Numerical experimental study on influence factors of anchoring force of constant resistance bolt

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ABSTRACT

It is one of the most important problems to judge anchoring force of constant resistant bolt in anchoring support. This involves the properties of rock, binder and bolt material as well as the interaction among them in the process of stress. In this paper, a series of numerical experiments have been carried out by using the software “Real Failure Process Analysis (RFPA)”, which is suitable for the study of complex mechanism of nonlinear and discontinuous media. Four factors that affect the anchoring force of constant resistance bolt are analyzed, the whole process of mechanical failure of anchored rock with constant resistance bolt is monitored, and the failure mode of anchored rock under different conditions is revealed. Draw a few conclusions with reference significance.

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1. Introduction

With the gradual reduction and depletion of shallow resources, the depth of underground mining is increasing. As mining depth increases, the geological environment is becoming more and more complex, sudden engineering disasters and major malignant accidents caused by non-linear large deformation and destruction have occurred frequently (Ma et al., 2016; Meng et al., 2020; Zhu et al., 2020). Because the deformation of roadway surrounding rock allowed by traditional bolt is generally less than 200 mm, the support technology based on traditional bolt can no longer withstand the large nonlinear deformation of deep roadway (Hyett et al., 1996; Kang et al., 2013; Zhu et al., 2019). In recent years, the use of energy-absorbing anchors to prevent and control ground disasters has gradually become a research hotspot (St-Pierre et al., 2009; Cai and Champaigne, 2012). In this regard, scholars from all over the world have carried out some research and formed a series of achievements,
such as Cone bolt (Simser et al., 2002; Cai, 2013; Chen et al., 2014), D-bolt (Li, 2010, 2012; Li and Doucet, 2012), Roofex bolt (Stacey, 2016), Garford bolt (Ansell, 2004, 2005, 2006). The above-mentioned energy-absorbing bolts have a relatively small deformation, the support resistance of these bolts is not constant, so their applicability is relatively poor for underground chambers with large destruction depth and large deformation. Therefore, Professor M.C. He in China has designed and developed a constant resistance and energy-absorbing bolt, which has been successfully applied to soft rock large-deformation tunnels and has achieved good support effects (He et al., 2014).

As an active support form, bolt has been widely used in various projects such as tunnel engineering, slope engineering, etc. The anchoring force of bolt is a key problem in the above engineering design. About this matter, many scholars have done more research and made great progress (Li, 2010, 2012; Cai and Champaigne, 2012; Li and Doucet, 2012; Ma et al., 2016; Stacey, 2016; Meng et al., 2020; Zhu et al., 2020). However, the magnitude of bolt anchoring force is the result of the joint action of rock, bonding material, bolt, etc. in the process of bolt stress, and there are many factors involved (Hyett et al., 1996; Cai, 2013). Among these factors, the current research is more about the influence of bolt length on the anchoring force, but the influence of anchoring force is less studied on some other factors at present. In particular, there is less research on the effect of rock properties on anchoring force. The present research results are limited to the difference in the value range of certain bolt anchoring force in different types of rocks (Simser et al., 2002; Chen et al., 2014), and the specific relationship between the two is not yet clear. The RFPA software developed by Professor C.A. Tang in China provides a new way to analyze the rock fracture process and has attracted the attention of many scholars (Tang, 1997). In this paper, one numerical model is subjected to stress test by changing the value of each influencing factor, the influence degree of each factor on the anchoring force of constant resistance bolt is evaluated. The content of this article has further improved the supporting mechanism of the constant resistance bolt.

2. Introduction to the basic principles of RFPA and constant resistance bolt

2.1. Stress–strain relationship of RFPA

RFPA is based on the meso statistic damage theory and adopts a constitutive relationship with residual strength. The elastic damage constitutive relation of element is shown in Figure 1. When the stress or strain states in the element meet a given threshold, the element begins to accumulate damage. The cumulative damage process of element elastic modulus can be given by

$$E = (1-D)E_0 \quad (1)$$

where $D$ is the damage variable; $E$ and $E_0$ are elastic modulus before and after the element damage, respectively.
The equation of the damage variable under uniaxial tension can be expressed as follows:

\[
D = \begin{cases} 
0 & \varepsilon < \varepsilon_{t0} \\
1 - \frac{\lambda \varepsilon_{t0}}{\varepsilon} & \varepsilon_{t0} < \varepsilon < \varepsilon_{tu} \\
1 & \varepsilon \geq \varepsilon_{tu}
\end{cases}
\]  

(2)

where \(\lambda\) is element residual tensile strength coefficient, i.e., \(\sigma_{tr} = \lambda \sigma_{t0}\); \(\varepsilon_{t0}\) is the tensile strain corresponding to the elastic limit state of element, and it is also the initial damage threshold of element; \(\varepsilon_{tu}\) is the ultimate tensile strain of element, which can be described by the formula \(\varepsilon_{tu} = \eta \varepsilon_{t0}\), and \(\eta\) is the ultimate strain coefficient.

