Study on upper limit solution and its application on bearing capacity of rock slope foundation

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Abstract. The determination and analysis of the bearing capacity of a slope foundation has become an important research topic in foundation design theory. To explore the ultimate bearing capacity of the rock foundation adjacent to a slope on the basis of the shear failure mode of a rock foundation on a semi-infinite plane, the failure mode of the rock foundation near the slope was established by analyzing the failure mechanism of the Bell solution. The upper limit theorem of limit analysis was used to build a velocity field allowable for maneuvering. Using this method, the ultimate bearing capacity of the Sandaozhuang open-pit mine was analyzed and calculated. The results show that the foundation load of the mine is less than the allowable bearing capacity; hence, the building on the slope meets the requirements of foundation bearing capacity. At the same time, this paper uses this method and existing research methods to compare and analyze this project with others, proving the rationality and feasibility of the proposed method. This paper further explores factors affecting the ultimate bearing capacity, such as the horizontal setback distance of the footing from the edge of the slope and the dip angle. It is concluded that the ultimate bearing capacity increases with increases in the distance to the slope and decreases in slope angle. The study also shows that the calculations developed and proposed in this paper for the bearing capacity of the rock foundation adjacent to the slope are reasonable and feasible, can be applied to the calculations of the bearing capacity of rock foundations near the slope, and have guiding significance for the project.

Key words: foundation adjacent to the slope; upper limit method; ultimate bearing capacity; rock foundation; Bell solution; Sandaozhuang Open-pit mine

1. Introduction

Rock has the characteristics of high strength and low deformation, and it is the first choice of natural foundation in construction engineering. At the same time, because of the discontinuity, heterogeneity, and anisotropy of rock, studies determining the theoretical bearing capacity of rock foundations are less than perfect. Especially for the determination of the bearing capacity of a rock foundation adjacent to a slope, there is less global research on classical theoretical solutions. Regarding the calculation methods of the bearing capacity of the rock foundation, the most classical calculation method is the Bell solution proposed in 1915 [1], which is suitable for the calculation of the bearing capacity of
homogeneous rock masses, including intact and weak fractures. Thus, a close relationship between the bearing capacity of rock foundations and rock mass structural planes is indicated. At the same time, it proposes the most common shear failure mode of rock, and the theoretical calculation formula for ultimate bearing capacity of rock foundations is determined by the limit equilibrium method. Later, with further development of plastic mechanics, the use of limit analysis methods in rock and soil has made great progress. As the limit analysis method is used to analyze the bearing capacity of the foundation, there is no need to accurately determine the stress distribution inside each rigid block in the failure mechanism. It is suitable for bearing capacity calculation and stability analysis of complex geotechnical engineering structures. In the mid-twentieth century, DC Drucker and W Prager [2] constructed static and velocity fields for soils, derived upper and lower theorems, and established a complete limit analysis theory. American scholar Chen [3] summarized the existing research results and systematically expounded the principles of the limit analysis method and its applications in rock and soil bodies and some rock-like materials, which provided significant progress for the study of limit analysis in rock and soil. Collins et al. [4] put forward the double-slider failure mode of slopes when studying slope failure, and the calculation of this failure mode is relatively simple. Saran et al. [5] adopted the C-M failure criterion for the bilateral failure model of the slope-side foundation; however, the soil parameters on both sides of the foundation changed, and the analytical solution of the problem was obtained by the limit equilibrium method and the limit analysis method. Michalowski [6-7] successively considered the multislider failure mode of circular foundations and rectangular foundations for three dimensions and obtained ideal results using the upper theorem. Lyamin [8] combined the finite element with the upper theorem, which was determined to be faster and more accurate than the commonly used methods in 2D and 3D. This also provided a new idea for solving the problem of the ultimate bearing capacity of slope foundations. Wang et al. [9-10] used Sokolovski's loose medium statics theory and Prandtl's weightless foundation slip network and constructed an asymmetric failure mode of slope foundations and the foundation adjacent to the slope through FLAC-3D. Yang et al. [11] considered the asymmetry of the destruction of the foundation on the slope and obtained the ultimate bearing capacity through upper limit analysis; however, they did not consider the impact of slope distance on ultimate bearing capacity. Hu et al. [12-14] successively constructed a one-side sliding failure mode and a two-side sliding failure mode under asymmetric characteristics of the failure of a strip foundation adjacent to a slope, constructed the velocity field allowable for maneuvering, and deduced the expression forms of the bearing capacity of the foundation adjacent to the slope in two cases. Then, for the multislider failure mode, nonlinear strength criteria were used to calculate the ultimate bearing capacity of the soil slope, which has reference significance for the study of the bearing capacity of rock foundations. Xiao [15] used the upper limit theorem to determine the failure mode of the critical instability of slopes and the ultimate bearing capacity of the failure mode. By changing the internal friction angle, slope distance, and slope angle, he explored the factors affecting ultimate bearing capacity.

