal velocity to zero. According to the obtained spatial distribution data of the joints, the joint network was simplified to two dominant sets, with an average spacing of 1.5 m: joints striking in the X-direction with a dip angle of 82°, and joints striking in the Z-direction with a dip angle of 65°. The final numerical model of the tunnel is shown in Figure 4.

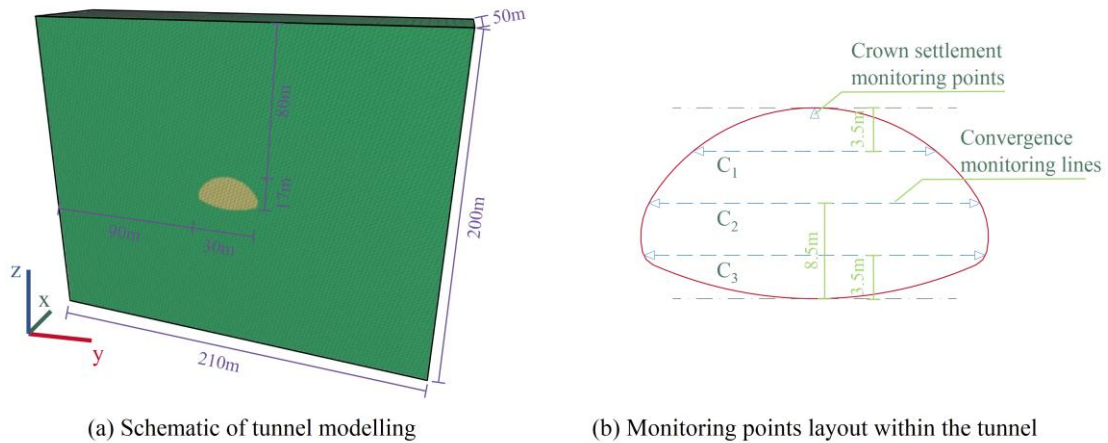


Figure 4. Large-span tunnel model incorporating joints.

Excavation of the FC5 large-span model was simulated using the three-bench seven-step method. The mechanical parameters of the joints, as determined from the experimental data extensively validated by Shreedharan and Kulatilake and subsequently refined through back analysis, are summarised in Table 4^[34]. The deformation of the surrounding rock was assessed by monitoring the crown settlement and peripheral convergence. A comparison between the numerical results and the field monitoring data is presented in Figure 5. It can be observed that the final crown settlement and the convergence along the peripheral monitoring lines are in close agreement with the in situ measurements. The maximum convergence occurs at the shoulders, while the convergence at the haunches and the feet is relatively small. The numerical results also reproduce the overall evolution trends observed in the monitoring data.

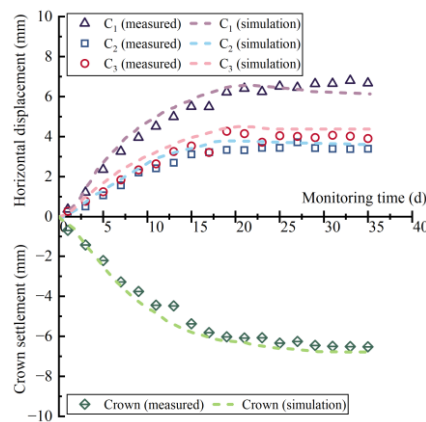


Figure 5. Numerical simulation results versus in situ monitoring of tunnel crown settlement.

Table 4. Calibrated mechanical parameters of joints.

Normal stiffness	Shear stiffness	Tensile strength	Cohesion	Friction angle
k_{nJ} (MPa·m ⁻¹)	k_{sJ} (MPa·m ⁻¹)	σ_{tJ} (kPa)	c_J (MPa)	φ_J (°)
6.4×10^3	2.7×10^3	5	0.4	20

To further quantify the agreement, the coefficients of determination, MAE, and RMSE were calculated for the three convergence lines and crown settlement, as summarised in Table 5. The R² values ranged from 0.858 to 0.959, while the MAE and RMSE values remained below 0.5 mm. These results indicate that the numerical model satisfactorily reproduces both the deformation trends and magnitudes observed in the field.

Table 5. Error metrics for model validation.

Monitoring item	(R ²)	MAE	RMSE
C1	0.908	0.297	0.323
C2	0.858	0.420	0.465
C3	0.959	0.351	0.394
Crown settlement	0.958	0.329	0.377

2.3 Stability criterion for the surrounding rock

The factor of safety was determined using the strength-reduction method, and the critical state was identified from the development of the shear-slip zone. Conventional indicators of tunnel instability commonly include surrounding-rock displacement and plastic-zone extent. However, displacement thresholds are strongly affected by the elastic modulus, tunnel geometry, and monitoring position, while plastic-zone criteria are primarily developed for continuous or homogeneous media. These indicators therefore have limitations when applied to jointed rock masses, in which failure is mainly controlled by slip and coalescence along discontinuities.

In the DEM model, the deformation zones can be classified into joint-opening, shear-slip, and block-detachment zones. During progressive failure, joint opening and local detachment develop only within limited regions, whereas the shear-slip zone expands rapidly when cracks coalesce and blocks undergo relative movement. The rapid expansion of the shear-slip area therefore represents the transition from local joint damage to large-scale kinematic instability. Accordingly, the strength-reduction factor corresponding to the sudden expansion of the shear-slip zone was taken as the factor of safety. This criterion is

particularly suitable for the joint-controlled failure mechanism considered in this study, although it should be regarded as a numerical indicator rather than a universal stability criterion.

