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Stability Analysis and Control Measures of Tunnel Face in Water-Rich Sandy Dolomite Stratum

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Abstract: Revealing the conditions for the occurrence of sand gushing accidents in tunnels with water-rich sandy dolomite strata can help the tunnels to be constructed safely. In this paper, we found that sandy dolomite and the water environment are the key factors causing sand surges through geophysical prospecting at the tunnel face where 12,000 m³ sand surge occurred. Through the flow-solid coupling model, the extrusion deformation at the tunnel face is the main deformation form in the tunnel of water-rich sandy dolomite strata. The influence of different factors on the deformation value of the tunnel face is from strong to weak: sandification degree, head height, and tunnel depth. Combined with the study of pressure arch characteristics of the model, the limit equilibrium theory considering seepage effects and pressure arch characteristics is proposed. We get that sand surges will occur in the tunnel in the intensively sandy dolomite strata where the head exceeds 80m. For the intensively sandy dolomite strata with head below 80m, the stability of the tunnel face is controlled by the degree of seepage damage. According to the reason of sand gushing, the comprehensive control measures consisting of risk identification by over-detection, double-layer close-packed pipe shed, grouting sealing technology and dewatering technology, and Milling and blasting combined construction method are proposed, and the effective control of tunnel working face is realized in practical application.

Key words: Water-rich sandy dolomite; Water and sand inrush; The pressure arch; Limit equilibrium theory; Milling and blasting combined construction method

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24 1. Introduction

25 As a sedimentary carbonate rock, dolomite is widely distributed in Yunnan-Guizhou-Sichuan mountain area of China.
26 Sandification is a weathering karst phenomenon. The dolomite rock mass without sandification is intact and has good engineering
27 properties. After sandification, dolomite is characterized by argillaceous composition, unclear bedding and loose fragmentation of
28 surrounding rock. The sandy dolomite is a mixture of sand and powder after excavation disturbance. In addition, sandy dolomite
29 becomes saturated sand state when soaked in water. Under the action of groundwater osmotic pressure, it is prone to osmotic
30 deformation, resulting in water inrush and sand inrush, which poses extremely high construction safety risks (Schneider and
31 Lavdas, 2013; Wu et al., 2017; Cui et al., 2015).

32 There have been relevant studies on the disaster mechanism of water-rich karst tunnels. Based on statistical analysis of 221
33 water-inrush and mud-inrush disaster cases, Li et al. (2018) divided the disaster-induced structures into three categories: karst,
34 fault and other causes. Four typical hazard modes of water and mud inrush in tunnels were proposed, including the directly
35 revealed type of water/mud inrush, the progressive failure type of water/mud inrush, the seepage instability type of water/mud
36 inrush and the intermittent failure type of water/mud inrush. Many scholars have carried out a lot of research on different types of
37 water and mud inrush. Through theoretical derivation and laboratory tests, the disaster-causing mechanism of karst was analyzed,
38 and the risk evaluation system and treatment measures were proposed (Wu et al., 2021; Yang et al., 2019; Xu et al., 2021).
39 Through theoretical derivation, model test and numerical simulation, the influence of water-rich fault on the stability of tunnel
40 face was investigated, and the control measures were put forward (Zhou et al., 2021; Xue et al., 2017; Zhang et al., 2019). For
41 other causes, Yuan et al. (2019) conducted laboratory tests on the causes of instability of tunnels with fully weathered granite, and
42 found that the causes of instability of tunnel face are poor stability of fully weathered granite, high groundwater level, high
43 groundwater flow rate, poor pre-reinforcement of surrounding rock and disturbance caused by tunnel construction.

44 The stability control of the tunnel face is related to the safety of tunnel construction. The limit equilibrium analysis theory is
45 one of the common methods to analyze the stability of tunnel face. Horn (1961) introduced a 3D wedge model that assumed a
46 sliding wedge loaded by a soil silo. This model was extended to layered soils by Broere (2001). Based on the limit equilibrium
47 analysis theory, Anagnostou, G (2012) used the slice method to obtain the supporting force required for the face balance.

48 Murayama et al. (1966) proposed a two-dimensional logarithmic spiral failure model based on the actual tunnel face instability
49 mode. Liu et al. (2019) proposed a ring-silo failure model considering the soil-arching effect. Anagnostou and Kovári (1994, 1996)
50 proposed a limit equilibrium analysis method considering the effect of seepage. Perazzelli et al. (2014) improved Kovári's seepage
51 model and got a more accurate analytical solution. At present, the limit equilibrium theory is relatively mature, but there is no
52 limit equilibrium theoretical model considering both seepage and earth pressure arch effects. Since the influence of the earth
53 pressure arch on the stress distribution cannot be ignored, it is necessary to pay attention to this factor..

