Research Article

Research on Cumulative Damage Characteristics of Rock Anchor Beam Concrete Supporting Structure by Blasting Vibration of Underground Powerhouse

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Aiming at the dynamic disturbance problem of the upper rock anchor beam concrete support structure caused by the blasting excavation of the lower rock mass of the underground powerhouse of the hydropower station, the dynamic finite element software was used to numerically simulate the cumulative damage characteristics of the rock anchor beam concrete structure under multiple blasting, which aims to study the effect of blasting times and blast center distance on the cumulative damage characteristics of rock anchor beam concrete structures in two directions at different ages. The results show that 7 days after the completion of the concrete pouring of rock anchor beam, the damage and destruction effects are produced under the single blasting action. 28 days after the completion of the concrete pouring, there is basically no damage and destruction under the action of five times of blasting. 14 days after the completion of the concrete pouring, the growth process of the cumulative damage effect of the rock anchor beam concrete structure under the disturbance of the blasting excavation of the underground powerhouse shows nonlinear increasing characteristics with the increase of blasting times and nonlinear decreasing characteristics with the increase of blasting center distance. The cumulative damage of rock anchor beam along and perpendicular to the axis of powerhouse conforms to the relationship of cubic polynomials with the blasting times, which also conform to the relationship of quartic polynomial and quadratic polynomial, respectively, with the blast center distance.

1. Introduction

In the layout of underground powerhouses of modern hydropower stations, especially pumped storage power stations, rock anchor crane beam structures are mostly used [1–3]. Because it undertakes the operation function of the crane in the powerhouse, it belongs to the core control object in structural design and construction. As the pouring of rock anchor beam is often carried out at the same time with the layered excavation of the powerhouse [4], if the blasting construction is not properly controlled, the concrete cracking of rock anchor beam caused by blasting vibration, and the loss of bond force between concrete and rock will

have adverse effects on the later safe operation of rock anchor beam [5, 6].

Frequent and repeated blasting will cause cumulative damage to surrounding rock and concrete supporting structure, reducing the physical and mechanical properties and durability of the structure. When the damage accumulates to a certain extent, it may lead to local or even overall instability of the structure, seriously affecting the efficiency and economy of the engineering construction [7–10]. Ramulu et al. [11] pointed out that repeated blasting loading during excavation will accelerate the development of damage area of surrounding rock. The research shows that the process of the cumulative damage leading to the

macrofailure of engineering rock mass is caused by repeated blasting disturbance. It is of great significance to carry out the experimental research on the cumulative damage degree of concrete structure blasting and the attenuation law of blasting vibration propagation.

In the aspect of damage research of blasting load on concrete supporting structure, Duan et al. [12] used wavelet packet analysis technology to calculate the energy value in different frequency band of vibration signal under different blasting conditions (blasting times and blasting center distance) and discussed the influence rule of different blasting conditions on energy signal distribution. Liu et al. [13] obtained the change characteristics of the energy distribution law of each frequency band of the acoustic signal after the wavelet decomposition with the concrete damage degree. Liang et al. [14] believed that the overpressure of blast shock wave is the main factor of concrete damage, and the explosion gas has an important effect on the propagation of existing cracks and the generation of new cracks. Miao [15] found that when the radial uncoupling charge coefficient of the blasthole is greater than 3, the surrounding rock damage decreases rapidly under the effect of blasting load.

In regard to the propagation law of blasting vibration in concrete structure, Zhang [16] proposed for the first time that the particle vibration speed of 7 cm/s was taken as the blasting safety control standard of 28 days after the completion of concrete pouring of rock anchor beam. Yao et al. [17] believed that as the height difference increased, the particle vibration velocity of the high side wall gradually decreased and the maximum tensile stress appeared at the top or bottom of the rock anchor beam bonding surface, which has a good correlation with the horizontal particle vibration velocity. The maximum tensile stress and particle vibration velocity at the arch foot of the crown are relatively large, while the maximum tensile stress and particle vibration velocity above the arch foot decrease rapidly [18].