Based on the imperfections of the classical calculation method of the bearing capacity of a rock foundation adjacent to a slope in the current project, this paper adopts the double-slider failure mode for the rock foundation adjacent to the slope. The study constructs the shear failure mode of the rock foundation adjacent to the slope on the basis of the Bell solution. Then, we establish the velocity field allowed by the maneuverability using the C-M strength criterion, deduce the expression form of the bearing capacity of the foundation, and calculate and analyze it in combination with the actual project case so as to explore a convenient calculation method for the bearing capacity of the rock foundation.

2. Failure mode and upper limit analysis of the ultimate bearing capacity of foundation

2.1. Principle of upper limit analysis method

In the upper bound theorem, in a hypothetical deformation mode satisfying the velocity boundary condition and compatibility condition of strain and velocity, that is, in the velocity field of motion permission, the load obtained by external power equal to the internal power consumed must be greater than the actual load of failure. Therefore, the upper theorem is to be considered for the failure mode and energy loss, and the question of whether the object meets the equilibrium condition should not be
considered. According to the theory of upper limit analysis, for a permitted velocity field, the power of external force and the dissipation rate of internal energy must be equal. In this problem, the external force is the bearing capacity of the foundation, the gravity of rock mass, and the overload on the outside of the foundation. The dissipation rate of internal energy is the dissipation rate of cohesion on the failure surface and velocity cross-section:

$$\sum \int Fv' dV + \sum \int pv' dA = \sum \int c \Delta v_t dS$$

(1)

where $V, A,$ and $S$ are the integral volume, area and arc length, respectively; $F$ and $p$ are the physical force and surface force acting on the object, respectively; $v'$ is the speed of the object under the action of the corresponding external force; $c$ and $\Delta v_t$ are the cohesive force and intermittent speed on the failure surface and the speed section, respectively.

### 2.2 Failure mode and composition of the velocity field in semi-infinite plane foundation

The Bell solution is a classic theory in solving the bearing capacity of rock foundations. It is suitable for complete rock masses or broken and weak rock masses (containing four or more structural planes). To analyze the failure mechanism of the rock slope, this paper starts from the Bell solution of a flat foundation, analyzes the failure mechanism of the bearing capacity of a foundation under the flat foundation, and then constructs the velocity field under the flat foundation and compares it to obtain the failure mode and velocity field of the rock slope.

For rock foundations, assuming that the foundation width is $b$, it is equivalent to a uniform strip load set on a semi-infinite plane, as shown in Figure 1. Under load action, the failure area is divided into active failure areas and passive failure areas. The role of the transition zone line segment DB is to transfer the normal stress between the passive failure zone and the active failure zone.

![Figure 1. Description of the failure mode of the Bell solution. AD is the position of the foundation, and its length is b.](image_url)

The basic assumptions are as follows:

- The failure surface formed under load consists of two mutually perpendicular planes, which are shown in Figure 1 as follows: the angle between AB plane and the horizontal plane is $45^\circ + \phi/2$, and the angle between BC plane and the horizontal plane is $45^\circ - \phi/2$;
- The action range of load $Q_u$ is quite long, and the rock foundation can be considered a stress state;
- The failure surface is smooth and flat, and there is no shear stress in the vertical plane between Wedge I and Wedge II nor in the bearing plane;
- For each wedge, the average volume force can be used.

According to the hypothesis of the Bell solution [1] and related theories in literature [16], we can make...
the velocity field of the failure mode, as shown in Fig. 2. According to the relevant fluidity rule, the velocities \( V_1 \) and \( V_2 \) on the two failure surfaces are, respectively, the \( \varphi \) angles with their respective failure surfaces. As the plastic zone only plays the role of transferring force and as there is no shear stress, when the velocity direction of the plastic zone is vertical, there will be no energy dissipation in the plastic zone at this time. When \( \varphi \) is 0, the velocity field is as described in [3].