3. EFFECT OF DISCONTINUOUS JOINT TRACE LENGTH

In the simulation of discontinuous joints, two joint sets were introduced to systematically segment the intact rock mass, adopting a uniform distribution commonly employed in numerical modelling to construct the discontinuous joint network^[35]. The segments corresponding to joint traces were assigned the calibrated joint mechanical parameters, while the rock bridge segments were assigned the bonded parameters of intact rock, thereby representing these zones as continuous portions of the rock mass. To investigate the influence of different joint dip angles and joint distributions, numerical simulations were conducted for representative configurations of single and double joint sets. Specifically, four distributions were considered: X-direction joints with a dip angle of 80° , XY double joints with a dip angle of 60° , XZ double joints with a dip angle of 60° , and YZ double joints with a dip angle of 80° . Joint trace lengths were selected based on the observed distribution in the rock mass, primarily ranging from 0.5 m to 20 m. Accordingly, trace lengths of 1 m, and from 2 m to 20 m in 2 m increments, were adopted, resulting in a total of 11 models for the computation of safety factors. To isolate the effects of rock bridge proportion and relative rock bridge dip, a compromise value of 0.5 was adopted for the rock bridge ratio, while the relative dip was set to 90° . Partial models for discontinuous joints with different trace lengths, under a fixed rock bridge ratio of 0.5, are illustrated in Figure 6.

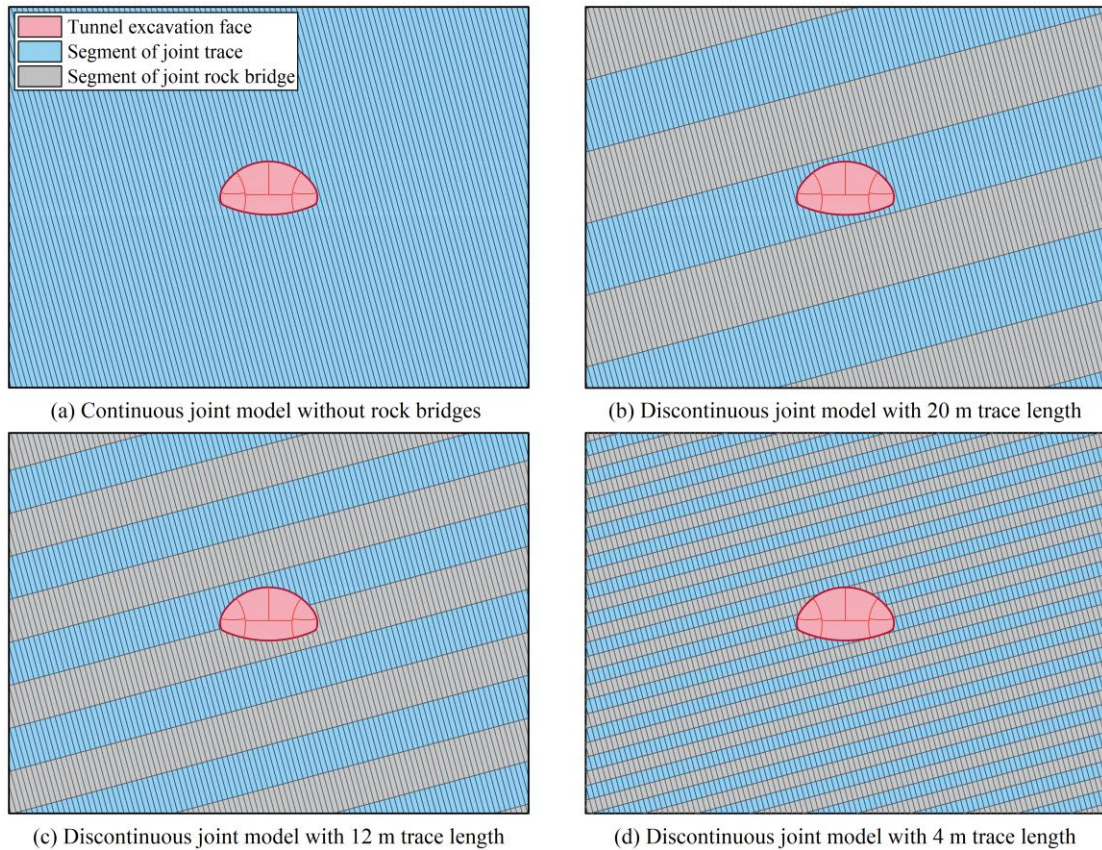


Figure 6. Schematic of discontinuous joint models with varying trace lengths.

The area of the shear-slip zones was used as a criterion: a sudden increase in the shear-slip zone area was regarded as an indication of tunnel instability, based on which the safety factor was calculated. The curves of safety factors versus joint trace length for the four joint distributions are presented in Figure 7(a). It can be observed that, despite variations in joint strike and dip among the four configurations, the overall trend of safety factor variation with trace length is similar. When the joint trace length is less than 4 m, the equidistant distribution of joints and rock bridges can lead to joint tips propagating toward neighbouring joint segments around the excavation perimeter, potentially causing even relatively short rock bridges to become penetrated^[36]. Locally, under the action of surrounding rock stress, this may induce fracturing of the rock mass and result in minor block collapse. Consequently, variations in joint trace length between 0 and 4 m have only a limited effect on the increase of the safety factor.