2. Theoretical Analysis of Blasting Load

2.1. Constitutive Model. The RHT constitutive model is selected, and the equation of state in the constitutive model RHT describes the relationship among hydrostatic pressure, material density, and internal energy. The equation of state after full compaction is as follows:

$$P = A_1\mu + A_2\mu^2 + A_3\mu^3 + (B_0 + B_1\mu)\rho_0 e, \quad \mu > 0, \quad (1)$$

$$P = T_1\mu + T_2\mu^2 + B_0\rho_0 e, \quad \mu < 0, \quad (2)$$

where $\mu = \rho/\rho_0 - 1$. When $\mu > 0$, the material is compressed, and when $\mu < 0$, the material is stretched. ρ_0 is the density when the material is not stretched or compressed, ρ is the real-time material density, and e is the initial internal energy of the material. A_1 , A_2 , and A_3 are Hugoniot polynomial coefficients. B_0 , B_1 , T_1 , and T_2 are the state equation parameters.

When there are pores in the material, the effect of porosity α should be taken into account. If the density of

porous material is ρ_p , the density of material is $\rho_p\alpha$, and the corresponding equation of state is as follows:

$$P = f(\rho\alpha, e), \quad (3)$$

$$\alpha = 1 + (\alpha_0 - 1) \left[\frac{P_{\text{lock}} - P}{P_{\text{lock}} - P_{\text{crush}}} \right]^n, \quad (4)$$

$$\alpha = \frac{\rho_0}{\rho_p}, \quad (5)$$

where P_{crush} is the pressure when the pores in the material begin to be compressed ($\alpha = \alpha_0$), P_{lock} is the pressure when the pores in the material completely disappear ($\alpha = 1$), and n is the compression index.

The RHT constitutive model defines the elastic limit surface, failure surface, and residual strength surface to describe the strength change law of the material. They described the initial yield strength, failure strength, and residual strength of the material. Since the material is assumed to be isotropic, the elastic limit surface and failure surface under the triaxial load can be represented by a curved surface in a three-dimensional principal stress space, and it is triple symmetrical in the off-plane. The relationship among the elastic limit surface, residual strength surface, and failure surface is shown in Figure 1.

In the process of plastic deformation in the elastic stage and the linear strengthening stage, there is no damage in the material. Only when the stress state of the material reaches the failure surface, that is, when the effective force exceeds the failure strength, can the material begin to enter the damage softening stage, resulting in damage and accumulation.

(a) Definition and evolution of damage variable

The definition of the damage variable D_0 is

$$0 \leq D_0 = \sum \frac{\Delta \varepsilon_p}{\varepsilon_p^{\text{failure}}} \leq 1, \quad (6)$$

$$\varepsilon_p^{\text{failure}} = D_1 (P^* - HTL^*)^{D_2} \geq \varepsilon_p^m, \quad (7)$$

where $\Delta \varepsilon_p$ is the equivalent plastic strain increment, D_1 and D_2 are the parameters of the material, and ε_p^m is the minimum equivalent plastic strain when the material is damaged.

(b) Residual equivalent stress intensity

After the material reaches the failure surface and accumulates a certain amount of damage, due to the effect of confining pressure, there is still some frictional bite force between the broken blocks, so that the material still has a certain shear resistance. For this reason, the residual stress surface is introduced into RHT constitutive, and its equivalent stress intensity is

$$\sigma_{\text{residual}} = A_f \times (P^*)^{n_f}. \quad (8)$$

Among them, A_f is the residual stress intensity parameter, and n_f is the residual stress intensity index.

FIGURE 1: Schematic diagram of elastic limit surface, failure surface, and residual strength surface of the RHT model.