Under uniaxial compression, the damage variable \(D\) is expressed as follows:

\[
D = \begin{cases} 
0 & \varepsilon < \varepsilon_{c0} \\
1 - \frac{\lambda \varepsilon_{c0}}{\varepsilon} & \varepsilon \geq \varepsilon_{c0}
\end{cases}
\]  

(3)

where \(\varepsilon_{c0}\) is ultimate compressive strain of element; \(\lambda\) is residual compressive strength coefficient of element.

It is worth noting that when the residual strength coefficient is 0.1, the stress-strain relationship is a typical elastic brittle constitutive relationship. When the residual strength coefficient is 1, the stress-strain relationship is a typical elastic-plastic constitutive relationship.

2.2. Element assignment of RFPA

RFPA software has been widely used to simulate the process of rock failure and engineering stability under various conditions (Tang and Kaiser, 1998; Tang et al., 2000; Li and Tang, 2015; Chen et al., 2017). RFPA software assumes that the mechanical properties of discrete meso elements obey Weibull distribution, thus establishing the relationship between the meso mechanical properties and the macro mechanical properties.
properties of materials. The introduction of Weibull statistical distribution function is described as follows (Weibull, 1951):

\[ f(u) = \frac{m}{u_0} \left( \frac{u}{u_0} \right)^{m-1} \exp \left[ -\left( \frac{u}{u_0} \right)^m \right] \] (4)

where \( u \) represents element mechanical properties, such as elastic modulus, compressive strength, etc.; \( u_0 \) represents average value of element mechanical properties; \( f(u) \) is the statistical distribution density of element mechanical properties; \( m \) is the property parameter of distribution function, whose physical meaning reflects the uniformity of material, it is defined as the uniformity coefficient of materials.

Equation (4) reflects the inhomogeneous distribution of meso mechanical properties of a material (such as rock). The larger the homogeneity coefficient \( m \) is, the more uniform the mechanical properties of material are, and the mechanical properties of element are concentrated in a narrow range. On the contrary, the smaller the homogeneity coefficient \( m \) is, the more inhomogeneous the mechanical properties of material are, then the mechanical properties of element are distributed in a wide range, as shown in Figure 2.

The distribution form of elastic modulus of different homogeneity coefficient \( m \) is shown in Figure 3. If the lower the coefficient of homogeneity is, the greater the difference between the elastic modulus values of elements (as shown in Figure 3a), the more discrete the elastic model of whole model is. On the contrary, the distribution of elastic model shows the opposite law. When \( m \) is equal to 50, the elastic modulus of each element is the same, and the numerical model can be regarded as a homogeneous body, as shown in Figure 3d.

2.3. Constant resistance bolt

The constant resistance bolt is mainly composed of a nut, tray, constant resistance sleeve, and rod body, as shown in Figure 4. Among them, the constant resistance sleeve and the rod body constitute the constant resistance device of constant
resistance bolt, and the nut and tray are located at the tail of constant resistance sleeve (He et al., 2012, 2014). When surrounding rock is subjected to a static load or dynamic disturbance, the constant resistance bolt accounts for the magnitude of stress, and then the constant resistance device can resist the deformation and destruction of rock mass through frictional slip. In addition, a constant resistance bolt can maintain a constant supporting force (He and Guo, 2014).

3. Constant resistance bolt uniaxial tension mechanical properties

3.1. Constant resistance bolt numerical model

In order to study the tensile mechanical properties of constant resistance bolts, the numerical model of constant resistance bolt is established using RFPA software, and
A uniaxial tensile test is carried out. As shown in Figure 5, the numerical model size of constant resistance bolt is 750 mm × 34 mm, and the number of discretized elements is 750 × 34. Using displacement loading method, and displacement increment Δs is 0.002 mm. The physical and mechanical parameters of constant resistance bolt are shown in Table 1. Because the constant resistance bolt deforms considerably to maintain a certain working stress and has a time-dependent mechanical behaviour, the constitutive relation is simplified as an ideal elastic-plastic model (see Figure 6). The residual strength coefficient is set to 1, and the constitutive relationship of the material can be set to elastic-plastic in RFPA.