For joint trace lengths between 4 m and 14 m, the safety factor increases rapidly, reaching a peak at approximately 14 m. In this range, the length of rock bridges progressively increases, enabling them to sustain higher shear forces. Loosened blocks around the excavation perimeter are supported by deeper rock

bridges, and deformation remains largely confined to the segments corresponding to the joint traces. Crack propagation from the joint tips is minimal, and no significant rock bridge penetration occurs, resulting in an overall stable surrounding rock mass.

For joint trace lengths exceeding 14 m, the length of rock bridges engaged in sustaining the loosening pressure gradually decreases, as the transmission distance of stress within the surrounding rock approaches the limit of the plastic zone. Consequently, the safety factor declines slightly. When the joint trace length reaches sufficiently large values, the rock mass behaviour increasingly resembles that of a mass with through-going joints, and the safety factor approaches a state of negligible growth.

The computed safety factors of the tunnel surrounding rock for varying joint trace lengths were compared with those corresponding to through-going joints having identical strike and dip orientations. This comparison enables the quantification of the increment in safety factor resulting from the presence of rock bridges for each trace length. The methodology for this calculation is presented in Equation (1).

$$\gamma = \frac{FOS_D - FOS_G}{FOS_G} \quad (1)$$

where γ denotes the increment of the safety factor; FOS_D represents the safety factor computed for discontinuous joints; and FOS_G represents the safety factor computed for through-going joints.

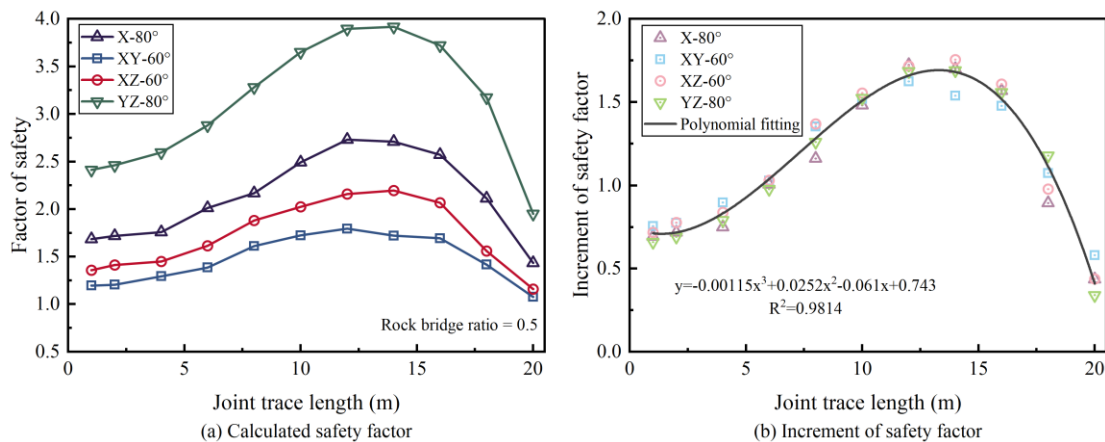


Figure 7. Variation of safety factor with joint trace length.

The computed increments in safety factor were plotted as a scatter diagram in Figure 7(b). It can be observed that the increments for different joint strike and

dip configurations largely coincide, indicating a general trend in the variation of safety factor with joint trace length. A polynomial fitting of the scatter distribution was performed, yielding a cubic function with a coefficient of determination $R^2=0.981$, demonstrating a clear functional relationship between the safety factor increment and joint trace length. According to the fitted polynomial and its corresponding curve, the increment reaches a maximum at a trace length of approximately 14 m, while for trace lengths exceeding 20 m, the increment approaches zero. This is likely because, beyond 20 m, the joint trace extends beyond the plastic zone, and the rock bridges no longer participate in the deformation of the rock mass, exerting negligible influence on joint slip. Consequently, for joint trace lengths greater than 20 m, the surrounding rock stability approximates that of a rock mass with through-going joints. Therefore, the applicability of the fitted function is limited to trace lengths between 0 m and 20 m.

To investigate the distribution pattern of joint trace length corresponding to the maximum increment in the safety factor, the XZ joint set with a dip angle of 60° was selected for analysis under varying tunnel spans. Numerical calculations were performed for flat, large-span highway tunnel models with spans of 12 m, 18 m, 24 m, and 30 m. The computed safety factors are presented in Figure 8(a). It can be observed that although the variation trends differ slightly across spans, the overall stability of the tunnel increases with decreasing span. Nevertheless, all safety factor curves exhibit a distinct peak. Moreover, as the span decreases, the joint trace length at which the maximum safety factor occurs also shifts towards smaller values. This observation suggests that the location of the safety factor peak is closely associated with tunnel span.