![Figure 2. Velocity field in the Bell solution failure mode](image)

2.3. Failure mode of the rock slope
The failure mode of the rock slope foundation is shown in Figure 3 below. According to existing research [17], the failure mode is divided into two parts. Block I is the active failure area. When the base is smooth, \( \alpha_1 = 45^\circ + \varphi/2 \), and there is no active load on the slope, which is the free plane. Block II is the passive failure area. The angle between the passive failure surface and the slope is \( \alpha_2 = 45^\circ - \varphi/2 \). When the slope is large, the failure mode is as shown in Figure 3 (a); when the slope is small, the failure mode is as shown in Figure 3 (b); when the dip angle is 0, the failure mode is the Bell solution.

![Figure 3. Failure mode of the rock slope foundation](image)

To explore the bearing capacity and calculation convenience of the foundation under the above failure mode, the following basic assumptions were made under assumptions of the Bell solution:

- The foundation is a strip foundation; the ultimate bearing capacity \( Q_u \) is evenly distributed; the foundation width is \( b \); the distance between the outer edge of the foundation and the top of the slope is \( L \); and parameter \( a \) is introduced, where \( a = L/b \). Relative sliding between the bottom of the foundation and the foundation rock is allowed, and there is no
shear stress at the bottom of the foundation;

- The foundation is a shallow foundation, regardless of the influence of the shear strength of the buried rock on the bearing capacity of the foundation. The most common treatment method is adopted for the influence of the buried depth of the foundation: the influence of the rock above the foundation is replaced by the vertical average load \( q \) acting on the outer top surface of the foundation, which is equal to the product of the buried depth \( h \) of the foundation and the heavy \( \gamma \) of the rock above the foundation, that is, \( q = \gamma h \).

2.4. Composition of the velocity field of the rock slope
In this paper, according to the failure mode, an allowable velocity field of maneuvering is constructed. As shown in Figure 4, Blocks I and II are rigid bodies. It is assumed that the velocity of the vertical downward movement of the foundation is \( V_p \), and the horizontal sliding velocity is \( V_0 \). In fact, Block I moves obliquely downward at velocity \( V_1 \), and the rock at the lower left side of AB is still. Therefore, \( V_1 \) is the discontinuity velocity of discontinuity AB, which is determined by the correlation flow rule [16]. The discontinuity velocity \( V_1 \) and discontinuity AB are rock friction angles \( \phi \), and the discontinuity velocity \( V_1 \) is determined by the vertical downward movement velocity \( V_p \) and horizontal sliding velocity \( V_0 \) of the foundation. In the same way, the discontinuity velocity on discontinuity BC is \( V_2 \), which has a friction angle \( \phi \) with discontinuity BC.

For the relative velocity on the DB plane, because there is only normal stress but no internal energy dissipation on the DB plane, the relative velocity \( V_{12} \) on the DB plane is parallel to the DB plane; hence, no energy dissipation on the DB plane can be guaranteed in this manner.

\[
V_p = V_1 \sin(\alpha - \phi) \tag{2}
\]

\[
V_0 = V_1 \cos(\alpha - \phi) \tag{3}
\]
As the discontinuous velocity of BD plane can be obtained from the vector difference between $V_1$ and $V_2$, these three velocities can form a velocity relationship as shown in Figure 5 (b). According to the sine theorem of the triangle, the following relationship can be obtained:

$$
\frac{V_1}{\sin \left(\frac{\pi}{2} + \eta - \alpha_2 - \varphi\right)} = \frac{V_2}{\sin \left(\frac{\pi}{2} - \alpha_1 + \varphi\right)} = \frac{V_{12}}{\sin \left(\alpha_1 + \alpha_2 - \eta\right)}
$$

(4)

- By relationship (4), the discontinuous velocity $V_{12}$ on the BD cross-section is expressed by the velocity $V_1$, and its magnitude can be expressed as follows:

$$
V_{12} = \frac{\sin \left(\alpha_1 + \alpha_2 - \eta\right)}{\sin \left(\frac{\pi}{2} + \eta - \alpha_2 - \varphi\right)} \cdot V_1
$$

(5)

- Via relationship (4), the speed $V_2$ is expressed by the speed $V_1$, and its magnitude can be expressed as follows:

$$
V_2 = \frac{\sin \left(\frac{\pi}{2} - \alpha_1 + \varphi\right)}{\sin \left(\frac{\pi}{2} + \eta - \alpha_2 - \varphi\right)} \cdot V_1
$$

(6)

3. Calculation of bearing capacity of the foundation adjacent to the slope

3.1. Upper bound theorem power calculation

After calculating the relationship between each speed, use the point multiplication of force and speed to calculate the value of each power. The power of the external force consists of gravity power, overload power, and the power of ultimate bearing capacity. The power of internal energy dissipation is determined by the power of cohesion.