### 3.2. Analysis of the tensile results

A series of tensile properties, such as tensile strength and elongation, of the material under tensile loading can be measured in tensile tests. A tensile test is an important method for determining whether materials meet required standards. The simulated tensile test of constant resistance bolt shows that the plastic strain is 12 times the magnitude of the elastic strain after tensile fracturing of constant resistance bolt, the stress magnitude stably fluctuates in the range of 345.07—347.26 MPa during plastic deformation, without sudden increases or decreases (see Figure 7). Therefore, this...
bolt has a very good constant resistance performance. When the strain is $1.944 \times 10^{-3}$, the stress begins to drop, and its value is 137.59 MPa. At this time, the rod body cracks (see Figure 8a), and the bolt starts to enter failure stage. When the cracks continue to expand, the stress value continues to decline, which is almost a "cliff" decline. When the strain value is $1.952 \times 10^{-3}$, the fracture surface is completely formed (see Figure 8b), the stress value is 0, and this bolt loses its bearing capacity. The stress-strain relationship of constant resistance bolt obtained by numerical simulation is very similar to the experimental results of Ansell (2005), indicating the accuracy and feasibility of this numerical test method.

It can also be seen from Figure 7 that when the stress value of constant resistance bolt reaches 273.97 MPa, the rod body generates acoustic emission, and the strain at this time is $0.140 \times 10^{-3}$. When the stress value of bolt reaches 345.07 MPa, that is to say, the bolt has just entered the stage of plastic deformation, the number of acoustic emissions reaches the maximum value, its number is 3629, and the strain at this time is $0.172 \times 10^{-3}$. In the subsequent loading process, the number of acoustic emissions continues to decrease. When the strain value of bolt is $0.364 \times 10^{-3}$, the number of acoustic emissions is only 10. After that, the acoustic emission phenomenon only appears occasionally, the number of which is only single digits. It can be seen that, compared with the entire deformation process of rod body, the acoustic emission number of constant resistance bolt has completed the process from appearance, peak value, and then to single digits in a shorter time. That is, the acoustic emission number mainly occurs in the late stage of elastic deformation and the early stage of plastic deformation.

4. Influencing factors of anchoring force

4.1. Numerical model

It is an important content to study the size of anchoring force and the law of crack growth in anchored rock. Therefore, a numerical model is established as shown in
Figure 8. Constant resistance bolt failure graph. (a) The stress value is $1.944 \times 10^{-3}$. (b) The stress value $1.952 \times 10^{-3}$.

Figure 9. Numerical model of constant resistance bolt drawing test.

Figure 9. The overall size of the model is $2000 \times 1500$ mm, and the number of divided units is $400 \times 300$. The left, right and bottom sides of this numerical model are fixed. The tensile load is applied to the upper end face of constant resistance bolt, and the displacement increment is 0.002 mm. The size of constant resistance bolt is
750 × 34 mm, which is located in the transverse middle of this model. The binder is located around constant resistance bolt, and binder thickness is the size of one unit.

The influence of binder elastic modulus, rock elastic modulus and rock compressive strength on the anchoring force of constant resistant bolt has been mainly considered. The numerical calculation parameters of binder and rock are shown in Table 2.

Table 2. Numerical test scheme.

| Rock              | Binder          |
|-------------------|-----------------|
| Elastic modulus (GPa) | Compressive strength (MPa) | Elastic modulus (GPa) | Compressive strength (MPa) |
| 20                | 100             | 13.24            | 90                        |
| 40                | 50              | 13.24            | 90                        |
| 60                | 100             | 13.24            | 90                        |
| 80                | 150             | 13.24            | 90                        |
| 100               | 200             | 13.24            | 90                        |
| 150               | 13.24           | 90                |
| 200               | 6.62            | 90                |
| 26.48             | 13.24           |
| 39.72             | 200             |
| 100               | 13.24           | 70                |
| 90                | 110             |
| 110               | 130             |

4.2. Failure modes in anchored rock

According to the pull-out numerical test of constant resistance bolt, there are three failure modes in anchored rock. The failure mode of anchored rock with different numerical test schemes is shown in the appendix.