(c) Equivalent stress intensity at the damage stage after failure

After the stress state of the material reaches the failure surface, with the accumulation of material damage, the deviator stress in the material decreases gradually. Therefore, the equivalent stress intensity σ_{damage} at the damage stage can be determined according to the interpolation value between the equivalent stress intensity at the failure surface and the residual equivalent stress intensity:

$$\sigma_{\text{damage}} = (1 - D_0)\sigma_{\text{failure}} + D_0\sigma_{\text{residual}}. \quad (9)$$

2.2. Explosion Load. In this study, the simplified triangle explosion load curve is adopted for dynamic loading [19] as shown in Figure 2.

According to the C-J theory of the detonation wave of condensed explosive, for the caving hole, the charge uncoupling coefficient is small, and the pressure peak P_b of the detonation gas acting on the wall of the blast hole is

$$P_b = \frac{\rho_e(V)^2}{2(\gamma + 1)} \left(\frac{d_c}{d_b} \right)^{2\gamma}. \quad (10)$$

For buffer holes and smooth blasting holes, the uncoupling coefficient is large, and the expansion of gas needs to go through two stages: $P \geq P_k$ and $P < P_k$. P_k is the critical pressure of explosion gas, taken as 200 MPa. In the calculation, the isentropic index is treated as a piecewise constant. When $P \geq P_k$, $\gamma = 3.0$. When $P < P_k$, $\gamma = \nu = 4/3$. Then, the above formula becomes

FIGURE 2: Time history curve of explosion pressure of equivalent triangle.

$$P_b = \left[\frac{\rho_e(V)^2}{2(\gamma + 1)} \right]^{\frac{\nu}{\gamma}} P_k^{(\gamma - \nu)/\gamma} \left(\frac{d_c}{d_b} \right)^{2\gamma}, \quad (11)$$

where ρ_e and V are the explosive density and detonation velocity. d_c is the charge diameter, and d_b is the blast hole diameter. The load boosting action time is $t_r = 0.2$ ms, and the positive pressure action time is $t_d = 1.2$ ms. The explosive density is $\rho_e = 1.0$ g/cm³, and detonation wave velocity is $V = 3600$ m/s.

The schematic diagram of the design hole layout of pilot tunnel blasting in the first floor of the underground powerhouse of Qingyuan pumped storage power station is shown in Figure 3. The interval time of blasting load in each section is controlled by the delay initiation time of detonator, as shown in Table 1.

The blasting excavation of underground powerhouse adopts blast hole with the diameter of $\phi 42$ mm and 2# rock emulsion explosive with the diameter of $\phi 25$ mm. After

FIGURE 5: Layered excavation of underground powerhouse (unit: cm).

used for numerical calculation [22]. Considering the influence of symmetry and boundary conditions, 1/4 of the actual size is selected to establish the numerical model [23–26]. The size of this modeling is 33.25 m $\times$ 20 m $\times$ 67.2 m (length $\times$ width $\times$ height). The model boundary and the finite unit model are shown in Figure 6. The RHT constitutive model is embedded in the numerical calculation model, and the relevant rock and concrete material parameters are obtained based on experimental value of the field test as shown in Table 2.

Considering the full-section blasting excavation process of the first 5 sections of the third floor of the powerhouse, the cumulative damage effects of surrounding rock and rock anchor beam concrete under 5 blasting loads are calculated, and the damage condition of rock anchor beam caused by blasting times and blasting center distance is calculated by regression. Figure 7 shows the schematic diagram of the monitoring points design of rock anchor beam concrete.

3.3. Analysis of Cumulative Damage of Rock Anchor Beam Concrete

3.3.1. Working Condition 1. The cumulative damage change process of rock anchor beam concrete obtained by numerical simulation calculation is shown in Figure 8. It can be seen from the cumulative damage nephogram of the blasting that the full-section blasting excavation of the third floor of the powerhouse will be started 7 days after the completion of the concrete pouring of rock anchor beam.

