Stability analysis of rock slope and calculation of rock lateral pressure in foundation pit with structural plane and cave development

Jin Xu (✉ 137112492@qq.com)

China University of Mining and Technology, Xu Zhou

Yansen Wang

China University of Mining and Technology, Xu Zhou

Research Article

Keywords: Karst cave development, rock foundation pit, upper bound analysis, rock lateral pressure

Posted Date: December 9th, 2021

DOI: <https://doi.org/10.21203/rs.3.rs-1150045/v1>

License: This work is licensed under a Creative Commons Attribution 4.0 International License.

[Read Full License](#)

Stability analysis of rock slope and calculation of rock lateral pressure in foundation pit with structural plane and cave development

Jin Xu¹ Yansen Wang¹

(1. State key Laboratory for Geomechanics and Deep Underground Engineering ,
China University of Mining and Technology, Xu Zhou, 221116, China)

Abstract: In this study, numerical simulations were carried out to analyze the influence of caves in different positions and shapes, in combination with structural planes, on the stability of the slope and the failure characteristics of a rock slope in a deep foundation pit with high inclination structural planes and cave development. The schemes for substituting a single karst cave for karst caves were constructed. Based on the penetration failure characteristics of karst caves between parallel structural planes, methods for calculating the safety factor of the rock foundation pit and the upper bound of the lateral pressure of the supporting structure under the combined influence of the caves and structural planes were developed, which can be used to assess the safety factor of a rock mass and to calculate the lateral pressure under complex geological conditions.

Keywords: Karst cave development, rock foundation pit, upper bound analysis, rock lateral pressure

Corresponding author at: State Key Laboratory for Geomechanics and Deep underground Engineering, China University of Mining and Technology, Xuzhou, Jiangsu 221116, China.

E-mail address: 137112492@qq.com (Jin Xu).

1 Introduction

A variety of methods have been developed to study the stability of soil slopes in foundation pits. However, for rock slopes, the current consensus is that the failure of rock slopes is related to the structure of the rock mass or the structural plane [1], but the failure mechanism has not been fully identified.

Current theoretical methods for analyzing slope stability in foundation pits mainly include the limit equilibrium

method [2-6], limit analysis method [7-11], numerical analysis [12-15], fuzzy analysis [16-18], and reliability analysis [19-21]. There are few studies of the stability of rock slopes in areas where karst caves are developed. This is primarily due to the difficulty of the transformation between a numerical model of karst caves and theoretical analysis.

The existing numerical models used to analyze the influence of karst caves on the stability of foundation pits are

on the failure mode of the foundation pit, the diameter-to- distance ratio (DDR) was introduced, which is defined as the ratio of the diameter of a circular cave (or the major axis of an elliptical cave) to the true distance between the structural planes. The DDR ranged from 0.9 to 0.2.

2.1 Influence of the Inclination Angle of the Structural Plane on the Failure Mode of the Rock Slope

Structural planes with varying inclination angles (i.e., 60°, 70°, and 80°) were simulated, and Figure 2-2 presents the displacement map of the first calculation point under each inclination angle.

Taking a DDR of 0.9 as an example, the cave penetration failure occurred from point 1 at the top of the foundation pit under all three conditions. As the inclination angle of the structural plane increased, the angle between the crack and the lower structural plane also gradually increased, that is, the failure mode changed from shear failure to

tensile failure.

Figure 2-3 shows the end point of the penetration failure under different inclination angles. For an inclination angle of 60°, the penetration failure extended to point 3; and for an inclination angle of 70°, it extended to point 2. In comparison, the penetration failure only reached point 1 when the inclination angle was 80°.

When the DDR was decreased 0.8 (Figure 2-4) , the starting point of the penetration failure did not change, yet the end point of the penetration failure changed from point 3 (DDR=0.9) to the fourth point (DDR=0.8) for an inclination angle of 60°. However, for inclination angles of 70° and 80°, the end point did not change. The failure of the rock bridge changed from shear failure to tensile failure. Thus, it can be concluded that the penetration failure mode of karst caves is related to the size of the rock bridge between the karst cave and the structural plane and the inclination of the structural plane.