Based on the calculated safety factors, the increments were plotted as a scatter diagram (Figure 8(b)). To facilitate the identification of the peak increment, polynomial fitting was applied to the safety factor data for tunnels of different spans, with the corresponding maxima indicated. The results show that the joint trace length corresponding to the peak safety-factor increment shifts with tunnel span, indicating that the peak position is controlled by the relative scale between the joint trace and the tunnel geometry rather than by the absolute trace length alone. To enable comparison among tunnels of different sizes, the joint trace length was normalised by the tunnel span using L_j/B , where L_j is the joint trace length and B is the tunnel span. For the four tunnel spans considered, the joint trace lengths corresponding to the peak safety-factor increments ranged from 46.5% to 52.3% of the tunnel span, with an average ratio of 50.0%. These ratios are annotated at the corresponding peak positions in Figure 8(b).

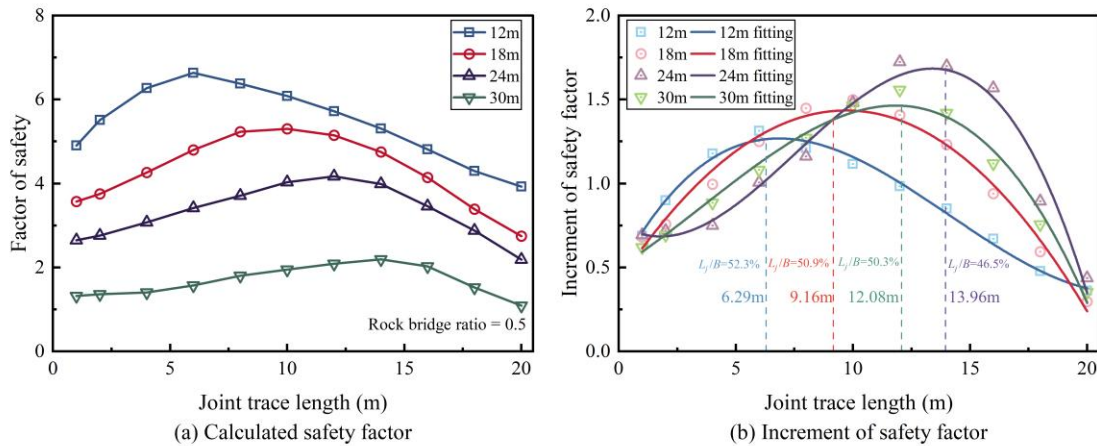


Figure 8. Variation of safety factor with joint trace length for tunnels of different spans.

This peak response results from the competing effects of joint development and rock-bridge support. At relatively small trace lengths, cracks initiated from adjacent joint tips can coalesce readily, and the load-transfer capacity of the rock bridges remains limited. As the trace length increases, the rock bridges become more effective in transferring loads and restraining local block movement, resulting in an increase in the safety factor. However, when the joint trace extends across most of the excavation-disturbed region, the effective rock-bridge length within this region decreases, and the surrounding rock gradually approaches the behaviour of a rock mass containing through-going joints. Consequently, the stabilising contribution of the rock bridges declines.

Therefore, under the tunnel geometries and mechanical parameters considered in this study, the balance between these two effects occurs when the joint trace length is approximately half of the tunnel span. This relationship should be regarded as a scale-dependent trend under the investigated conditions rather than a universal threshold.

4. EFFECT OF ROCK BRIDGE RATIO ON SURROUNDING ROCK STABILITY

The total length of discontinuous joints comprises both rock bridge length and joint trace length, which are generally unevenly distributed. Therefore, the influence of the proportion between rock bridge length and joint trace length on surrounding rock stability must also be considered. Following the same four joint configurations as in the previous section, the total joint length was fixed at 10 m, with a joint spacing of 2 m. Since a rock bridge ratio of 1 corresponds to an

essentially intact rock mass with high strength and a large safety factor, it was excluded from the calculations. Rock bridge ratios ranging from 0.1 to 0.8, at intervals of 0.1, were considered, resulting in eight cases for each joint configuration. An example of the model setup under varying rock bridge ratios is shown in Figure 9. In the figure, white regions represent joints assigned rock contact parameters, while green regions represent joints assigned joint parameters, with the total length of each joint trace and rock bridge segment equal to 10 m.

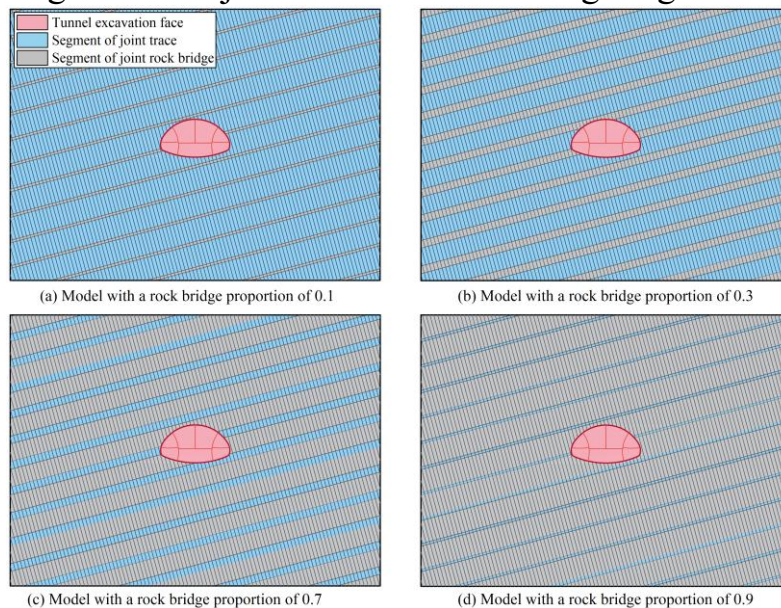


Figure 9. Schematic of tunnel models with different rock bridge ratios.