54 The strong hazard of the sandy dolomite stratum has brought great harm to the project, so it is of great significance to
55 propose a reasonable method of stability analysis of the tunnel face. Based on the previous limit equilibrium analysis considering
56 the seepage flow, an analytical solution for the limit equilibrium analysis considering the soil arching effect and seepage effect is
57 constructed, and the analytical solution is applied to analyze the stability of the tunnel face of the water-rich sandy dolomite
58 stratum. Moreover, the impact of burial depth, water pressure and sandification level on the stability of the tunnel face is explored,
59 and corresponding disposal measures are proposed based on the results of the stability analysis of the tunnel face.

60 **2. Engineering background**

61 **2.1 Tunnel overview**

62 The Jixin tunnel of the Emei-Miyi section of the Chengdu-Kunming railway double line is located in Yuexi County, Sichuan
63 Province (Fig. 1(a)). The tunnel entrance mileage is D2K298+490, and the exit mileage is D2K316+117, with a total length of
64 17,607.335m. The Jixin Tunnel passes through the water-rich sandy dolomite formation. According to the results of geological
65 survey, the dolomite section is 10280m long, accounting for 58.3% of the total length, of which the sandy section is 1620m long,
66 accounting for 16.2% of the dolomite section. The surrounding rock of the water-rich sandy dolomite formation is broken and has
67 poor self-stability. During the construction of the Jixin Tunnel in the sandy dolomite stratum, disasters such as water inrush and
68 sand inrush and landslides are often encountered (Fig. 1(b)).

Fig. 1 Sandy dolomite stratum tunnel; (a) Location of Jixin Tunnel and Distribution of Dolomite Section; (b) Photos of sandy dolomite geological hazards in Jixin Tunnel

2.2 Geological survey of the catastrophic section

In this paper, we take the sand gushing accident of water-rich sandified dolomite stratum at mileage D4K307+375 in Jixin Tunnel as the research object, and the volume of gushing sand reaches 12,000 square meters(Fig. 2). Geological radar and transient electromagnetic are used to investigate the surrounding rock condition and water-rich situation of the tunnel. Geological radar transmits high-frequency electromagnetic waves to the detected body through the transmitting antenna, and the reflected electromagnetic waves are received by the receiving antenna of geological radar, thus forming a radar image. Large amount of distorted signals in radar images of fractured rock masses. Transient electromagnetism works by generating a primary magnetic field propagating in the direction normal to the receipt loop line from the sending loop line. Under the excitation of the primary magnetic field, the geological body will generate eddy currents whose magnitude depends on the degree of electrical conductivity of the geological body. For the water-rich fractured rock, the detection results will show low resistance characteristics, and for the water-free cavity, it will show high resistance characteristics.

Fig. 2 Sand surge at mileage D4K307+375 in Jixin Tunnel

Fig. 3 is the surface transient electromagnetic detection. The area with the apparent resistivity value less than $500\Omega\cdot\text{m}$ is judged as the physical exploration V anomaly area, which corresponds to the weak rock body with extreme fragmentation and water; the area with the apparent resistivity value more than $550\Omega\cdot\text{m}$ is judged as the physical exploration IV anomaly area, which corresponds to the weak fragmentation rock body. It can be seen that the stratum that the tunnel traverses is a Type V anomaly zone, indicating that the stratum is extremely broken and water-rich.

Fig. 3 Transient electromagnetic detection results on the surface

Fig. 4 shows the transient electromagnetic results in front of the tunnel face. There is a cavity formed in the arch of the tunnel face due to sand surge (high resistance area in Fig. 4), and the front of the tunnel face is water-rich broken sandy dolomite in the detection range (low resistance area in Fig. 4).

Fig. 4 Transient electromagnetic detection results at the tunnel face

Geological radar was used to detect the stratigraphy around the tunnel (Fig. 5). The sandy dolomite stratum was severely weathered and broken, and the distortion range of geo-radar detection was large.

Fig. 5 Geological radar detection results around the tunnel

The detection results reveal that the sand surging is due to the loose and broken characteristics of the sandy dolomite stratum and the joint action of the water environment. Through the statistics of the geological situation revealed on the tunnel face, the sandy dolomite stratum can be divided into three types of sandification degrees: lightly sandy, moderately sandy, and intensively sandy dolomites (Fig. 6). The different degrees of sandification and the water environment make it difficult to predict the stability of the tunnel surface. Later in this paper, the analysis focuses on the catastrophic occurrence conditions of sanded dolomite.