Table 2-1 Rock mass parameters of the calculation model

Name	Density (kg/m ³)	Young's modulus (Pa)	Poisson's ratio	Cohesion (Pa)	Internal friction angle (°)	Compressive strength (Pa)
Limestone	2300	21.0e9	0.22	3.0e6	30	1.0e6

Table 2-2 Structural plane parameters of the calculation model

Name	Normal stiffness (Pa)	Shear stiffness (Pa)	Cohesion (Pa)	Internal friction angle (°)	Compressive strength (Pa)
------	--------------------------	----------------------------	------------------	-----------------------------------	------------------------------

Structural plane IV	2.0e9	1.0e9	2.0e5	15	1.0e5
------------------------	-------	-------	-------	----	-------

inclination angle of 60°,
point 1

inclination angle of 70°,
point 1

inclination angle of 80°,
point 1

Figure 2-2 Displacement of the starting point of the penetration failure under different inclination angles of the structural plane (DDR=0.9)

inclination angle of 60°,
point 3

inclination angle of 70°,
point 2

inclination angle of 80°,
point 1

Figure 2-3 Displacement of the starting point of the penetration failure under different inclination angles of the structural plane (DDR=0.9)

inclination angle of 60°,
point 4

inclination angle of 70°,
point 2

inclination angle of 80°,
point 1

Figure 2-4 Displacement of the starting point of the penetration failure under different inclination angles of the structural plane (DDR=0.8)

2.2 Influence of the Shape of a Karst Cave between Parallel Structural Planes on the Failure Mode

In order to analyze the influence of the shape of the cave on the failure mode of a rock mass between structural planes, elliptical caves with the same DDR were simulated. The angles of the major axis were set as 0° , 40° , 80° , and 120° . Taking the horizontal axis as the reference, the major axis was rotated clockwise. The results are shown in Figure 2-5.

When the major axis of the cave was horizontal (0°), the penetration failure extended along the two ends of the major axis to the parallel structural planes. The upper rock bridge underwent tensile failure, and the lower rock bridge underwent shear failure. When the angle was increased to 40° , the starting point of the penetration failure extended from the top of the foundation pit to the position of the karst cave, and the failure of the upper rock bridge changed from tensile failure to shear failure. Then, at 80° , the penetration failure extended towards the

upper and lower structural planes along the major axis, and the two cracks were approximately parallel. Both the upper and lower rock bridges underwent shear failure. When the angle was increased to 120° , the penetration developed along the two ends of the major axis toward the parallel structural planes, and the failure mode of the rock bridge was tensile failure.

As is shown in Figure 2-6, for the structural plane with an inclination angle of 60° , the failure of the cave with a horizontal major axis (0°) penetrated the upper and lower structural planes at the second calculation point, exhibiting overall shear failure. When the major axis angle increased to 40° , the cave and the surrounding rock mass slid down as a whole. When the major axis of the cave was 80° , the failure mode was similar to that of 0° , i.e., shear failure passing through the cave. When the angle was further increased to 120° , the failure mode was the same as that of the first calculation point.

Figure 2-5 Failure modes of elliptical caves with different angles at point 1 for an inclination angle of 60° (DDR=0.9)

Figure 2-6 Failure modes of elliptical caves with different angles at point 2 for an inclination angle of 60° (DDR=0.9)

Due to space limitations, the conditions at other inclination angles will not be discussed here. By changing the shape of the cave, the DDR, and the angle of the major axis, the failure at the 10 calculation points was analyzed and the conclusions that follow were obtained.

(1) For inclination angles of 60°–80°, the presence of karst caves may cause penetration failure of the structural plane. However, there was a minimum DDR value for the occurrence of penetration failure at each inclination angle. When the DDR was less than the minimum value, overall slip failure occurs.

(2) For a given DDR value, as the inclination of the structural surface increased, the penetration position of the cave with a circular shape gradually moved to the top of the foundation pit, that is, the area of overall rock mass failure increased.

(3) Regarding the influence of the major axis angle of the elliptical cave on

the occurrence of penetration failure, when the inclination angle of the structural plane was 60° and 70°, and the major axis angle was 0°–40° and 80°–120°, the cave underwent penetration failure. When the major axis angle was 40°–80°, overall slip failure occurred.

3 Penetration Paths of Karst Cave Groups Based on Single Cave Failure Mode

Karst caves sometimes appear in groups in the slopes of foundation pits. In order to simplify the theoretical analysis of karst cave groups, we analyzed the feasibility of substituting karst cave groups with a single cave to obtain the same failure path.

Taking the structural plane with an inclination of 60° as an example (Figure 3-1), an asymptotic line with a DDR of 0.9 was drawn parallel to the structural plane. Then, 4 failure paths were selected based on possible rock failure angles. Path 1 was perpendicular to the line and 30° from the horizontal plane, path 2 was

45° from the horizontal plane, path 3 was 60° from the horizontal plane (the internal friction angle of the rock was 30°), and path 4 was perpendicular to the horizontal plane.