3.1.1. Calculation of external force power

- Gravity power $W_I$, $W_{II}$

First, calculate the area of Blocks I and II, respectively, expressed by $S_I$ and $S_{II}$:
\[ l_1 = a + \frac{\tan \alpha_1}{\tan \eta} \]  
\[ l_2 = l_1 \frac{\sin \eta}{\sin \alpha_2} \]  
\[ A_1 = (a + l_1) \tan \alpha_1 \]  
\[ A_2 = l_1 l_2 \sin(\eta - \alpha_2) \]  
\[ S_I = \frac{1}{2} b^2 \tan \alpha_1 \]  
\[ S_{II} = \frac{1}{2} b^2 (A_1 + A_2) \]

where \( \eta \) is the dip angle, \( b \) is the width of strip foundation, and \( a \) is the ratio of \( L \) over \( b \).

The gravity power of Blocks I and II can be represented by \( W_I \) and \( W_{II} \).

\[ W_I = G_I \cdot V_I = \gamma S_I V_1 \cos\left(\frac{\pi}{2} - \alpha + \varphi\right) \]  
\[ W_{II} = G_{II} \cdot V_2 = \gamma S_{II} V_1 \frac{\sin\left(\frac{\pi}{2} \cdot \alpha_1 + \varphi\right)}{\sin\left(\frac{\pi}{2} + \eta \cdot \alpha_2 - \varphi\right)} \cos\left(\frac{\pi}{2} - \eta + \alpha_2 + \varphi\right) \]

where \( G_I \) and \( G_{II} \) are the gravities of two sliding blocks, respectively, and \( \gamma \) is the weight of the rock.

- Overload power \( W_q \)

\[ W_q = q \cdot V_2 L = qabV_1 \frac{\sin\left(\frac{\pi}{2} - \alpha_1 + \varphi\right)}{\sin\left(\frac{\pi}{2} + \eta \cdot \alpha_2 - \varphi\right)} \cos\left(\frac{\pi}{2} - \eta + \alpha_2 + \varphi\right) \]

where \( q \) is the vertical uniform pressure caused by the burial depth outside the foundation, and \( L \) is the distance between the outer edge of the foundation and the top of the slope, which can be expressed as follows:

\[ q = \gamma h \]  
\[ L = ab \]

- Foundation bearing capacity power \( W_{Q_b} \):

\[ W_{Q_b} = Q_u \cdot V_2 b = Q_u b V_1 \cos\left(\frac{\pi}{2} - \alpha + \varphi\right) \]

3.1.2. Internal energy dissipation power

The internal energy dissipation power of the problem studied in this paper is the energy loss rate of the velocity cross-section, and the energy loss rate of the velocity cross-section is the cohesion point on the surface multiplied by the velocity vector on the surface. The velocity cross-section has three
surfaces: AB, DB, and BC. The cohesion on each surface can be expressed by \( c \), and the specific analysis method is as follows:

\[
D_{ab} = c l_{ab} V_1 \cos \varphi = c \frac{b}{\sin \alpha_i} V_1 \cos \varphi
\] (19)

\[
D_{bc} = c l_{bc} V_2 \cos \varphi = c b l_2 \frac{\sin \left( \frac{\pi}{2} - \alpha_i - \varphi \right)}{\sin \left( \frac{\pi}{2} + \eta \cdot \alpha_i - \varphi \right)} V_1 \cos \varphi
\] (20)

\[
D_{bd} = 0
\] (21)

3.2. Expression derivation of bearing capacity of slope foundation

To calculate the ultimate bearing capacity of the strip foundation of the rock slope, the external force power and energy dissipation rate obtained from the foregoing analysis shall be substituted into Energy Equation (1), and the substituted equation is as follows (22):

\[
W_{q_1} + W_q + W_i + W_R = D_{ab} + D_{bc} + D_{bd}
\] (22)

After simplification, the expression of \( Q_u \) can be obtained and written as the expression of traditional ultimate bearing capacity:

\[
Q_u = \frac{1}{2} \gamma b N_q + c N_c + q N_q
\] (23)

where,

\[
N_q = \sin(\alpha_i - \varphi) [\tan \alpha_i \sin(\varphi - \alpha_i) - (A_i + A_t) \frac{\sin(\varphi - \alpha_i)}{\cot(\eta \cdot \alpha_i - \varphi)}]
\] (24)

\[
N_c = \sin(\alpha_i - \varphi) [\frac{\sin \eta \cos(\varphi - \alpha_i) \cos \varphi}{\sin \alpha_i \cos(\eta \cdot \alpha_i - \varphi)} + \frac{\cos \varphi}{\cos \alpha_i}]
\] (25)

\[
N_q = \sin(\varphi - \alpha_i) [\frac{-a \sin(\alpha_i + \varphi - \eta) \cos(\varphi - \alpha_i)}{\cos(\eta \cdot \alpha_i - \varphi)}]
\] (26)

According to the foregoing analysis, the expression of ultimate bearing capacity and the bearing capacity coefficient can be obtained.