Fig. 6 Sandy dolomite three types of sandification degree

3. Tunnel Deformation and Pressure Arch Characteristics in Water-Rich Sandy Dolomite Stratum

3.1 Establishment of fluid-solid coupling model

The calculation model is based on the tunnel design drawings. The width and height of the tunnel are 12.42 m and 11.25 m. Based on the range of stress affected by tunnel excavation, the size of the model is designed to be 180m×160m×100m, so as to reduce the boundary effect of the model. The model is shown in Fig. 7. Horizontal constraints are applied to the left and right sides of the model, vertical constraints are applied to the bottom surface, and normal constraints are applied to the front and back surfaces. Only the self-weight stress field is considered. The model adopts a three-step construction method with a step distance of 10m. The initial support is composed of steel arches, grouting anchor rods, and C25 shotcrete.

Fig. 7 Schematic diagram of finite element model

3.2 Calculation parameter selection

According to laboratory tests and literature on the mechanical properties of sandy dolomite (Zhang, 2012), the calculation parameters of lightly sandy, moderately sandy, and intensively sandy dolomites are calibrated. This calculation assumes that all

materials are continuous and uniform ideal substances. The Mohr-coulomb Model is used as the constitutive relationship of the surrounding rock. And the calculated parameters of the stratum are shown in Table 1. According to the experimental research of related scholars (Chen, 2009), the elastic modulus of the surrounding rock is improved by more than 30%, the cohesion and friction angle are increased by 20%~30%, and the permeability coefficient is reduced to 0.1%~0.5% of the original one after the reinforcement of the surrounding rock by grouting. The water pressure value at the tunnel surface is adjusted by setting different water level heights of the model. Adjustment of burial depth and initial ground stress by applying stress to the upper surface of the model.

Table 1 Calculation parameters

Material	Elastic modulus (GPa)	Poisson's ratio	Friction angle	Cohesion (kPa)	Density (kg/m³)	Permeability	
Lightly sandy dolomite	10.2	0.28	29.7	420	2320	2.00E-07	
Moderately sandy dolomite	7.0	0.29	25	280	2020	3.60E-07	
Intensively sandy dolomite	3.6	0.3	20	115	1680	2.00E-06	
Grouting reinforcement area	Lightly sandy dolomite	13.26	0.28	38.61	546	2800	1.00E-09
	Moderately sandy dolomite	9.1	0.29	32.5	364	2800	1.80E-09
	Intensively sandy dolomite	4.68	0.3	26	150	2800	1.00E-08

The steel frame is equivalently replaced by beam elements with equal section stiffness. The initial support uses linear elastic elements to simulate C25 shotcrete, and the support structure parameters are shown in Table 2.

Table 2 Support calculation parameters

Material	Elastic modulus (GPa)	Poisson's ratio	Density (kg/m ³)
C25 shotcrete	27.57	0.2	2500
Steel frame	210	0.3	7350

3.3 Deformation characteristics of tunnel face in sandy dolomite stratum under seepage action

As show in Fig. 8 and Fig. 9, the sandification degree of the dolomite strata, the depth of tunnel burial and the water level have a negative impact on the extrusion deformation of the tunnel tunnel face and the settlement of the vault. Among them, the sandification degree of the dolomite stratum has a dominant position, and the increase of sandification degree will significantly increase the sensitivity of tunnel deformation to burial depth and water level. In addition the effect of water level variation is greater than the effect of burial depth variation. Comparing the extrusion deformation values with the vault settlement values (Fig. 10), the deformation of the sandy dolomite tunnel is dominated by the extrusion deformation of the tunnel face. This corresponds

well to the accident of sand gushing at the tunnel face that is prone to occur when building tunnels in sandy dolomite strata.

Fig. 8 The influence of degree of sandification, water level and buried depth on the extrusion deformation of the face

Fig.9 The influence of degree of sandification, water level and buried depth on the settlement deformation of vault

Fig.10 Comparison of extrusion deformation and vertical deformation

3.4 Pressure arch characteristics of tunnel face in sandy dolomite stratum under seepage action

3.4.1 Pressure arch criterion

The most notable feature of the pressure arch is that the streamline of the maximum principal stress vector can form a complete ring. In the pressure arch, the direction of the maximum principal stress in the arch is the horizontal direction, and the maximum principal stress outside the arch will be restored to the vertical direction. Therefore, the boundary of the pressure arch is determined according to the changes in the horizontal and vertical stresses of the element. The position where the maximum principal stress direction is the horizontal direction is defined as the inner boundary of the pressure arch, while the position where the maximum principal stress direction is deflected from horizontal to vertical is defined as the outer boundary (Zhang et al., 2018).