Figure 3-1 Similar failure modes of karst cave groups and circular karst cave

In this analysis, the DDR of a single cave was taken as the reference, the total DDR value of the cave group was changed, and the parameters of the cave group and the minimum total DDR needed to achieve similar failure paths were determined. For simplification, the distance between each karst cave was the

same, and four working conditions were simulated (i.e., 3, 5, 7, and 9 karst caves). The cloud diagram and the substitution scheme are shown in Figure 3-2 and Table 3-1 respectively.

Figure 3-2 Penetration failure paths of karst cave groups

It can be seen from Table 3-1 that by conducting a simple analysis of the distribution of the karst cave group, the single cave substitution parameter with the same failure mode can be quickly determined, thereby achieving simplified analysis of the karst cave group.

Table 3-1

Table 3-1 Karst cave group substitution table

Parameters of single cave					Parameters of cave groups					
No.	Failure path	radius/major semi-axis(mm)	DDR	Range of major and minor axis ratio	Radius (mm)	Cave distance (mm)	Analysis path	Number of caves	Total DDR	Failure mode
1	1	1125	0.9	3~9	250	875	1	3	0.6	Overall
2	1	1000	0.8	3~5	292	833	1	3	0.7	Penetration
3	1	1000	0.8	3~5	200	463	1	5	0.8	Overall

Stability analysis of rock slope and calculation of rock lateral pressure in foundation pit
with structural plane and cave development

4					225	450	1	5	0.9	Penetration
5					143	327	1	7	0.8	Overall
6					161	321	1	7	0.9	Overall
7					111	254	1	9	0.8	Overall
8					125	250	1	9	0.9	Overall
9					216	942	2	3	0.5	Overall
10					259	897	2	3	0.6	Penetration
11	2	1165	0.9	≥ 7	181	489	2	5	0.7	Overall
12	2	1035	0.8	≥ 8	207	475	2	5	0.8	Penetration
13					111	350	2	7	0.6	Overall
14					129	344	2	7	0.7	Penetration
15					115	262	2	9	0.8	Overall
16					241	1024	3	3	0.5	Overall
17	3	1125	0.9	1	289	968	3	3	0.6	Penetration
18	3	1000	0.8	1	144	567	3	5	0.5	Overall
19	3	1155	0.8	≥ 7	173	551	3	5	0.6	Penetration
20	3	1300	0.9	≥ 7	124	386	3	7	0.6	Overall
21					144	378	3	7	0.7	Penetration
22					128	288	3	9	0.8	Overall
23					250	1750	4	3	0.3	Overall
24					333	1583	4	3	0.4	Penetration
25					200	925	4	5	0.4	Overall
25	4	2250	0.9	≥ 10	250	875	4	5	0.5	Penetration
26	4	2000	0.8	≥ 9	143	655	4	7	0.4	Overall
27					179	631	4	7	0.5	Penetration
28					83	521	4	9	0.3	Overall
29					111	507	4	9	0.4	Penetration

Note: (1) The total diameter-to-distance ratio (DDR) of the karst cave group is the ratio of the diameter of all of the substitutive karst caves on the specified path and the length of the failure path.

(2) The penetration failure mode refers to penetration along the cave group, and the other failure modes are regarded as overall failure.

(3) Cave distance refers to the distance between the centroids of the karst caves.

Figure 4-1 Upper bound analysis model for
stability of rock slopes with structural plane
and karst cave development

c_1 and φ_1 are the shear strength
indices of the rock mass; and c_2 and φ_2
are the shear strength indices of the
structural plane. If K is the safety factor
of the foundation pit, then, the reduced
strength parameter is

$$\tan \varphi_{m1} = \frac{\tan \varphi_1}{K} \quad c_{m1} = \frac{c_1}{K} \quad 4.2$$

$$\tan \varphi_{m2} = \frac{\tan \varphi_2}{K} \quad c_{m2} = \frac{c_2}{K} \quad 4.3$$