The cumulative damage of the solid unit corresponding to the rock anchor beam under blasting is shown in Figures 9 and 10. The lower part of the rock anchor beam is damaged in a large range under the first blasting action, and under the second blasting action, the rock anchor beam has a continuous and extensive damage. With the increase of blasting

times, the cumulative damage of the rock anchor beam gradually expands, and by the fifth blasting, most of rock anchor beam units are damaged. The monitoring points of the solid unit corresponding to the parts of the rock anchor beam are extracted for analysis.

3.3.2. Working Condition 2. The process of cumulative damage change of rock anchor beam concrete is obtained through numerical simulation calculation. It can be seen from Figure 11 that when the full-section blasting excavation of the third floor of the powerhouse is carried out 28 days after the completion of the concrete pouring of the rock anchor beam, the entire rock anchor beam unit does not produce large damage and destruction under the action of 5 times of repeated blasting, and only a few units produced local damage under the action of reflected stress waves. It shows that at this time, the concrete has reached sufficient strength, and the blasting excavation will not have a significant impact on it.

By analyzing the monitoring points of the solid units corresponding to the rock anchor beams (Figures 12 and 13), it can be seen that only the P_1 and Q_1 of the extracted relevant monitoring points have been damaged, while the damage values of other units along or perpendicular to the axis of the powerhouse all less than 1. With the increase of blasting times, the cumulative damage does not increase gradually.

3.3.3. Working Condition 3. The process of cumulative damage change of rock anchor beam concrete is obtained through numerical simulation calculation. It can be seen from Figure 14 that when the full-section blasting excavation of the third floor of the powerhouse is carried out 14 days after the completion of the concrete pouring of the rock anchor beam, some units in the lower area of the rock anchor beam are damaged. With the increase of the blast times, the cumulative damage of the entire rock anchor beam is centered on the location of the blast source and gradually expands along and perpendicular to the axis of the powerhouse, and the cumulative damage gradually increases. As shown in Figures 15 and 16, the exposed surface of the upper part of the third floor of the powerhouse is closer to the rock anchor beam, and the length of the rock anchor beam along the axis of the powerhouse is much greater than the height perpendicular to the axis of the powerhouse. Therefore, under the action of multiple blasts, the cumulative damage of the rock anchor beam perpendicular to the axis of the powerhouse is greater than that along the axis of the powerhouse.

4. The Relationship between Cumulative Damage and Blasting Times and Blast Center Distance

4.1. Relationship between Cumulative Damage and Blasting Times

4.1.1. Cumulative Damage Analysis of Concrete along the Axis of Powerhouse. This time, the blasting excavation of the third floor of the underground powerhouse 14 days after the

FIGURE 6: Finite unit model diagram. (a) Perspective view of finite unit model network. (b) Finite unit model.

TABLE 2: Parameters of the RHT concrete model.

Parameter	Value	Parameter	Value
Material density ρ_0	1780 kg/m ³	Failure surface parameters A	1.6
Initial porosity α_0	1.1613	Failure surface index n	0.55
Pressure at the beginning of pore crushing p_{el}	18.5 MPa	Tension compression meridian ratio parameter Q_0	0.68
Pressure at pore is fully compacted p_{comp}	0.556 GPa	Rhode angle correlation coefficient B	0.08
Porosity index N	2.0	Shrinkage strain rate index c	0.032
Hugoniot coefficient A_1	39.75 GPa	Tensile strain rate index t	0.036
State equation parameters B_1	1.306	Residual stress intensity A_f	2.42
Shear modulus G	12.0 GPa	Initial damage parameters D_1	0.052

FIGURE 7: Schematic diagram of the monitoring point design of rock anchor beam concrete.

completion of the concrete pouring of the rock anchor beam was taken as the analysis object. Deducing the damage value of the monitoring point of the cumulative damage of the rock anchor beam along the axis of the powerhouse, the relationship between the damage value and blasting times can be obtained. As shown in Figure 17, the monitoring points along the axis of powerhouse are R_1, R_2, R_3, R_4, R_5 , and R_6 , and their distances from the tunnel face are 0.2 m,

1.2 m, 2.2 m, 3.2 m, 4.2 m, and 5.2 m, respectively. The curve relationship between the damage value of each unit and blasting times is shown in Figure 18.