3.4.2 Analysis of pressure arch characteristics

Using 300m burial depth and 100m water level with intensively sandy dolomite as the base condition, the effect of burial depth, water level and sandification degree on the form of pressure arch after tunnel excavation was investigated under the effect of seepage. The cross section at the tunnel face was selected as the study object. As we all know, before the tunnel strata are disturbed by excavation, the maximum principal stresses around the tunnel are in the vertical direction. After the tunnel excavation, the strata were in force imbalance, and in order to reach a new equilibrium, the direction of the principal stress in the rock around the tunnel was significantly deflected, and the direction of the streamline distribution of the maximum principal stress showed a ring shape (Fig.11). As shown in Fig. 11, at a certain height directly above the top of the tunnel, the direction of the maximum principal stress is deflected to the horizontal direction (rotated by nearly 90°). Moreover, the stress deflection point appears at the top of the shaded part, forming a stress deflection zone. The outer contour of the shaded part is the outer boundary of the pressure arch.

Fig. 11 Principal stress distribution tensor diagram of dolomite stratum

Fig. 12 reflects the influence of different factors on the morphology of the pressure arch in the tunnel surface area after tunnel excavation. As shown in Fig. 12(a)~(b), the deeper the tunnel is, the more the surrounding rock needs to be involved in stress rebalancing after excavation, resulting in a greater thickness of the pressure arch. As shown in Fig. 12(a), (b) and (e), the presence of water reduces the effective stress of the surrounding rock on the one hand, and the presence of osmosis pressure reduces the stability of the tunnel face on the other. Therefore, the thickness of the pressure arch increases significantly as the water level rises. As shown in Fig. 12(a), (f) and (g), the degree of sanding has the greatest influence on the morphology of the pressure arch. The greater the degree of sanding, the weaker the self-stability of the formation and the greater the thickness of the pressure arch. For lightly sanded dolomite strata with good bearing capacity, shear damage did not occur in the surrounding rock at the tunnel face after tunnel excavation, so no pressure arch was formed (Fig. 12 (g)).

(e) Buried 200m-water level 40m-intensively sandy dolomite (f) Buried 300m-water level 100m-moderately sandy dolomite (g) Buried 300m-water level 100m-lightly sandy dolomite

Fig. 12 Stress distribution curve after tunnel excavation

The position of the outer boundary of the pressure arch under different working conditions is shown in the Fig. 13.

Fig. 13 Pressure arch height under different working conditions

4. Limit equilibrium analysis of the tunnel face of a deep-buried sandy dolomite tunnel

4.1 The shape of the instability zone of the tunnel face

Taking the intensively sandy dolomite stratum with a buried depth of 300m and a water level of 100m as an example, the shear strain contour of the tunnel face is extracted as shown in Fig. 14. It can be seen that after the excavation of the tunnel, a wedge-like shear strain connection zone will appear on the face of the upper step, and the rock mass in this area will slip along the shear slip layer under the action of the overlying load, which will lead to instability.

Fig. 14 Range of instability zone

Fig. 15 Limit equilibrium analysis model

A limit equilibrium analysis model(Fig. 15) was constructed to modify the limit equilibrium model for wedge prisms considering seepage proposed by Perazzelli et al (2014) by considering the effect of stress deflection in the pressure arch region on the overlying load of the wedge. The prism area is considered as the pressure arch influence area, the prism length is taken as the outer boundary of the pressure arch, and the load on the overlying rock of the prism is P_0 . In the model, the height H of the wedge is determined according to the shape of the excavation surface, which is the maximum distance between the top and the bottom of the arch. Suppose the area of the excavation surface is A , and then according to the principle of equal area of the excavation surface, the width B of the wedge can be determined. As shown in Fig. 15, w is the rupture angle of the wedge, t is the height of the prism, and h_0 is the height of the model head.

The basic assumptions of this model calculation are as follows: (1) The rock mass in the wedge-prism region conforms to the Terzaghi theory. (2) The stratum is evenly distributed and isotropic. (3) The vertical stress on the failure surface is linearly distributed. (4) Only the self-weight load is considered, and the influence of tectonic stress is ignored.

4.2 Calculation of face stability based on limit equilibrium theory

4.2.1 Calculation of earth pressure on the wedge.

The load on the wedge comes from the prism. Therefore, it is necessary to analyze the stress distribution of the prism first. As shown in Fig. 16, using the slice method, the vertical earth pressure in the prism can be calculated according to Terzaghi theory. Since the stress direction in the pressure arch area is deflected (Fig. 11), the influence of the pressure arch effect is generally reflected by the correction of the prism side pressure coefficient in the calculation. The value can be obtained by referring to the literature (Handy,1998):

$$K_h = \frac{\sigma_h}{\sigma_v} = \frac{\cos^2 \theta + K_a \sin^2 \theta}{\sin^2 \theta + K_a \cos^2 \theta} \quad (1)$$

where σ_h is the horizontal stress; σ_v is the vertical stress; $\theta = 45^\circ + \phi/2$ and $K_a = \tan(45^\circ - \phi/2)$, ϕ is the friction angle of the rock mass.