The relationship between v_0 , v_1 , v_2 ,
and v can be expressed as

$$\begin{aligned} v_0 &= v_1 \cdot \cos \varphi_{m1} \\ v &= v_1 \cdot e^{\theta \tan \varphi_{m1}} \end{aligned} \quad 4.4$$

$$W_{total} = W_{BCEO} + W_{OEF} + W_q + W_{OFG} + W_p \quad 4.6$$

$$\text{where } W_{BCEO} = \gamma \cdot v_1 \cdot \cos \varphi_{m1} \left(\overline{BO} \cdot L + \frac{1}{2} r_0^2 \sin \varphi_{m1} \cos \varphi_{m1} \right)$$

$$W_q = q \cdot L \cdot v_1 \cdot \cos \varphi_{m1}$$

$$W_p = F_p \cdot v_1 \cdot T$$

$$\text{and } W_{OEF} = \frac{1}{2} \gamma \cdot r_0^2 \cdot v_1 \cdot \frac{T^3 \cdot [(3 \tan \varphi_{m1} \cdot \cos(\varphi_{m1} + \theta_1)) + \sin(\varphi_{m1} + \theta_1)] - 4 \sin \varphi_{m1}}{1 + 9 \tan^2 \varphi_{m1}}$$

$$E_{total} = W_{total} \quad 4.7$$

The above equation can then be rewritten as

$$AL^2 + BL + Cr_0^2 + Dr_0 + Er_0L + F = 0 \quad 4.8$$

$$\text{where } A = \gamma \cos \varphi_{m1} \cdot \tan \beta$$

$$B = \gamma \cos \varphi_{m1} \frac{h}{\cos \beta} - c_{m1} \cdot \cos \varphi_{m1} \tan \beta + q \cos \varphi_{m1}$$

$$C = \frac{1}{2} \gamma \left\{ \frac{T^3 \cdot [(3 \tan \varphi_{m1} \cdot \cos(\varphi_{m1} + \theta_1)) + \sin(\varphi_{m1} + \theta_1)] - 4 \sin \varphi_{m1}}{1 + 9 \tan^2 \varphi_{m1}} + T^2 \cdot \cos \varphi_{m2} \cdot \right.$$

$$\left. \tan(90 - \varphi_{m1} - \theta_1) \right\} + \frac{1}{2} \gamma \cdot \cos^2 \varphi_{m1} \cdot \sin \varphi_{m1}$$

$$D = -\frac{c_{m1}(T^2 - 1)}{\tan \varphi_{m1}} - c_{m2} \cdot T^2 \cos \varphi_{m2} \cdot \tan(90 - \varphi_{m1} - \theta_1)$$

$$E = -\gamma \cos \varphi_{m1} \cdot \frac{T}{\cos(90 - \varphi_{m1} - \theta_1)}$$

$$F = -c_m \cdot \cos \varphi_{m1} \cdot \frac{h}{\cos \beta} + F_p \cdot T - \cos \varphi_{m1} \cdot \sin \varphi_{m1}$$

$$v_2 = v_1 \cdot e^{\theta_1 \tan \varphi_{m1}}$$

The total work of the internal forces

E_{total} is:

$$\begin{aligned} E_{total} &= c_{m1} \cdot v_1 \cdot \cos \varphi_{m1} \overline{DE} + \\ &\frac{c_{m1} \cdot v_1 \cdot v_0^2}{r_0 \tan \varphi_{m1}} (e^{2\theta_1 \tan \varphi_{m1}} - 1) + E_{FG} \end{aligned} \quad 4.5$$

where:

$$\overline{DE} = L \cdot \tan \beta - \frac{r_0 T}{\cos(90 - \varphi_{m1} - \theta_1)} + \frac{h}{\cos \beta} +$$

$$r_0 \cdot \sin \varphi_{m1}$$

$$\text{and } T = e^{\theta_1 \tan \varphi_{m1}}$$

The total work of the external forces

is equal to the sum of the work done by
the weight of the sliding rock mass, the
earth pressure, the external force
equivalent to the weight of the overlying
rock, and the supporting force.

$$q = \left[(N - L)^2 \frac{\sin^2 \beta}{\cos \beta} \cdot \frac{1}{2} \gamma + (N - L)^2 \tan \beta \cdot \cos \beta \cdot \frac{1}{2} \gamma \tan \varphi_{m2} - \right. \\ \left. c_{m2} (N - L) \tan \beta \cdot \gamma \right] / L$$

$$\text{and } N = \left[H_c - \frac{h}{\sin(90^\circ - \beta)} \right] \cdot \tan(90^\circ - \beta)$$

To find the position of the karst cave failure between the structural planes, that is, to obtain the minimum weight required for the overlying rock mass, the minimum value of L is required. If $dL/dr_0 = 0$.