4.2.1. Failure mode of progressive crack growth in rock (abbreviated as class I failure mode)

As shown in Figure 10, this failure mode is characterized by no significant damage to the binder. In the initial stage of loading, cracks first appear in the stress concentration area at the end of the rod body. When the loading continues, these cracks develop downward along the rod body, and at the same time, some original cracks begin to expand laterally. Under the continued action of tensile force, the number of broken elements at the bottom of bolt increases, forming through cracks. As the load continues to increase, the crack at the bottom of bolt extends perpendicular to the rod axial direction until this numerical model loses its load-bearing capacity.

4.2.2. Failure mode of binder and rock simultaneous failure (abbreviated as class II failure mode)

The characteristic of this failure mode is that the binder on the right side of bolt head first breaks down at the beginning of loading (see Figure 11). The binder on the left side of bolt head also breaks when the loading is continued, and at the same
Figure 10. Failure process of progressive crack propagation in rock (The left column is the elastic modulus diagram, the right column is the acoustic emission diagram).
Figure 11. Failure mode of binder and rock simultaneous failure (The left column is the elastic modulus diagram, the right column is the acoustic emission diagram).
Figure 12. Failure process of binder complete failure (The left column is the elastic modulus diagram, the right column is the acoustic emission diagram).
time, a through crack has been formed in the middle and upper part of bolt. When the binder on the right and bottom of rod body is completely destroyed, the rock at the bottom of bolt also forms through cracks under the continuous action of load, and these cracks propagate in the direction perpendicular to the axial direction of bolt until the specimen loses its bearing capacity. When the sample has no bearing capacity, the binder on the right side of bolt has not been completely broken.

4.3.3. Failure mode of binder complete failure (abbreviated as class III failure mode)

As can be seen from Figure 12, the characteristic of this kind of failure mode is that the binder on both sides of rod body is gradually destroyed from top to bottom, and the constant resistance bolt is obviously pulled out.

4.3. Influence of consideration factors on the anchoring force

Figures 13–16 are the relationship curves of rock strength-anchoring force, rock elastic modulus-anchoring force, binder strength-anchoring force and binder elastic modulus-anchoring force, respectively. The anchoring force of anchored rock with different numerical test schemes is shown in the appendix. When the strength of rock is from 50 MPa to 100 MPa, the increase of anchoring force is 67 MPa (shown in Figure 13). When the rock strength is from 100 MPa to 200 MPa, although the anchoring force increases with the increase of rock strength, the curve is relatively flat, and the increase of anchoring force is small. As elastic modulus of rock increases, the anchoring force generally shows a decreasing trend (shown in Figure 14). However, the difference between the maximum value and the minimum value of anchoring force is only 11 MPa, indicating that the rock elastic modulus has no significant effect on anchoring force of bolt. It can be seen from Figure 15 that the anchoring force of rock increases in a straight line with the increase of binder strength, and the difference between the maximum value and the minimum value of
The anchoring force is 107 MPa. The anchoring force of rock generally decreases linearly with the increase of binder elastic modulus, and the maximum and minimum anchoring forces differ by 138 MPa (shown in Figure 16). From the above analysis, we can see that the above four factors, except that rock elastic modulus has no significant effect on the anchoring force, the other three factors have significant effect on the anchoring force, and the degree of influence from the largest to the smallest is binder elastic modulus, binder strength, and rock strength.

4.4. Acoustic emission analysis of anchored rock

When the material is subjected to external load, the strain energy stored in the material is quickly released to produce elastic waves, which is called acoustic emission. Rock is generally a non-uniform material, and there are many defects inside. Under
the action of external force, new cracks will be generated inside the rock, and the acoustic emission phenomenon will be produced during the crack initiation and propagation. Using acoustic emission technology to study the initiation and propagation of micro-cracks in rocks is helpful for comprehensively and truly understanding the process of rock failure and deformation law.