As shown in Fig. 16, the load P_0 is overlaid on the prism. The slice thickness is taken as dz , the upper and lower surfaces are respectively subjected to effective stresses $\sigma'_z + d\sigma'_z$ and σ'_z . The sides are subjected to the shearing force of the surrounding rock mass, and the rock mass is subjected to seepage. The vertical force balance formula is given:

Fig. 16 Schematic diagram of calculation model for prism earth pressure

$$\frac{d\sigma'_z}{dz} - \frac{K_h \sigma'_z \tan \phi}{R} = -\gamma' - \gamma_w i_{av} + \frac{c_1}{R} \quad (2)$$

where the coefficient R is the ratio of the area to the circumference of a horizontal cross-section of the prism; γ' is the buoyant unit weight of rock mass; γ_w is the weight of water; c_1 is the cohesion of the rock mass in the prism; ϕ is the friction angle of the rock mass; i_{av} is the average vertical hydraulic gradient in the prism at height z . The calculation formula of i_{av} is shown in Eq. (3).

$$i_{av}(z) = \frac{1}{BH \tan w} \int_0^{H \tan w} \int_{-B/2}^{B/2} \frac{\partial h(x, y, z)}{\partial z} dx dy \quad (3)$$

where w is taken as 32° according to the shape of the instability zone in Fig. 14; $h(x, y, z)$ is the hydraulic head distribution formula, and the approximate expressions of the water head in the prism and wedge are shown in Eq. (4):

$$\left. \begin{aligned} h(x, y, z) &= h_t + \left\{ 1 - \exp\left[a\left(1 - \frac{z}{H}\right) - b\frac{x}{H}\right] \right\} \Delta h \\ h(x, y, z) &= h_t + \left\{ 1 - \exp\left[-b\frac{x}{H}\right] \right\} \Delta h \end{aligned} \right\} \quad (4)$$

where h_t is the water head at the tunnel face; Δh is the model water head; a and b are parameters obtained by fitting the water head distribution of the model, as shown in Table 3.

Table 3 Values of coefficients a and b for different water levels

Water levels (m)	a	b
100	0.676	1.01
70	0.596	1.01
40	0.534	1.01
10	0.49	1.01

By substituting Eq. (4) into Eq. (3), the following is obtained.

$$i_{av}(z) = \frac{\Delta h}{H} \frac{a(1 - e^{-b \tan w})}{b \tan w} e^{a(1 - \frac{z}{H})} \quad (5)$$

Combined with the boundary conditions $\sigma'_z(H + t) = P_0$, the vertical stress distribution formula in the prism is obtained by

solving the differential Eq. (2).

$$\sigma'_z = \frac{\gamma'R - c_1}{K_h \tan \phi} \left(1 - e^{-\frac{K_h \tan \phi}{R}(z-H)}\right) + P_0 e^{-\frac{K_h \tan \phi}{R}(z-H)} + \gamma_w \Delta h \bar{\alpha} \quad (6)$$

where $\bar{\alpha}$ is the effective vertical load increase factor caused by seepage, which can be calculated by Eq. (7).

$$\bar{\alpha} = \frac{a(1 - e^{-b \tan w})}{b \tan w} \frac{R}{H\lambda \tan \phi + Ra} \left(1 - e^{-\frac{(z-H)(H\lambda \tan \phi + Ra)}{HR}}\right) \quad (7)$$

Combining Eq. (6) and Eq. (7), the load on the wedge can be obtained:

$$V_{silo} = BH \tan w \cdot \min \left\{ 0, \frac{\gamma'R - c_1}{K_h \tan \phi} \left(1 - e^{-\frac{K_h \tan \phi}{R}t}\right) + P_0 e^{-\frac{K_h \tan \phi}{R}t} + \gamma_w \Delta h \bar{\alpha} \right\} \quad (8)$$

where

$$\bar{\alpha} = \frac{a(1 - e^{-b \tan w})}{b \tan w} \frac{R}{H\lambda \tan \phi + Ra} \left(1 - e^{-\frac{t(H\lambda \tan \phi + Ra)}{HR}}\right) \quad (9)$$

4.2.2 Stress analysis of the wedge

The wedge is sliced for analysis. As shown in Fig. 17, the following forces are applied to the slices: its submerged weight dG , the effective stress V acting on the bottom surface of the slice, the effective stress $V + dV$ acting on the top surface of the slice, the effective normal stress dN of the inclined sliding surface, the effective tangential stress dT of the inclined sliding surface, the effective tangential stress dT_s of the vertical sliding surfaces on both sides, seepage forces dF_x and dF_z , and the virtual support dS at the tunnel face. The balance equations parallel and perpendicular to the sliding direction are expressed:

Fig. 17 Forces acting upon wedge slice

$$\begin{cases} dT_s + dT + (dS - dF_x) \sin w = (dV + dG' + dF_z) \cos w \\ dN = (dV + dG' + dF_z) \sin w + (dS - dF_x) \cos w \end{cases} \quad (10)$$

The parameters in the Eq. (10) are shown in Appendix A. The differential equation (11) can be obtained by Eq. (10), and the expressions of each parameter in Eq. (11) are shown in Appendix A:

$$B \frac{dV}{dz} - \Lambda V = M \frac{z}{B} + P - B \frac{dF_z}{dz} - B P_s \frac{dF_x}{dz} \quad (11)$$

Solve the differential equation (11) with $V(0) = 0$:

$$V(z) = C_s(\xi) B^2 S + C_c(\xi) B^2 c_2 - C_\gamma(\xi) B^3 \gamma' - C_{\Delta h}(\xi) B^2 \gamma_w \Delta h \quad (12)$$

where $\xi = \frac{z}{H}$, and the parameters $C_s(\xi)$, $C_c(\xi)$, $C_\gamma(\xi)$, $C_{\Delta h}(\xi)$ are shown in Appendix A.

The vertical force at the wedge-prism interface reads as follows:

$$V(H) = C_s(1) B^2 S + C_c(1) B^2 c_2 - C_\gamma(1) B^3 \gamma' - C_{\Delta h}(1) B^2 \gamma_w \Delta h \quad (13)$$

4.4.3 Effective virtual support on the face

In the limit equilibrium state, the load applied by the prism is equal to the load on the top surface of the wedge as shown in

Eq. (14):

$$V_{silo} = V(H) \quad (14)$$

Combined Eq. (8), Eq. (13) and Eq. (14), the following is obtained.

When $V_{silo} \neq 0$,

$$S = F_1 \gamma_w \Delta h + F_2 \gamma' H - F_3 c_1 - F_4 c_2 + F_5 P_0 \quad (15)$$

where

$$F_1 = \frac{1}{C_s(1)} \left(C_{\Delta h}(1) + \bar{\alpha} \frac{H \tan w}{B} \right) \quad (16)$$

$$F_2 = \frac{1}{C_s(1)} \left(\frac{C_\gamma(1) B}{H} + \frac{R \tan w \left(1 - e^{-K_h \tan \phi \frac{t}{R}} \right)}{B K_h \tan \phi} \right) \quad (17)$$

$$F_3 = \frac{H \tan w \left(1 - e^{-K_h \tan \phi \frac{t}{R}} \right)}{C_s(1) B K_h \tan \phi} \quad (18)$$

$$F_4 = \frac{C_c(1)}{C_s(1)} \quad (19)$$

$$F_5 = \frac{H \tan w}{C_s(1) B} e^{-K_h \tan \phi \frac{t}{R}} \quad (20)$$

When $V_{silo} = 0$,

$$S = F_1\gamma_w\Delta h + F_2\gamma'H - F_3c_2 \quad (21)$$

where

$$F_1 = \frac{C_{\Delta h}(1)}{C_s(1)} \quad (22)$$

$$F_2 = \frac{C_\gamma(1)B}{C_s(1)H} \quad (23)$$

$$F_3 = \frac{C_c(1)}{C_s(1)} \quad (24)$$

When the virtual supporting force of the tunnel face $S > 0$, it indicates that the wedge in front of the tunnel face cannot be stable, and the supporting force is needed to maintain stability. When $S \leq 0$, it means that the wedge can maintain the stability without supporting force.

4.4.4 Stability analysis of tunnel face

Calculated by Eq. (15) and Eq. (21), the virtual supporting force of the tunnel face under different working conditions is shown in Fig. 18. It can be seen that the virtual supporting force of the tunnel face in the water-rich sandy dolomite stratum has a strong correlation with the water level. Due to the better bearing capacity of the moderately and lightly sandy dolomite stratum, the tunnel face can maintain its own stability. However, for intensively sandy dolomite stratum, when the water level reaches 80m or more, the tunnel face will lose stability, which is consistent with the phenomenon of water and sand inrush induced by excavation in high-head area. However, according to the site construction situation in low-head area, the tunnel face will also lose stability over time.

Fig. 18 Virtual supporting force of the tunnel face under different working conditions

The phenomenon that the fine particles in the loose rock body will be transported and lost under the action of seepage, thus

294 leading to changes in the pore structure of the filling medium, cannot be ignored (Fig. 19). This seepage damage can lead to an
 295 increase in water surges and eventually induce sudden water and mud disasters (Ma et al., 2017; Zhang et al., 2016). In order
 296 to explore the degree of weakening of the loss of fine particles on the bearing capacity of surrounding rock (Fig. 20), triaxial
 297 compression tests were carried out on intensively sandy dolomite with different proportions of fine particles. The test shows that
 298 with the decrease of fine particle proportion, the porosity of rock samples increases, the bonding effect between particles decreases,
 299 and the cohesion of rock samples decreases obviously. Since the seepage loss occurs in the area of the tunnel face, it is considered
 300 that the parameters of the wedge are attenuated, while the parameters of the prism area are not attenuated. The attenuated rock
 301 sample parameters are brought into Eq. (15) and Eq.(21) for calculation, and the virtual supporting force of the working face is
 302 obtained, as shown in Fig. 21.