4. Calculation and analysis of engineering cases

4.1. Engineering background

Sandaozhuang open-pit mine is a large-scale open-pit mine in China. The designed minimum open-pit mining elevation is +1072 m, the final mining depth is 486 m, and the final slope angle designed for the hanging wall is about 50°, which is a typical high and steep slope [18]. There is an important observation platform building on the slope top of the hanging wall of the mine (Class I). It is responsible for the propaganda window function of the mine. It is significant to the mine and enterprise for the exhibition of enterprise products and mining technology and visits and inspections of superiors and colleagues. The observation platform is typically built on the deposit of slag discharge.
products at an early stage of the mine at the top of the slope, which is a circularly reinforced concrete frame structure with two floors above and below the foundation. The circular foundation is located on the rock foundation under the slag soil, which is an independent foundation under the column. The diameter of the disc foundation of the observation platform is about 23 m, and the upper building applies 38 kPa of uniform load to the foundation through the foundation. The relative position of the observation platform and the slope is shown in Figure 6.

Figure 6. Relative position of viewing platform and rock slope. The accumulation of upper waste rock is caused by the accumulation of waste slag discharged from early mine production. The foundation is a circular one located on the second layer of rock mass. Above the foundation is a concrete pillar that passes through the upper waste rock accumulation.

Because of large-scale landslides in the adjacent slope area of the building, the surrounding area of the building can be cracked with different degrees of settlement. To explore the causes of landslides and demonstrate whether the bearing capacity of the building foundation meets the requirements and ensures the safety of the observation platform and slope, the bearing capacity of the rock foundation will be calculated by the method proposed in this paper.

4.2 Parameter selection
The upper waste rock pile is mainly skarn waste rock, and the lower fracture zone rock mass is primarily middle weathered schist. In this paper, the parameters of the rock are shown in Table 1 according to reference [18] and engineering rock mass classification.

| Rock type                        | $\gamma$ (kN/m$^3$) | $c$ (kPa) | $\varphi$ ($^\circ$) |
|----------------------------------|----------------------|-----------|----------------------|
| Upper waste rock accumulation    | 17.7                 | \         | \                    |
| Lower fracture zone rock mass    | 24                   | 36        | 32                   |

The depth of the upper waste rock accumulation was 5 m, and the diameter of the circular foundation was 23 m. It is located on the surface of the rock mass in the lower fracture zone. The buried depth is about 5 m, which is a shallow foundation. The slope is taken as 50° and the distance to the slope is measured as 13.5 m, regardless of the base friction.

4.3. Results analysis
As the foundation of the mine is a circular one, according to existing research, the shape coefficients are multiplied by some items of the ultimate bearing capacity under the strip foundation, multiplied by
1.2 in the $N_c$ term and by 0.7 in $N_γ$. In engineering, 1/2~1/3 of the ultimate load is generally taken as the allowable bearing capacity of the foundation. Therefore, the ultimate bearing capacity under this condition is obtained by the method in this paper as shown in Table 2 below.

### Table 2. Calculation results

| $N_γ$ | $N_c$ | $N_q$ | Ultimate bearing capacity (kPa) | Allowable bearing capacity (kPa) |
|-------|-------|-------|-------------------------------|----------------------------------|
| 0.773 | 8.782 | 0.206 | 546.83                        | 182.28~273.42                    |

The base-load at the bottom of the observation platform is the self-weight of the observation platform, which is 38 kPa. The base-load is less than the allowable bearing capacity calculated by this method. Therefore, it can be judged that when the mine slope is excavated to the final slope angle, the foundation bearing capacity meets these requirements.

To further prove the rationality and basis of this method, other research methods were used to comparatively analyze the results. These calculation results are shown in Table 3 below.