Figure 17 is the stress-loading step diagram and acoustic emission-loading step diagram obtained from numerical simulation of representative samples of three failure modes. As can be seen from Figure 17, no matter which type of failure mode, the maximum number of acoustic emissions occurs in the stage of stress drop. There is still a certain amount of acoustic emission after acoustic emission of Class I and II failure mode reaches the maximum value. It shows that there are still local failures in these two kinds of numerical models, and the cracks extend to the interior of rock. These two types of failure modes also have a certain anchoring force at the later stage of loading. This anchoring force is transformed into a force that causes cracks to propagate in rock, and the damage is relatively concentrated, that is, only one main crack continues to expand. It can be seen from Table 3 that the maximum number of acoustic emissions and the cumulative number of acoustic emissions for Class I failure mode are the largest. This is due to the overall failure of the rock mass for Class I failure mode, so the numerical model has the largest number of unit failures, that is, the largest number of acoustic emissions. The maximum number of acoustic emissions and the cumulative number of acoustic emissions for Class II failure mode are ranked second. It can also be seen from Figure 17 (a) and (b) that these two types of failure are typical brittle failure, and there is no obvious sign before numerical model failure.

When the number of acoustic emission has reached the maximum value, the number of acoustic emission has no value and the stress is reduced to 0 for Class III failure mode. This is because the constant resistance bolt is pulled out due to the destruction of binder, which makes the anchored system no longer bear tensile force,
Figure 17. Change process of acoustic emission quantity in different failure modes.
and the numerical model no longer has any destructive elements, which also makes the maximum number of acoustic emissions and the cumulative number of acoustic emissions in this destruction mode is the minimum (see Table 3). It can also be seen from Figure 17(c) that once the numerical model has been damaged, the anchoring force is reduced to 0, so it is necessary to guard against such damage and take measures to avoid it as much as possible.

### 5. Conclusion

In this paper, RFPA software has been used to carry out a series of numerical experiments and combine the experimental data for analysis. The following conclusions are obtained:

1. According to the uniaxial tensile numerical experiment of constant resistance bolt, the maximum plastic strain is 12 times of the maximum value of elastic strain after tensile fracture, and the stress value fluctuates in the range of 345.07—347.26 MPa during plastic deformation, the fluctuation is stable, which shows this bolt has good constant resistance characteristics. In the whole process of tensile deformation, acoustic emission mainly occurs in the late stage of elastic deformation and the early stage of plastic deformation.

2. Three failure modes of anchored rock with constant resistance bolt are obtained, which are failure mode of progressive crack growth in rock, failure mode of progressive crack growth in rock failure mode of binder complete failure, respectively.

3. Among the four factors examined in the numerical experiment, three factors have significant effect on the anchoring force of constant resistance bolt, and the degree of influence is determined by the binder elastic modulus, the binder strength and the rock strength. There is one factor that the rock elastic modulus has no significant effect on the anchoring force of the bolt.

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**Table 3. Acoustic emission quantity in different failure modes.**

| Failure mode     | Maximum number of acoustic emission (time) | Acoustic emission cumulative value (time) |
|------------------|-------------------------------------------|------------------------------------------|
| Class I failure mode | 509                                       | 3015                                     |
| Class II failure mode | 324                                       | 1868                                     |
| Class III failure mode | 131                                       | 894                                      |
References