The curves of safety factor versus rock bridge ratio for the four joint configurations, computed using 3DEC, are presented in Figure 10(a). It can be observed that, as the rock bridge ratio increases, the length of the rock bridges progressively exceeds that of the joint traces, and the surrounding rock approaches the behaviour of intact rock, resulting in a continuous increase in rock mass strength and, consequently, the safety factor. The variation trends of safety factor are similar across the four joint configurations, indicating that the influence of rock bridge ratio on tunnel stability is largely independent of joint distribution. However, at higher rock bridge ratios, the increase in safety factor is limited and does not reach the levels observed for tunnels excavated in intact granite. This is likely because, although the rock bridges are relatively long, cracks may propagate and connect between the tips of adjacent joint segments, causing minor block falls in the surrounding rock, which in turn reduces the computed safety factor.

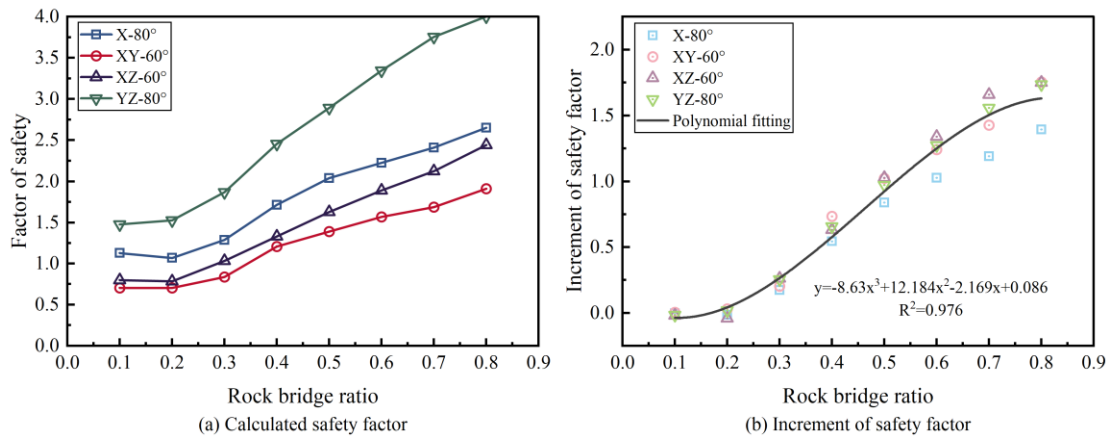


Figure 10. Variation of safety factor with rock bridge ratio.

To more clearly analyse the relationship between rock bridge ratio and safety factor, the increments in safety factor were plotted as a scatter diagram in Figure 10(b). From the distribution of the scatter points, it is observed that when the rock bridge ratio is 0.2 or less, the increment in safety factor is nearly zero, with the points tightly clustered. The negligible increase in safety factor at this ratio can be attributed to the fact that, at a rock bridge ratio of 0.2, the rock bridge length is exactly 2 m, which coincides with the joint spacing of 2 m.

When the rock bridge length is shorter than the joint spacing, cracks at the joint tips develop in a divergent manner. In the numerical model, this manifests as crack propagation both parallel and perpendicular to the joint orientation. At the given dip angle, the propagation rates in these two directions are comparable. However, when the rock bridges are short, the joints tend to connect first in the parallel direction, resulting in the formation of continuous through-going joints. Under such conditions, the strength of the discontinuous joints is nearly equivalent to that of through-going joints, leading to only a marginal improvement in tunnel surrounding rock stability, as illustrated in Figure 11(a).

When the rock bridge length is relatively long, cracks at the joint tips tend to connect first with adjacent joints in the perpendicular direction. As the rock bridge ratio increases, the shear resistance along the joint orientation also grows, leading to a corresponding increase in the safety factor. However, at low rock bridge ratios, close to 0.2, isolated blocks are prone to form around the tunnel periphery, resulting in block detachment. Considering the X-oriented joints with a dip angle of 80° and a rock bridge ratio of 0.2, the displacement contour from the numerical simulation, shown in Figure 11(b), indicates that localized block falls occur at the crown shoulder, where large deformations were observed under through-going joint conditions. The maximum vertical displacement reached 7.88 mm and was concentrated at the crown shoulder, quantitatively confirming the

localized deformation and block detachment in this region. Consequently, at a rock bridge ratio of 0.2, the safety factor exhibits a slight decrease under certain joint configurations.