### Table 3. Comparison of results

| Method | Ultimate bearing capacity (kPa) | Wang | Hu | Proposed method |
|--------|---------------------------------|------|----|-----------------|
|        |                                 | 974.11 | 439.47 | 546.83          |

The difference between the method used in this paper and the other two methods is primarily due to the different failure modes. The method proposed in this paper is based on the failure mode of the Bell solution of the flat foundation to derive the failure mode of the rock slope. The failure surfaces are all straight lines, and the transition plastic zone is simplified to a straight line. The other two methods contain logarithmic spiral failure modes, due to which, the directions of gravity and speed continue to increase, resulting in less power increase and sometimes even a decrease. Hu also adopts an asymmetric failure mode for the active failure zone.

In this project, the angle calculated between the passive failure surface and the slope is greater than $\pi/4-\phi/2$, which will reduce the angle between gravity and velocity in the passive zone.

From the comparison of the results in Table 3, it can be concluded that the viewing platform meets the bearing capacity requirements and that the ultimate bearing capacity calculated by this method is a value between Wang’s method [9] and Hu’s method [12], which also proves the rationality of the newly developed method.

### 5. Study of the influencing factors of bearing capacity

As for the ultimate bearing capacity of a flat foundation, it increases with increasing friction angle and cohesion. For the foundation on the rock slope, the ultimate bearing capacity is also related to the slope angle and distance ratio between adjacent slopes. Therefore, this paper discusses the factors affecting ultimate bearing capacity of the slope foundation.

### 5.1. Influence of the ratio of $L$ over $b$

As can be seen from the data in Table 4, under the abovementioned conditions, the ultimate bearing capacity increases when the ratio increases because with ratio increases, the influence of the slope on the ultimate bearing capacity decreases and the ultimate bearing capacity increases.

### Table 4. Influence of the ratio of $L$ over $b$ on ultimate bearing capacity

| $a=L/b$ | Ultimate bearing capacity (kPa) |
|---------|---------------------------------|
| 0       | 241.30                          |
5.2. Impact of dip angle

For the above engineering cases, the remaining parameters remain unchanged in order for the study to explore the impact of dip angle change on the ultimate bearing capacity.

| Dip angle | Ultimate bearing capacity (kPa) |
|-----------|---------------------------------|
| η = 55°   | 278.87                          |
| η = 45°   | 840.61                          |
| η = 35°   | 1488.78                         |
| η = 25°   | 2190.72                         |

It can be seen from the data in the table that the ultimate bearing capacity increases with a decreasing slope angle under the condition that the other conditions remain unchanged. At different slope angles, the results of the ultimate bearing capacity coefficient of the rock foundation adjacent to the slope are shown in Table 6.

| η/°   | N_r  | N_c  | N_q  |
|-------|------|------|------|
| 55    | -0.4896 | 8.4171 | 0.1113 |
| 45    | 2.1640  | 9.1585 | 0.3036 |
| 35    | 5.2381  | 9.9787 | 0.5165 |
| 25    | 8.5375  | 10.9534 | 0.7693 |

It can be seen from the data in the table that when the slope angle gradually increases, the bearing capacity coefficient decreases. When the slope angle is 55°, N_r is a negative number. This is because as the slope angle increases, the passive block area increases, and the angle between the velocity direction of the failure surface of the passive area and the gravity direction may gradually decrease. When the slope angle is large, the angle between the velocity and gravity may be less than 90°. At this time, both the passive block area and the overload do positive work. The self-gravity and overload of the rock have no contribution to the bearing capacity of the foundation. However, on the contrary, it allows the sloping rock to slide down, which shows that the increasing slope angle induces the bearing capacity of the foundation to be significantly reduced.

6. Conclusion

- In this paper, according to the Bell solution to the failure mode of the foundation, the failure mode of the rock foundation adjacent to the slope is constructed, and the upper limit theorem in the limit analysis is used to construct the velocity field allowed by the maneuver. The expression of the formula for the calculation of the bearing capacity of the rock slope near the slope was derived according to the principle of the conservation of energy.
- This method was applied to the bearing capacity analysis of the rock foundation of high and
steep rock slopes, and the bearing capacity of the rock foundation meets the requirements of bearing capacity. The results of this paper were compared with those of other literature methods, which showed that the results were midway between the other results but with the same general conclusions. Furthermore, the rationality and applicability of the analysis and calculation methods proposed in this paper were demonstrated.

- This paper further explored factors, such as the ratio of L over b and the dip angle of the slope, that affect the ultimate bearing capacity and obtained the conclusion that ultimate bearing capacity increases with increasing L over b ratio and decreasing dip angle.

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