$$2Cr_0 + D + EL = 0 \quad 4.9$$

$$L = -\frac{2Cr_0 + D}{E} \quad 4.10$$

$$\text{and } L = f(\gamma, c_{m1}, \varphi_{m1}, \beta, r_0, c_{m2}, \varphi_{m2}, H_c)$$

The safety factor when the cave undergoes penetration failure and the supporting force $F_p = 0$ can be expressed as

$$K = f(\gamma, c_{m1}, \varphi_{m1}, \beta, L, c_{m2}, \varphi_{m2}, H_c) \quad 4.11$$

When the rock mass slides along the fracture surface, i.e., $K < 1$, the lateral pressure is

$$F_p = - (AL^2 + BL + Cr_0^2 + Dr_0 + Er_0L - c_{m1} \cdot \cos \varphi_{m1} \cdot \frac{h}{\cos \beta} - \cos \varphi_{m1} \cdot \sin \varphi_{m1}) / T \quad 4.12$$

$$F_p = f(\gamma, c_{m1}, \varphi_{m1}, \beta, L, c_{m2}, \varphi_{m2}, H_c, k) \quad 4.13$$

In the presence of caves, the rock mass parameters (c_1, φ_1) were reduced according to the ratio of the major axis

(or diameter) of the cave to the length of the failure path. Because the φ_1 value has little effect on the slope stability coefficient, the focus of the reduction was the c_1 value of the rock mass.

The above equations were programmed into MATLAB to obtain the solutions.

4.2 Comparison Between Calculation and Simulation Results of the Safety Factor

The simulation parameters ($c_1=300$ kPa, $\varphi_1=30^\circ$, $c_2=144$ kPa, $\varphi_2=30^\circ$, $\gamma=23$ kN/m³, and $H_c=25$ m) were substituted into Equation 4.11, and the safety factors under inclination angles of 60° and 70° were calculated. Figure 4-2 shows the relationship between the safety factor and the position of the cave centroid. For example, at an inclination angle of 60° for the structural plane, the load of the overlying rock mass at point 3 obtained from the numerical simulation was 43.55 kN/m, $L=9.63$ m. The theoretical safety factor K was 1.24, and the safety factor obtained from the simulation was 1.3,

which is slightly larger than the theoretical result. The calculation points are shown in Figure 2-1.

Similar findings were obtained when the inclination angle of the structural plane was 70° . However, the theoretical value was slightly larger than the simulation result at point 1, and the difference in the maximum safety factor was 0.08. Therefore, the difference between the theoretical result and the simulation result was minimal.

(a) 60° inclination angle

(b) 70° inclination angle

Figure 4-2 Comparison of theoretical and numerical results

4.3 Influence of the Presence of a Structural Plane on the Stability of the Foundation Pit Based on the Upper Bound Method

Figure 4-3 shows the influence of the presence of a structural plane on the

safety factor of the foundation pit, where $c_1=300$ kPa, $\phi_1=30^\circ$, $c_2=144$ kPa, $\phi_2=30^\circ$, $\gamma=23$ kN/m³, and $HC=25$ m. Figure 4-3(a), when the spacing of the structural planes was increased from 1 m to 8 m, the safety factor of the caves at the same position increased and reached the highest value at 6 m. The results for the curves between 6–8 m were the same, indicating a positive correlation between the safety factor of the foundation pit and the spacing of the structural planes. When the maximum safety factor is achieved, it remains unchanged even when the spacing continues to increase.

Figure 4-3(b) shows the relationship between the safety factor and the position of the cave under different inclination angles of the structural plane. It can be seen that when the inclination angle was 60° , the safety factor was the smallest; and when the inclination angle was 70° , the safety factor reached the maximum value. Moreover, the closer the cave was to the upper part of the foundation pit, the larger the depth of the lower structural plane was, and the smaller the influence of the inclination angle on the safety factor was.

(a) structural plane spacing

(b) inclination angle

Figure 4-3 Influence of the structural plane occurrence on the safety factor

4.4 Influence of the Rock Mass Parameters on the Stability of the Foundation Pit Based on the Upper Bound Method

Figure 4-4 shows the influences of the mechanical parameters of the rock mass on the safety factor, where $c_2=144$ kPa, $\varphi_2=30^\circ$, $\gamma=23$ kN/m³, $h=2.25$ m, $H_C=25$ m, and $\beta=60^\circ$. As the cohesive force continuously decreased, i.e., the DDR increased, the influence of the cave at the same position on the safety factor of the foundation pit gradually increased, and the safety factor decreased. Moreover, the influence of the internal

friction angle on the safety factor was relatively small. The five curves between 20° to 40° almost overlapped. Therefore, the internal friction angle did not significantly affect the safety factor of the foundation pit.