It can be seen from Figure 18 that the damage of rock mass shows a nonlinear increasing characteristic with the increase of the blasting times. According to the nonlinear characteristics of blasting damage evolution of rock anchor beam, a polynomial relationship is used for curve regression analysis, and the polynomial is obtained as follows:

$$y = A_0 + A_1x + A_2x^2 + A_3x^3, \quad (12)$$

where y is the blasting damage value of rock mass. x is the blasting times. A_0, A_1, A_2 , and A_3 are undetermined constants.

Through the regression calculation and analysis, the polynomial regression equation between unit blasting damage and blasting times is shown in Table 3, and the confidence levels are all greater than 0.98. It is indicated that the relationship between blasting damage of rock anchor beam and blasting times conforms to the cubic polynomial

FIGURE 8: Cumulative damage nephogram of surrounding rock and rock anchor beam concrete under blasting. (a) 3.1 ms, (b) 6.1 ms, (c) 9.1 ms, (d) 12.1 ms, and (e) 15.0 ms.

FIGURE 9: The curve of the relationship between cumulative damage of rock anchor beam monitoring points along the axis of powerhouse and blasting times.

FIGURE 10: The curve of the relationship between cumulative damage of rock anchor beam monitoring points perpendicular to the axis of powerhouse and blasting times.

FIGURE 11: Cumulative damage nephogram of surrounding rock and rock anchor beam concrete under blasting. (a) 3.1 ms, (b) 6.1 ms, (c) 9.1 ms, (d) 12.1 ms, and (e) 15.0 ms.

FIGURE 12: The curve of the relationship between cumulative damage of rock anchor beam monitoring points along the axis of powerhouse and blasting times.

FIGURE 13: The curve of the relationship between cumulative damage of rock anchor beam monitoring points perpendicular to the axis of powerhouse and blasting times.

FIGURE 14: Cumulative damage nephogram of surrounding rock and rock anchor beam concrete under blasting. (a) 3.1 ms, (b) 6.1 ms, (c) 9.1 ms, (d) 12.1 ms, and (e) 15.0 ms.

FIGURE 15: The curve of the relationship between cumulative damage of rock anchor beam monitoring points along the axis of powerhouse and blasting times.

FIGURE 16: The curve of the relationship between cumulative damage of rock anchor beam monitoring points perpendicular to the axis of powerhouse and blasting times.

FIGURE 17: The monitoring points of rock anchor beam concrete (unit: m).

FIGURE 18: The regression relationship between the blasting damage value of R_1 and blasting times.

TABLE 3: The relationship between the damage value of some unit monitoring points of the rock anchor beam along the axis of the powerhouse and blasting times.

Monitoring points	Regression analysis	
	Relationship between damage value (y) and blasting times (x)	Confidence level
R_1	$y = -0.05x^3 + 0.3929x^2 - 0.5571x + 0.2$	0.9838
R_2	$y = -0.05x^3 + 0.4214x^2 - 0.7286x + 0.36$	0.9993
R_3	$y = -0.04x^3 + 0.0071x^2 + 0.2071x - 0.22$	0.9964
R_4	$y = -0.0058x^3 + 0.0617x^2 - 0.0457x$	0.9874
R_5	$y = -0.025x^3 + 0.2179x^2 - 0.4571x + 0.26$	0.9860
R_6	$y = -0.02x^3 + 0.0214x^2 - 0.0786x + 0.06$	0.9821

hip. The damage range of rock anchor beam along the axis of powerhouse is about 2.2 m–3.2 m.