303
304 **Fig. 19 Schematic diagram of seepage damage**

305 **Fig. 20 Cohesion of sandy dolomite with different proportions of fine particles**
 306 **Fig. 21 The virtual supporting force of the tunnel face**
 307 **under different proportions of fine particles**

308 Due to seepage damage, the self-stabilization ability of the intensively sandy dolomite stratum has decreased significantly.

Corresponding to the construction site, as the exposure time of the tunnel face increases, the undisturbed tunnel face will also undergo catastrophe under the action of seepage. The failure modes of the water-rich and sandy dolomite stratum can be divided into two types: (1) when water head > 80m, the tunnel face will burst under the action of water (Fig. 22(a)); (2) when water head < 80m, the tunnel will experience seepage loss first, then the load-bearing capacity of the tunnel face decreased, followed by a surge disaster (Fig. 22(b)).

Fig. 22 The failure modes of the water-rich and sandy dolomite stratum

5. Construction Control Measures of Sandy Dolomite Tunnel

Combined with the stability analysis of the tunnel face in the sandy dolomite tunnel, in order to prevent disasters, it is necessary to timely reveal the sandy dolomite stratum, take reliable advance reinforcement measures, and take effective control measures against water seepage. As shown in Fig. 23, a comprehensive control technology for tunnel construction in water-rich and intensively sandy dolomite stratum is constructed based on the identification technology of high-risk areas, advanced control measures in high-risk areas and low-disturbance construction methods.

(a) High risk areas identification

(b) High risk areas identification

(c) High-risk area weak disturbance construction method

Fig. 23 Comprehensive control technology for tunnel construction in intensively sandy dolomite stratum with rich water

As shown in Fig. 23(a), the geological radar and transient electromagnetic technology were used to discover the broken surrounding rock and water-rich section in time before construction, and the geological radar signal distortion section and transient electromagnetic low-resistance section were initially determined as high-risk sections. For high-risk sections, pre-construction geological drilling was carried out to further explore the degree of sandification and water content of the surrounding rock. If the drilled rock sample is complete, it is lightly sandy dolomite, if the rock sample is blocky with fissures developed, it is moderately sandy dolomite, if the rock sample is gravelly, it is intensively sandy dolomite. For the area where water emerged from the borehole and the drilled rock sample is strongly sanded dolomite can be finalized as high risk area.

For high-risk areas, grouting sealing technology and dewatering technology are used to release water and pressure, and double-layer close-packed pipe shed (as shown in Fig. 23(b),) are used to control the stability of the tunnel face. Among them, grouting sealing technology can control seepage damage and water intruding at the tunnel face, dewatering technology can effectively reduce the water pressure at the tunnel face, and the double-layer close-packed pipe shed can effectively share the load of the overlying rock mass of the wedge and improve the bearing capacity of the tunnel.

The borehole-blasting method is easy to cause deterioration of the surrounding rock around the excavation area and increase

the seepage channel (Chen et al., 2016). For the water-rich and intensively sandy dolomite stratum with extremely low self-stability, ordinary borehole-blasting method construction is easy to induce catastrophe. Therefore, combination method of milling and blasting (Fig. 23(c)) is proposed to replace the original three-step method, thereby reducing temporary measures, controlling over-under-excavation, reducing surrounding rock disturbance, and preventing surrounding rock from peeling and collapsing. The key to the combination method of milling and blasting is that the reserved thickness in the excavation contour of the upper and middle steps of the tunnel is not less than 1m. The reserved thickness is used to reduce the impact of blasting, and this part is chiseled off with a milling machine. In addition, weak blasting and short footage (1.2m) is adopted to reduce the disturbance of surrounding rock.

6. Conclusion

Based on the Jixin Tunnel project of the Chengdu-Kunming railway double track, this paper analyzes the stability of the tunnel face in the water-rich sandy dolomite stratum, and proposes corresponding construction control measures. The main conclusions are as follows:

(1) Through the physical exploration of the tunnel surface at the tunnel face where 12,000 m³ sand surge occurred, it reveals that the two necessary conditions for the sand surge of the dolomite formation are the sandy dolomite and the water environment. Because of the poor mechanical properties of sandy dolomite, If there is no control measures in the water-rich conditions, it is very easy to gush sand, which will form a huge collapse cavity, for the later construction of the formation of engineering hidden trouble at the same time to improve the management of sand gushing difficulty.