(a) rock mass cohesion

(b) internal friction angle

Figure 4-4 Influences of the rock mass parameters on the safety factor

4.5 Influence of the Structural Plane Parameters on the Stability the Foundation Pit Based on the Upper Bound Method

Figure 4-5 shows the influences of the mechanical parameters of the structural plane on the safety factor of the foundation pit, where $c_1=300$ kPa, $\varphi_1=30^\circ$, $\gamma=23$ kN/m³, $H_C=25$ m, $\beta=60^\circ$,

and $h=2.25$ m. By changing the cohesive force of the structural plane, it was found that the five curves intersect at a distance of 11 m from the centroid. Thus, when the distance was less than 11 m, the lower the cohesion was, the lower the safety factor was. At distances of >11 m, the lower the cohesion was, the higher the safety factor was, but the magnitude of the increase was small. Similarly, by changing the internal friction angle of the structural plane, it was found that the influence on the safety factor was limited to a distance of 7 m from the cave centroid to the slope of the foundation pit. When the distance was >7 m, the safety factor was not affected. Therefore, the friction angle only affected the penetration of the karst cave at the bottom of the foundation pit.

(a) rock mass cohesion

(b) internal friction angle

Figure 4-5 Influences of the structural plane parameters on the safety factor

For the same spacing of the structural planes, the relationship between the distance from the cave centroid to the slope of the foundation pit and the depth of the structural plane is shown in Figure 4-6, where $c_1=300$ kPa, $\phi_1=30^\circ$, $c_2=144$ kPa, $\phi_2=30^\circ$, $\gamma=23$ kN/m³, and $h=2$ m. It can be seen that at the limit, when the structural plane's inclination angle was 60° , the minimum distance decreased as the depth of the structural plane increased. When the inclination angle was 80° , the minimum distance increased as the depth of the structural plane increased. For the other angles between 60° and 80° , the minimum distance from the centroid to the slope surface initially increased and then decreased as the depth of the structural plane increased. Thus, the results show that the failure risk of the foundation pit was greater for an inclination angle of 60° than for the other angles.

Figure 4-6 Relationship between the depth of the lower structural plane and the centroid position of the cave in the ultimate state

4.6 Relationship Between the Rock Mass Parameters and the Cave Position Based on the Upper Bound Method

Figure 4-7 shows the influences of the rock mass parameters on the distance from the cave centroid to the slope surface. It can be seen that as the depth of the lower structural plane increased, the distance initially increased slowly, gradually decreased, and then increased again after reaching the minimum value. In Figure 4-7(a), when $c_1=300$ kPa, the minimum centroid distance occurred at a structural plane depth of about 34 m. As the cohesion increased, the point of the minimum distance gradually increased. In foundation pit engineering, the excavation depth is generally less than 34 m. Therefore, for urban foundation pit engineering projects, as the depth of the lower structural plane increases, the distance for the cave to reach the ultimate

state gradually decreases. The internal friction angle of the rock mass has little effect on the ultimate state of the foundation pit. The safety factor of the foundation pit is primarily determined by the position of the cave relative to the structural plane in Figure 4-7(b).

(a) rock mass cohesion

(b) internal friction angle

Figure 4-7 Influences of the rock mass parameters on the centroid position of the cave

Figure 4-8 shows the influences of the mechanical parameters of the structural plane on the position of the cave centroid in the ultimate state. The intersection point in Figure 4-8(a) occurred when the depth of the lower structural plane was about 28 m. At this point, the position of the cave centroid at

the ultimate state of the foundation pit did not change with the parameters of the structural plane. In Figure 4-8(a), there are two pivot points at $c_2=100$ kPa and $c_2=150$ kPa, after which the change exhibits an increasing trend. When the internal friction angle of the structural plane changed Figure 4-8(b), there was a pivot point at each angle. As the internal friction angle increased, the position of the pivot point became deeper.