4.1.2. Cumulative Damage Analysis of Concrete Perpendicular to the Axis of Powerhouse. By deriving the damage value of the cumulative damage monitoring point of the rock anchor beam perpendicular to the axis of the powerhouse, the relationship between the damage value and the blasting times can be obtained. The monitoring points perpendicular to the axis of the powerhouse are S_1 , S_2 , S_3 , and S_4 , and their distances from the tunnel face are 10.4 m, 11.2 m, 12.0 m, and 12.8 m, respectively. The curve relationship between the damage value of each unit and blasting times is shown in Figure 19.

Through regression calculation and analysis, the polynomial regression equation between unit blasting damage and blasting times is shown in Table 4, and the confidence levels are all greater than 0.98. It is indicated that the relationship between blasting damage of rock anchor beams

and blasting times conforms to the cubic polynomial relationship. In this simulation calculation, the height of the rock anchor beam perpendicular to the axis of the powerhouse is limited, and under the action of 5 times of blasting, damage and destruction are eventually generated.

4.2. Relationship between Cumulative Damage and Blast Center Distance

4.2.1. Cumulative Damage Analysis of Rock Anchor Beam Concrete along the Axis of the Powerhouse. The cumulative damage data of the rock anchor beam concrete along the axis of the powerhouse are derived to analyze the evolution law of rock anchor beam damage with the increase of the blast times. The variation law of the cumulative damage of the rock anchor beam along the axis of the powerhouse with the blast center distance is shown in Figure 20.

According to the analysis of the change rules, it is concluded that with the increase of the blast center distance,

FIGURE 19: Regression relationship between the blasting damage value of S_1 and the blasting times.

TABLE 4: Relationship between the damage value of some unit monitoring points of rock anchor beams perpendicular to the axis of the powerhouse and the blasting times.

Monitoring points	Regression analysis	
	Relationship between damage value (y) and blasting times (x)	Confidence level
S_1	$y = -0.0167x^3 + 0.1286x^2 - 0.1548x + 0.64$	0.9957
S_2	$y = -0.05x^3 + 0.4143x^2 - 0.7357x + 0.56$	0.9846
S_3	$y = -0.0667x^3 + 0.5786x^2 - 1.1548x + 0.64$	0.9994
S_4	$y = -0.0167x^3 + 0.2357x^2 - 0.6476x + 0.44$	0.9885

FIGURE 20: The variation law of cumulative damage of rock anchor beam along the axis of the powerhouse with the blast center distance (the 2nd blast).

the degree of blasting damage of the rock anchor beam gradually decreases. The unit closest to the blast source has the greatest degree of blasting damage. The size of the blast center distance also has a significant effect on the cumulative growth curve of blasting damage. The closer the unit is to the blast source, the more obvious the nonlinear characteristic of the damage curve is, and vice versa. The damage of the rock anchor beam shows a nonlinear decreasing characteristic with the increase of the blast center distance. After fitting analysis, this nonlinear decreasing relationship between the damage of the rock anchor beam and blast center distance is as follows:

$$y = A_0 + A_1x + A_2x^2 + A_3x^3 + A_4x^4, \quad (13)$$

where y is the blasting damage value of rock mass. x is the blast center distance. A_0 , A_1 , A_2 , A_3 , and A_4 are undetermined constants.

Through regression calculation and analysis, the polynomial regression equation between blasting damage and blast center distance is shown in Table 5, and the confidence levels are all greater than 0.98. It shows that the relationship between the blasting damage value of the rock anchor beam and the blast center distance conforms to the quartic polynomial relationship.

4.2.2. Cumulative Damage Analysis of Concrete Perpendicular to the Axis of the Powerhouse

The cumulative damage data of the rock anchor beam perpendicular to the axis of the powerhouse are derived to analyze the evolution law of the rock anchor beam damage with the increase of the blast center distance. The variation law of the cumulative damage of the rock anchor beam perpendicular to the axis of the powerhouse with the blast center distance is shown in Figure 21.