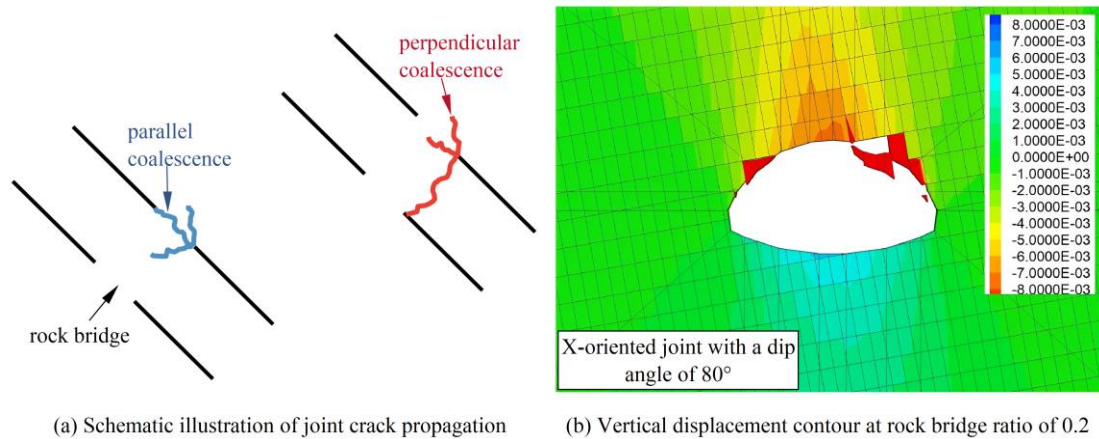


Figure 11. Crack development in surrounding rock with discontinuous joints.

To clarify the relationship between rock bridge ratio and joint spacing at the minimum safety factor, X-oriented joints with a dip angle of 80° were selected, with joint spacings of 2 m, 4 m, and 6 m, resulting in three model configurations. The variation of tunnel surrounding rock safety factor with rock bridge ratio under different joint spacings was computed, and the results are presented in Figure 12(a). The curves indicate that, at the same joint orientation and dip angle, increasing the joint spacing generally leads to a moderate increase in safety factor, which also rises with increasing rock bridge ratio. To facilitate analysis of the variation trend with rock bridge ratio, the computed safety factors were compared with those for through-going joints, and the corresponding increment curves are shown in Figure 12(b). It is evident that, as the rock bridge ratio increases, the curves can be divided into two segments according to the increment, with transition points at 0.2, 0.4, and 0.6, respectively. Prior to the transition points, the safety factor increases at a relatively slow rate, whereas beyond the transition points, the increment rises sharply. This confirms the previously discussed observation that, when the rock bridge length is shorter than the joint spacing, the strength enhancement of discontinuous joints relative to through-going joints is limited.

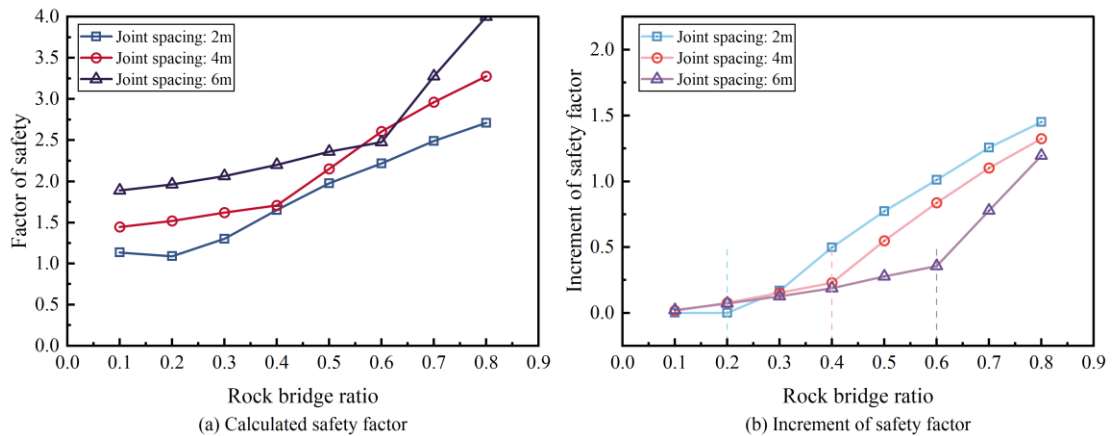


Figure 12. Variation of safety factor with rock bridge ratio for different joint spacings.

5. EFFECT OF RELATIVE ROCK BRIDGE DIP ANGLE ON SURROUNDING ROCK STABILITY

In the analysis of the effects of discontinuous joint trace length and rock bridge ratio on tunnel surrounding rock stability, the relative rock bridge dip angle was initially fixed at 90° for computational convenience. However, in practical conditions, a relative dip angle of 90° represents an idealised scenario, and it is therefore necessary to consider a range of relative rock bridge dip angles. The relative rock bridge dip angle is defined as the angle between the line connecting the tips of two adjacent joints and the joint orientation, and is never equal to 0° . Accordingly, four joint orientations were selected, with joint trace length fixed at 8 m and rock bridge ratio at 0.5. Relative rock bridge dip angles from 10° to 90° were considered at 10° intervals. Partial models for X-oriented joints with a dip angle of 80° under different relative rock bridge dip angles are shown in Figure 13.