Ansell A. 2004. In situ testing of young shotcrete subjected to vibrations from blasting. Tunn Underground Space Technol. 19(6):587–596.
Ansell A. 2005. Laboratory testing of a new type of energy absorbing rock bolt. Tunn Underground Space Technol. 20(4):291–300.
Ansell A. 2006. Dynamic testing of steel for a new type of energy absorbing rock bolt. J Constr Steel Res. 62(5):501–512.
Cai M. 2013. Principles of rock support in burst-prone ground. Tunnelling & underground space technology incorporating trenchless. Technol Res. 36(June):46–56.
Cai M, Champaigne D. 2012. Influence of bolt-grout bonding on MCB cone bolt performance. Int J Rock Mech Min Sci. 49:165–175.
Chen L, Sheng G, Chen G. 2014. Investigation of impact dynamics of roof bolting with passive friction control. Int J Rock Mech Min Sci. 70(9):559–568.
Chen X, Yu J, Tang C, Li H, Wang S. 2017. Experimental and numerical investigation of permeability evolution with damage of sandstone under triaxial compression. Rock Mech Rock Eng. 50(6):1529–1549.
He MC, Guo ZB. 2014. Mechanical property and engineering application of anchor bolt with constant resistance and large deformation. Chinese Journal of Rock Mechanics and Engineering. 33(7):1297–1308. in Chinese)
He M, Gong W, Wang J, Qi P, Tao Z, Du S, Peng Y. 2014. Development of a novel energy-absorbing bolt with extraordinarily large elongation and constant resistance. Int J Rock Mech Min Sci. 67(1):29–42.
He M, Xia H, Jia X, Gong W, Zhao F, Liang K. 2012. Studies on classification, criteria, and control of rockbursts. J Rock Mech Geotech Eng. 4(2):97–192.
Hyett AJ, Moosavi M, Bawden WF. 1996. Load distribution along fully grouted bolts with emphasis on cable bolt reinforcement. Int J Numer Anal Methods Geomech. 20(7):517–544.
Kang H, Wu Y, Gao F, Lin J, Jiang P. 2013. Fracture characteristics in rock bolts in underground coal mine roadways. Int J Rock Mech Min Sci. 62(5):105–112.
Li CC. 2010. A new energy-absorbing bolt for rock support in high stress rock masses. Int J Rock Mech Min Sci. 47(3):396–404.
Li CC. 2012. Performance of D-bolts Under Static Loading. Rock Mech Rock Eng. 45(2):183–192.
Li G, Tang CA. 2015. A statistical meso-damage mechanical method for modeling transscale progressive failure process of rock. Int J Rock Mech Min Sci. 74:133–150.
Li CC, Doucet C. 2012. Doucet C. performance of D-bolts under dynamic loading. Rock Mech Rock Eng. 45(2):193–204.
Ma S, Zhao Z, Nie W, Gui Y. 2016. A numerical model of fully grouted bolts considering the tri-linear shear bond-slip model. Tunn Underground Space Technol. 54:73–80.
Meng Q, Wang H, Cai M, Xu W, Zhuang X, Rabczuk T. 2020. Three-dimensional mesoscale computational modeling of soil-rock mixtures with concave particles. Eng Geol. 277:105802–105814.
Simser B, Joughin WC, Ortlepp WD. 2002. The performance of Brunswick mine’s rockburst support system during a severe seismic episode. J South African Instit Min Metallur. 102(4):217–223.
Stacey TR. 2016. Addressing the consequences of dynamic rock failure in underground excavations. Rock Mech Rock Eng. 49(10):4091–4101.
St-Pierre L, Hassani FP, Radziszewski PH, Ouellet J. 2009. Development of a dynamic model for a cone bolt. Int J Rock Mech Min Sci. 46(1):107–114.
Tang CA. 1997. Numerical simulation of progressive rock failure and associated seismicity. Int J Rock Mech Min Sci. 34(2):249–261.
Tang CA, Kaiser PK. 1998. Numerical simulation of cumulative damage and seismic energy release during brittle rock failure – Part I: fundamentals. Int J Rock Mech Min Sci. 35(2):113–121.
Tang CA, Liu H, Lee PKK, Tsui Y, Tham LG. 2000. Numerical studies of the influence of microstructure on rock failure in uniaxial compression - Part I: effect of heterogeneity. Int J Rock Mech Min Sci. 37(4):555–569.

Weibull W. 1951. A statistical distribution function of wide applicability. Trans ASME, J Appl Mech. 18:293–297.

Zhu C, He M, Karakus M, Cui X, Tao Z. 2020. Investigating toppling failure mechanism of anti-dip layered slope due to excavation by physical modelling. Rock Mech Rock Eng. 53(11):5029–5050.

Zhu C, Xu X, Liu W, Xiong F, Lin Y, Cao C, Liu X. 2019. Softening damage analysis of gypsum rock with immersion time based on laboratory experiment. IEEE Access. 7:125575–125585.

**Appendix**

Anchoring force and anchored rock failure mode of different numerical test schemes

| Rock Binder | Anchoring force (MPa) | Anchored rock failure mode |
|-------------|-----------------------|-----------------------------|
| Rock | Elastic modulus (GPa) | Strength (MPa) | Binder | Elastic modulus (GPa) | Strength (MPa) | | | | |
| | | 13.24 | 90 | | 285 | II |
| 20 | 100 | 13.24 | 90 | 285 | II |
| 40 | 50 | 13.24 | 90 | 205 | I |
| 100 | 6.62 | 90 | 328 | II |
| 40 | 100 | 13.24 | 70 | 233 | III |
| 90 | 272 | II |
| 110 | 320 | II |
| 130 | 340 | II |