According to the analysis of the 5 times of change rules, the nonlinear decreasing relationship between the rock

TABLE 5: The relationship between the damage value of some unit monitoring points of the rock anchor beam along the axis of the powerhouse and the blast center distance.

Blast times	Regression analysis	Confidence level
	Relationship between damage value (y) and blast center distance (x)	
2 nd	$y = 0.0042x^4 - 0.0376x^3 + 0.0868x^2 - 0.0608x + 0.2082$	0.9967
3 rd	$y = 0.0115x^4 - 0.1312x^3 + 0.502x^2 - 0.8969x + 1.1599$	0.9918
4 th	$y = -0.0044x^4 + 0.0633x^3 - 0.3051x^2 + 0.3053x + 0.955$	0.9897
5 th	$y = 0.0094x^4 - 0.0849x^3 + 0.1634x^2 - 0.0732x + 1.004$	0.9932

FIGURE 21: The variation law of cumulative damage of rock anchor beam concrete perpendicular to the axis of the powerhouse with the blast center distance (the 2nd blast).

TABLE 6: The relationship between the damage value of some unit monitoring points of the rock anchor beam along the axis of the powerhouse and the blast center distance.

Blast times	Regression analysis	Confidence level
	Relationship between damage value (y) and blast center distance (x)	
2 nd	$y = 0.1563x^2 - 3.875x + 24$	1
3 rd	$y = 0.1563x^2 - 3.95x + 24.97$	0.9947
4 th	$y = 0.1172x^2 - 3.0563x + 20.015$	0.9987
5 th	$y = 0.0391x^2 - 1.2438x + 9.715$	0.9986

anchor beam damage and the blast center distance is as follows:

$$y = A_0 + A_1x + A_2x^2, \quad (14)$$

where y is the blasting damage value of rock mass. x is the blast center distance. A_0 , A_1 , and A_2 are undetermined constants.

Through regression calculation and analysis, the polynomial regression equation between blasting damage and blast center distance is shown in Table 6, and the confidence levels are all greater than 0.99. It shows that the relationship between the blast damage of the rock anchor beam and the blast center distance conforms to the quadratic polynomial.

5. Conclusion

Through the numerical simulation study on the cumulative damage growth law of rock anchor beam concrete under multiple blasting, the functional relations between the cumulative damage degree of rock anchor beam concrete under the action of blasting dynamic load and the blast times and the blast center distance are obtained:

- (1) When the full-section blasting excavation of the third floor of the powerhouse is completed, the damage range of the rock anchor beam concrete along the axis of the powerhouse is about 2.2 m–3.2 m. Due to the limited height of rock anchor beam concrete perpendicular to the axis of the powerhouse, under the action of 5 times of blasting dynamic load, the damage and destruction are eventually produced.
- (2) 7 days after the completion of the concrete pouring of the rock anchor beam, damage and destruction effects are produced under a single blasting action, indicating that no blasting activities are allowed. 28 days after the completion of the concrete pouring of the rock anchor beam, there is basically no damage and destruction under multiple blasting, indicating that the current blasting conditions will not have a significant impact on the rock anchor beam concrete.
- (3) 14 days after the completion of the concrete pouring of the rock anchor beam, the growth process of the cumulative damage effect of the rock anchor beam concrete structure under the disturbance of the

blasting excavation of the underground powerhouse has obvious nonlinear characteristics. With the increase of the number of blasting, it shows a nonlinear increasing characteristic, and with the increase of the blasting center distance, it shows a nonlinear decreasing characteristic.

- (4) The relationships between cumulative damage of rock anchor beam along and perpendicular to the axis of powerhouse and blasting times are in accordance with the cubic polynomials. The relationships between the cumulative damage of rock anchor beam along and perpendicular to the axis of powerhouse and blast center distance are in accordance with the quartic polynomial and quadratic polynomial.

Data Availability

The curve data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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