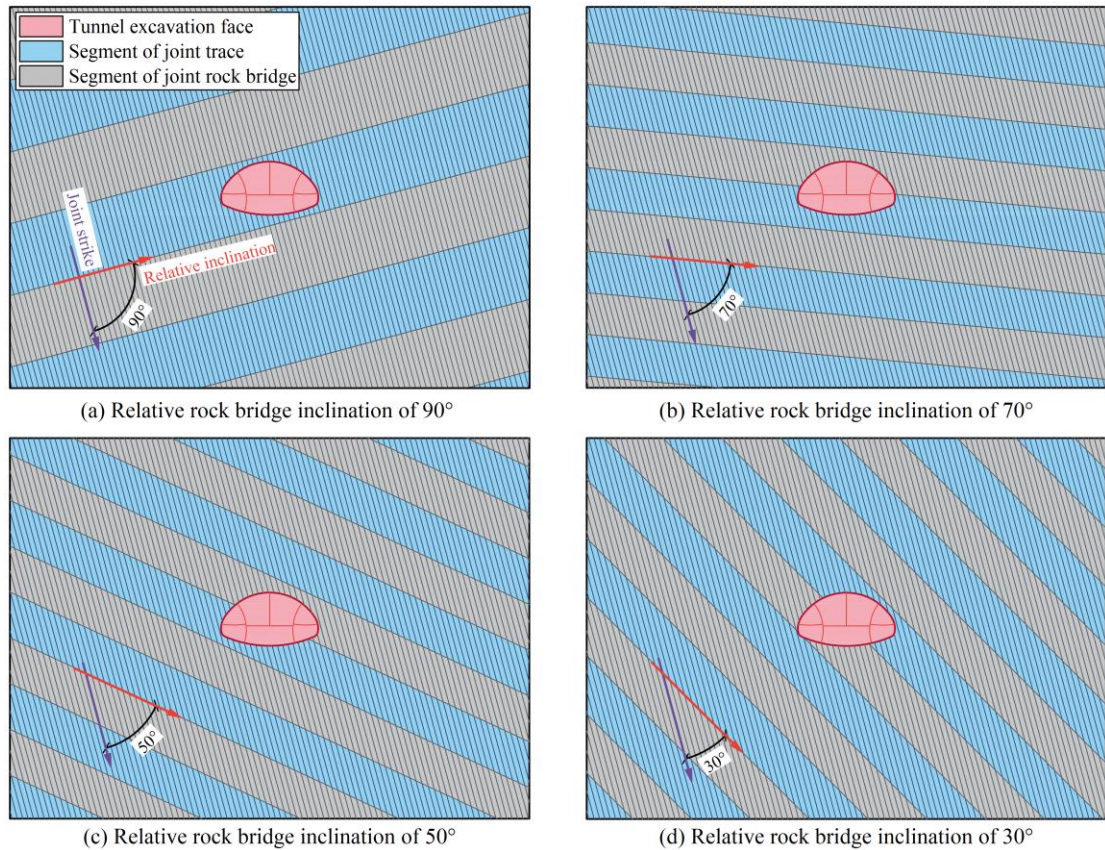


Figure 13. Schematic of tunnel models with varying relative rock bridge dip angles.

The safety factors computed for the four joint configurations under varying relative rock bridge dip angles are plotted in Figure 14(a). It can be observed that the safety factor varies only slightly across the four joint scenarios. In all cases, the safety factor gradually decreases with increasing relative rock bridge dip angle, exhibiting an overall linear trend. To facilitate analysis and highlight the variation, the increments of safety factor at each point were plotted in Figure 14(b). The results indicate that the largest variation in safety factor increment occurs for X-oriented single joints, followed by the XY and XZ joint combinations, while the YZ joint combination exhibits the smallest variation. This suggests that, when the rock mass contains only unidirectional joints, changes in relative rock bridge dip angle result in more pronounced variations in tunnel surrounding rock stability, with stability increasing as the relative dip angle decreases.

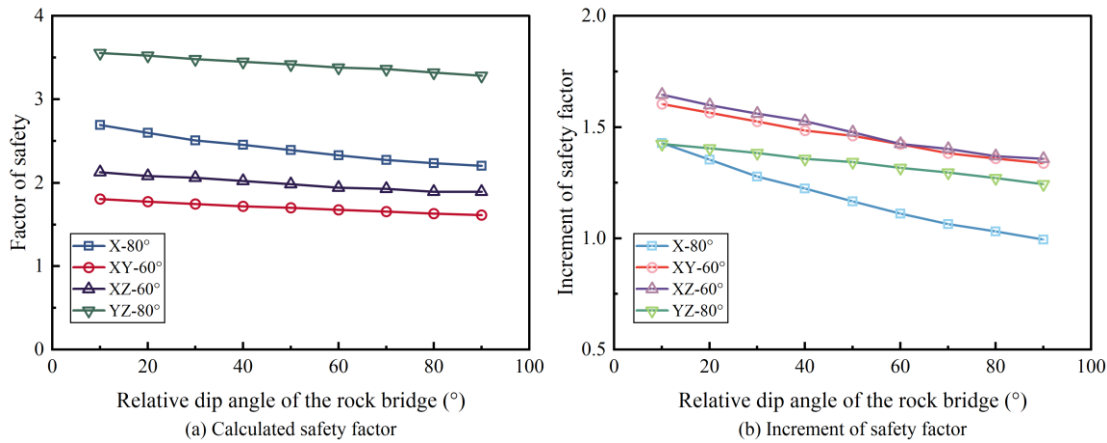


Figure 14. Variation of surrounding rock safety factor with relative rock bridge dip angle.