(a) rock mass cohesion

(b) internal friction angle

Figure 4-8 Influences of the structural plane parameters on the centroid position of the cave

4.7 Supporting Force Calculation Based on the Upper Bound Method

In order to provide design guidance

for engineering support systems, the safety assessment equation needs to reflect the supporting force required by the foundation pit when the safety factor is below the specified value. Figure 4-9 shows the relationship between the inclination angle of the structural plane and the supporting force (negative), where $c_1=300$ kPa, $\phi_1=30^\circ$, $c_2=144$ kPa, $\phi_2=30^\circ$, $\gamma=23$ kN/m³, $h=2.25$ m, and $k=2.0$. When the inclination angle of the structural plane was changed from 60° to 80° , the supporting force required by the foundation pit exhibited an increasing-decreasing-increasing pattern. The required supporting force was the largest when the inclination angle was 80° , and the distance from the cave centroid to the slope of the foundation pit was large. Therefore, the closer the cave between parallel structural planes is to the ground surface, the larger the required supporting force is. This finding is also supported by the variations in the strength coefficient of the supporting force in Figure 4-10

Note: the negative value of support force points out of the foundation pit

Figure 4-9 Relationship between the supporting force and the inclination angle of the structural plane

Note: the negative value of support force points out of the foundation pit

Figure 4-10 Relationship between the supporting force and the strength reduction coefficient

5 Applicability of the Different Lateral Pressure Calculation Methods for Rock Foundation Pits

Table 6-1 lists the common methods used to calculate the lateral soil pressure for a vertical slope with no overload at the top. Compared to other methods, the Rankine's lateral earth pressure formula (without considering the cohesion of the rock mass) obtained the smallest value. If the friction angle between the rock mass and the wall is considered, the results of the Coulomb's lateral earth pressure formula at 10 m and 20 m are much larger than those of the Rankine's lateral earth pressure formula at the same depths, but the results are similar at a depth of 30 m. For rock foundation pits, if there is no outwardly inclined structural plane within 10 m, the rock mass has a certain

self-stabilizing ability. Therefore, Coulomb's lateral earth pressure formula may overestimate the pressure in the shallow region.

The wedge algorithm in Table 5-1 considers the friction angle between the interfaces, and the result is slightly larger than that of the Coulomb's active earth pressure when the friction angle between the soil and the rock is 18° . If the calculated depth of each layer is taken as the starting point of the slip surface (the fracture angle is 60°) and the earth pressure formula for slip along an outwardly inclined rigid structure surface is used, the earth pressure of each layer is increased higher than the result of the wedge algorithm. This is because the friction angle of the contact surfaces in the wedge algorithm is an empirical parameter, whereas the value used in this study was the minimum value of the empirical parameter. In the equation for calculating the slip along an outwardly inclined rigid structural surface, the shear strength value measured via indoor shear tests is used. The parameters of the two methods are different.

If the position of the karst cave is in the top of the pile, then Equation 4.13, which was obtained using the upper bound method, becomes the equation for non-karst cave conditions. When karst caves are not taken into account, the

result of the upper bound method is slightly lower than that of the earth pressure of the slip along the outwardly inclined rigid structural surface. This is because the result of the upper bound method is the optimal solution that satisfies the energy equation, that is, the minimum residual thrust when the slope reaches the ultimate state. In a sense, the result calculated along the outwardly inclined structural plane satisfies the upper bound method. However, for the wedge algorithm, in order to obtain an accurate calculation result, an accurate

value of the friction angle of the interface is needed.

6 Conclusion

In summary, the proposed slope stability equation based on the upper bound method can be used for slopes with and without karst caves. With the help of the karst cave group substitution scheme presented in Table 3-1, complex cave conditions can be simplified in the theoretical analysis of complex rock slopes.

Table 6-1 Earth pressure calculation table

Basic calculation parameters								
Active earth pressure calculation methods	Soil weight (kN/m³)	Internal friction angle (°)	Cohesion (kPa)	Poisson's ratio	Additional load on slope top (kN/m²)	Angle between the fracture surface and the horizontal plane (°)	Internal friction angle of outwardly inclined structure plane (°)	Cohesion of outwardly inclined structure plane (kPa)
23	45	150	0.17	0	60	15	0	0
Active earth pressure calculation methods			Calculated depth (m)	Active earth pressure resultant force (kN)		Comment		
Rankine's active earth pressure			0	0		Cohesion not considered; failure angle: 45°+φ/2		
			10	197.3				
			20	789.2				
			30	1775.8				
Coulomb's active earth pressure			0	0		Friction angle between rock mass and wall: 0.33φ; failure angle: 45°+φ/2		
			10	966.4				
			20	1573.7				
			30	1814.9				
Wedge algorithm			0	0		Friction angle between fill soil and rock mass:18°; failure angle:		
			10	499.3				
			20	1997.1				

	30	4493.6	60°
Slip along the outwardly inclined rigid structure plane	0	0	Failure angle: 60°
	10	542.9	
	20	2171.7	
	30	4886.4	
Upper bound method	0	0	No karst cave; failure angle: 60°
	10	314	
	20	1834.7	
	30	4351.2	

Declaration of Competing Interest

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

Acknowledgement

This work was supported by the National Natural Science Foundation of China (grant no. 52074264, 51174194), the National Key Research and Development Program of China (grant no. 2016YFC0600903).