(2) Under the combined action of seepage force and ground stress, the deformation of the tunnel face in the water-rich sandy dolomite stratum is dominated by extrusion deformation. The influence of different factors on the deformation value of the tunnel face is from strong to weak: sandification degree, head height, and tunnel depth. Excavation of tunnel in sandy dolomite stratum results in significant pressure arches, and the order of the influence of each factor on the thickness of pressure arches is: sandification degree, head height, and tunnel depth.

(3) Combined with the force characteristics of the pressure arch, an analytical solution to the limit equilibrium of the tunnel face considering the seepage flow is constructed. The analysis shows that for tunnels with intensively sandy stratum with a water

head higher than 80m, the tunnel face will undergo catastrophe. For tunnels in intensively sandy stratum with a water head lower than 80m, the stability of the tunnel face is related to the reveal time of the tunnel face. With the occurrence of seepage, the bearing capacity of the tunnel face area decreases, leading to catastrophe. Therefore, there is a catastrophic risk in the water-rich, deep-buried and intensively sandy stratum.

(4) For the water-rich and intensively sandy dolomite stratum, risk control must first start with risk identification, and the risk identification in front of the tunnel face is completed by means of advanced forecasting. Based on the stability analysis of the tunnel face, grouting sealing technology and dewatering technology, as well as the construction of double-layer close-packed pipe shed, can effectively realize the control of the slip section of the tunnel face. The construction method combining milling and blasting is put forward innovatively to reduce the disturbance of surrounding rock and prevent the surrounding rock from peeling and collapsing. These methods have been proved to be effective in realizing stable excavation of the tunnel face in actual engineering applications.

Appendix A. Coefficients

The following expressions have been derived by Anagnostou (2012) and Perazzelli(2014) . The forces appearing in Eqs. (10) read as follows:

$$dG = \gamma B dA \quad (A1)$$

$$dA = z \tan w dz \quad (A2)$$

$$dS = S B dz \quad (A3)$$

$$dT = \frac{B dz}{\cos w} c_2 + dN \tan \phi \quad (A4)$$

$$dT_s = 2dA(c_2 + \lambda \sigma_z \tan \phi) \quad (A5)$$

$$\sigma_z = \frac{V}{B z \tan w} \quad (A6)$$

$$dF_x = \left(\int_0^{z \tan w} \int_{-B/2}^{B/2} \gamma_w \frac{\partial h(x, y, z)}{\partial x} dx dy \right) dz \quad (A7)$$

$$dF_z = \left(\int_0^{z \tan w} \int_{-B/2}^{B/2} \gamma_w \frac{\partial h(x, y, z)}{\partial z} dx dy \right) dz \quad (A8)$$

The coefficients appearing in Eq. (11) are:

$$\Lambda = \frac{2\lambda \tan \phi}{\cos w - \sin w \tan \phi} \quad (\text{A9})$$

$$M = \frac{\Lambda \tan w}{\lambda \tan \phi} B^2 c_2 - B^3 \gamma' \tan w \quad (\text{A10})$$

$$P = \frac{\Lambda B^2 c_2}{2\lambda \tan \phi \cos w} + B^2 S \tan(\phi + w) \quad (\text{A11})$$

$$P_s = \tan(\phi + w) \quad (\text{A12})$$

The coefficients $C_s(\xi)$, $C_c(\xi)$, $C_\gamma(\xi)$, $C_{\Delta h}(\xi)$ appearing in Eq. (15) read as follows:

$$C_s(\xi) = \frac{C_v(\xi) - 1}{\Lambda} P_s \quad (\text{A13})$$

$$C_c(\xi) = \frac{C_v(\xi) - 1}{2\lambda \tan \phi \cos w} + \frac{F(\xi) \tan w}{\Lambda \lambda \tan \phi} \quad (\text{A14})$$

$$C_\gamma(\xi) = \frac{F(\xi)}{\Lambda^2} \tan w \quad (\text{A15})$$

$$C_{\Delta h}(\xi) = P_s \left(\left(\frac{1}{\Lambda} - \frac{1}{\Lambda \frac{H}{B} + b \tan w} \frac{H}{B} \right) e^{\frac{\Lambda H}{B} \xi} + \frac{1}{\Lambda \frac{H}{B} + b \tan w} \frac{H}{B} e^{-b \xi \tan w} - \frac{1}{\Lambda} \right) \quad (\text{A16})$$

$$F(\xi) = C_v(\xi) - 1 - \frac{\Lambda H}{B} \xi \quad (\text{A17})$$

$$C_v(\xi) = e^{(\Lambda H / B) \xi} \quad (\text{A18})$$

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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