The underlying mechanism can be attributed to the effective increase in rock bridge length as the relative dip angle decreases. As illustrated in Figure 15, when the relative rock bridge dip angle is 90° , the rock bridge length exceeds the joint spacing, allowing the tips of adjacent joints to readily develop cracks that propagate and cause local failure. Reducing the relative dip angle increases the distance between the tips of adjacent joints, thereby lengthening the path required for crack propagation and joint connectivity. However, the effect of dip angle variation on the connectivity path is limited, resulting in only a moderate increase in the safety factor. In the case of bidirectional joint combinations, the presence of joints in two intersecting directions increases the number of adjacent joint tips at varying distances. Consequently, changes in relative rock bridge dip angle lead to only minor increases in the connectivity path, and the resulting variation in safety factor is comparatively small. Furthermore, the initial stability of the rock mass also influences the magnitude of the safety factor increment: the XY and XZ joint combinations exhibit lower tunnel safety factors than the YZ combination, making the relative impact of dip angle variation more pronounced, whereas the YZ combination shows minimal variation.

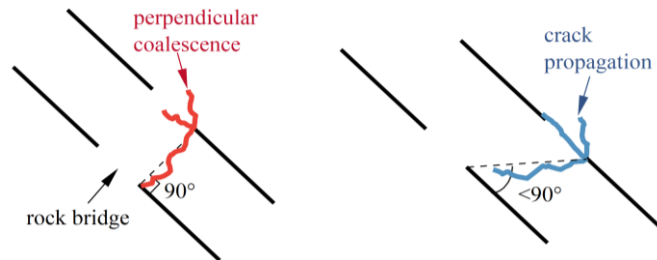


Figure 15. Joint fracture development for varying relative rock bridge dip angles.

Overall, it can be concluded that the effect of relative rock bridge dip angle is more significant for unidirectional joint distributions than for multidirectional joint combinations, and the poorer the initial stability of the rock mass, the greater the impact of relative dip angle variation; conversely, more stable configurations are less sensitive to these changes.

To quantitatively compare the sensitivities of the three joint parameters, the relative variation in the safety factor over each investigated parameter range was calculated as the difference between the maximum and minimum safety factors divided by the minimum value. The results are summarised in Table 6. The average relative variations caused by the rock bridge ratio, joint trace length, and relative rock bridge dip angle were 175.24%, 86.65%, and 13.63%, respectively. Therefore, within the investigated parameter ranges, the safety factor was most sensitive to the rock bridge ratio, followed by the joint trace length, whereas the relative rock bridge dip angle had the weakest influence. This sensitivity ranking was consistent with the overall trends obtained for the four joint configurations.

Table 6. Relative variation in safety factor for different joint parameters.

Joint parameter	X-80° (%)	XY-60° (%)	XZ-60° (%)	YZ-80° (%)	Mean (%)
Joint trace length	90.22	66.44	89.27	100.68	86.65
Rock bridge ratio	148.18	171.20	210.84	170.76	175.24
Relative rock bridge dip angle	22.18	11.75	12.29	8.32	13.63

The results obtained in this study should be interpreted within the investigated geological and numerical conditions. The quantitative peak positions, safety-

factor variations, and sensitivity rankings are affected by the adopted rock and joint properties, in-situ stress conditions, tunnel geometry, and joint configurations. Therefore, the numerical values reported herein are case-dependent. Nevertheless, the observed mechanisms, including the competition between joint continuity and rock-bridge load transfer, the transition in crack coalescence mode, and the comparatively weak influence of relative rock bridge dip angle, may provide general qualitative guidance for similar large-span tunnels in jointed rock masses.

6. CONCLUSION

(1) For discontinuous joints, the influence of joint trace length on the safety factor of large-span tunnel excavation follows a clear functional relationship, independent of joint orientation and dip. Under the investigated conditions, the peak safety-factor increment occurred when the joint trace length was approximately half of the tunnel span, reflecting a balance between rock-bridge load transfer and joint continuity. This relationship should be regarded as a scale-dependent trend under the investigated conditions rather than a universal threshold.

(2) Within the investigated joint spacing and geometric ranges, two joint-coalescence modes were identified. When the rock bridge length was smaller than the joint spacing, parallel coalescence predominated, resulting in limited changes in tunnel stability. When the rock bridge length exceeded the joint spacing, perpendicular coalescence became more pronounced, leading to greater variations in tunnel stability. These findings are applicable to the joint configurations analysed in this study.

(3) Within the investigated parameter ranges, the relative rock bridge dip angle exerted a comparatively minor influence on tunnel stability, and the safety factor generally decreased slightly as the relative dip angle increased. Joint configurations with lower inherent stability were more sensitive to changes in the relative rock bridge dip angle. The quantitative sensitivity comparison showed that the rock bridge ratio had the strongest influence on the safety factor, followed by the joint trace length, whereas the relative rock bridge dip angle had the weakest influence. This sensitivity ranking is specific to the parameter ranges and joint configurations considered in this study.

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Contributions

Jianfeng Chen: Conceptualization, Methodology, Formal analysis, Supervision, Writing-review & editing, Data curation, Investigation, Writing-original draft.

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