References

1. Wu SG. Stability and control of architectural rock slopes [D]. Chongqing University, 2005.
2. Sun JS. The numerical analysis of strip partition method[J]. Chinese Journal of Geotechnical Engineering, 1984, 12-16.
3. Ruan HY, Xu ZY. Limit equilibrium theory for slope engineering in complex rock masses[J]. Chinese Journal of Rock Mechanics and Engineering, 1993(1), 11-19.
4. Kumsar H, Aydan O, Ulusay R. Dynamic and static stability assessment of rock slopes against wedge failures[J]. Rock Mechanics and Rock Engineering. 2000, 33(1): 31-51.
5. Xu MY, Wang WM, Chen SH. Research on the dangerous sliding-block combination of rock slopes[J]. Rock and Soil Mechanics, 2000(2), 148-151.
6. Chen XL. Quasi-static analysis of seismic stability of rock slope with double slide blocks[J]. Chinese Journal of Underground Space and Engineering, 2016, 12(5), 1403-1409.
7. Chen. Limit analysis and soil plasticity[M]. Elsevier Scientific Pub. Co., 1975.
8. Chen ZY, et al. Stability analysis of rock slopes [M]. China Water Conservancy and Hydropower Press, 2005.
9. Zhang ZX, Xu Y, Huang X. Upper bound solution of limit analysis for stability of block- layered rock slope[J]. Journal of Tongji University (Natural Science), 2010, 38(5), 656-663.
10. Wang ZD, Xia YY, Xia GB, et al. Upper bound limit analysis method for stability analysis of bedding rock

- slopes[J]. *Rock and Soil Mechanics*, 2015, 36(2), 576-583.
11. Liu TP, Li SH, Liu XY. Study of dem and fem combination method for stochastic structural plane in jointed rock slope[J]. *Chinese Journal of Rock Mechanics and Engineering*, 2014, 33(S1), 3114-3122.
12. Zhang SR, Tan YS, Wang C, et al. Research on deformation failure mechanism and stability of slope rock mass containing multi-weak interlayers[J]. *Rock and Soil Mechanics*, 2014, 35(6), 1695-1702.
13. Lin H, Cao P, Li JT, et al. Numerical analysis of failure modes and stability of stratified rock slopes[J]. *Rock and Soil Mechanics*, 2010, 31(10), 3300-3304.
14. Kou XD, Zhou WY, Yang RQ, et al. Study on anchorage stability and mechanism of high slope in three-gorge locks[J]. *China Civil Engineering Journal*, 2002(1), 68-73.
15. Gao YT, Xiao S, Wu SC, et al. Numerical simulation of the deformation and failure characteristics of consequent rock slopes and their stability[J]. *Chinese Journal of Engineering*, 2015, 37(11), 1403-1409.
16. Li YS, Chen WF, Li XP, et al. Stability assessment of rock slope based on fuzzy neural network[J]. *Journal of Wuhan University of Technology*, 2013, 35(1), 113-118.
17. Zhou N, Fu HL, Yuan Y. Evaluation approach of slope stability based on fuzzy neural network[J]. *Chinese Journal of Underground Space and Engineering*, 2009, 5(S2), 1826-1832.
18. Wang YX. Application of fuzzy mathematics to slope stability analysis[J]. *Rock and Soil Mechanics*, 2010, 31(9), 3000-3004.
19. Tao L, Guodong L, Cong W, et al. The probability and sensitivity analysis of slope stability under seepage based on reliability theory[J]. *Geotechnical and Geological Engineering*. 2020 (prepublish).
20. Li K, Ju NP. Integrated application of Monte-Carlo simulation for landslide reliability analysis[J]. *The Chinese Journal of Geological Hazard and Control*, 2014, 25(1), 23-27.
21. Jiang SH, Ouyang S, Feng YW, et al. Reliability analysis of jointed rock slopes using updated probability distributions of structural plane parameters[J]. *Rock and Soil Mechanics*, 2021, 42(9), 2589-2599.
22. You ZJ, et al. Geotechnical plasticity theory and its application in underground engineering [M]. *Coal Industry Press*, 2017.