Research Article

Strength Assessment of Broken Rock Postgrouting Reinforcement Based on Initial Broken Rock Quality and Grouting Quality

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To estimate postgrouting rock mass strength growth is important for engineering design. In this paper, using self-developed indoor pressure-grouting devices, 19 groups of test cubic blocks were made of the different water cement ratio grouting into the broken rock of three kinds of particle sizes. The shear strength parameters of each group under different conditions were tested. Then this paper presents a quantitative calculation method for predicting the strength growth of grouted broken rock. Relational equations were developed to investigate the relationship between the growth rates of uniaxial compressive strength (UCS), absolute value of uniaxial tensile strength (AUTS), internal friction angle, and cohesion for post- to pregrouting broken rock based on Mohr-Coulomb strength criterion. From previous test data, the empirical equation between the growth rate of UCS and the ratio of the initial rock mass UCS to the grout concretion UCS has been determined. The equations of the growth rates of the internal friction coefficient and UCS for grouting broken rock with rock mass rating (RMR) and its increment have been established. The calculated results are consistent with the experimental results. These observations are important for engineered design of grouting reinforcement for broken rock mass.

1. Introduction

Geotechnical and underground engineering are often in complex geological environments where broken rock mass is one of the frequently encountered medium. In such cases, grouting is usually used for reinforcement, which can well improve the condition, integrity, and the bearing capacity (Lombardi [1], Glossop [2]). For such engineering, when designed or calculated by the engineers, it is an important problem to accurately determine the mechanical model and the parameters of the grouting broken rock mass. For example, Li et al. [3] converted the mechanical parameters of the original rock to determine the parameters of grouting reinforcement in the mechanical analysis of broken rock mass section in a tunnel construction design; Youn and Tonon [4] used parameters obtained from the experimental results of grouting layer of Texas Gulf Coast Soil (Sellountou [5])

to study the behavior of postgrouted drilled shafts at the Brazo River Bridge; these two methods of taking parameters will inevitably lead to relatively large error. In the support design of No. 1 Coal Mine, Yangquan Coal Industry Group Co. Ltd., Wang et al. [6] considered the grouting layer as a homogeneous medium and combined the linear elastic model with the Mohr-Coulomb criterion to simulate the support model; the same method was used in the analysis for undersea tunnel stability and in prediction of displacement of tunnel surface (He and Wang [7], Sharghi et al. [8]); Dai et al. [9] used the Kelvin-Hooke model to characterise the mechanical model of the grouting reinforced bodies in broken rock mass; in the study of support in a tunnel, Dai et al. [10] combined the elastic-plastic model with the D-P criterion for the grouting layer; the most critical step in these studies was to determine the mechanical parameters (strength, modulus, friction angle, and cohesion) of the

model by means of field tests rather than by empirical formulas or any specifications.

From the above, it can be seen that the research on the determination of mechanical models and parameters of grouting broken rock mass is still in the initial stage and it is difficult to determine the parameters quantitatively. The main obstacle lies in the heterogeneity of broken rock mass and the complexity of fracture structure (ISRM [11]). For the support design of underground engineering, slope engineering, and foundation pits in broken rock mass, the unpredictability of the mechanics parameters of the postgrouting broken rock mass makes the design and numerical analysis result in great errors.

At present, study of the mechanics model, parameters, and effect prediction of postgrouting broken rock mass is relatively sparse. Some of them are typical; for example, through the mechanical experiments and borehole expansion test during the grouting process, Kikuchi et al. [12] found that the mechanical properties of the broken rock tend to become uniform after grouting; based on the experimental data of Quantai Coal Mine and damage mechanics, the constitutive model of postgrouting broken rock mass was established by Yang and Zhang [13]; Butrón et al. [14] performed laboratory tests, including fall-cone, unconfined compression, and triaxial tests, to determine the dynamic and quasi-static mechanical model of slurry consolidation body; Zolfaghari et al. [15] applied Q system into estimation of the change of elastic modulus before and after grouting through field test at the Bakhtiari dam site; experimental results showed that the Q system can be used as a practical and undemanding method in evaluation of rock mass quality by the grouting treatment, while there is no research on estimating parameters of grouting reinforcement by GSI index or RMR; through neural networks, Tinoco et al. [16] analyze a large number of experimental data to predict the strength of grouting reinforced rock mass, which is time-consuming and not universal; Akan et al. [17] studied the fitting model of predicting the compressive strength of grouting body by nonlinear regression of experimental data; Zhou [18] calculated the relationship between grouting parameters and bearing capacity of pile by numerical simulation. In general, the grouting broken rock mass is still in the exploratory stage in the study of mechanical model and parameters; qualitative research is more, while quantitative research is less and one-sided in this field.

In view of these deficiencies, this paper focuses on broken rock mass within rock mass classes IV (poor) and V (very poor) of the classification scheme based on the rock mass rating (RMR) system (Bieniawski et al. [19, 20]). Through self-developed laboratory pressure-grouting device and the direct shear apparatus, a large number of uniaxial compression test and direct shear test have been carried out on specimen of slurry consolidation bodies and broken rock mass pre- and postgrouting reinforcement; relationship between the growth rates of the uniaxial compression strength (UCS), absolute value of uniaxial tensile strength (AUTS), internal friction coefficient, and cohesion for pre- and postgrouting reinforcement are derived by using the Mohr-Coulomb strength criterion; combined with the empirical formulae of

FIGURE 1: Medium particle broken rock in the steel mold.

Aydan et al. [21, 22], mathematical calculation process and curves for each growth rate of these four parameters and rock mass rating (RMR) are presented in Sections 3–7, with the UCS of grout concretion into account. Through comparison, it indicates that the experimental data are in good agreement with the equation curve. The RMR-based method to predict the mechanical parameters of broken rock mass grouting reinforcement and the derived equations can be used effectively for engineering design of various broken rock mass engineering and for the determination of parameters in numerical analysis, which will reduce the calculation error. Based on previous studies [23, 24], RMR and Mohr-Coulomb parameters can be converted into GSI index, Q system, and Hoek-Brown parameters by further research.

2. Shear Strength Tests on the Concretion of Grouting Reinforced Broken Rock

2.1. Test Method. We carried out the shear strength test of grouting reinforced bodies of broken rock mass. The test methods are as follows.

(1) *Simulate Broken Rock Mass with Crushed Stone (or Broken Rock).* The rock is weak weathered limestone from the Mengmu quarry in Nanjing, Jiangsu.

(2) *Broken Degree of the Rock Mass Is Represented by Different Particle Size of Broken Rock.* We used broken rock with three kinds of particle size, namely, fine (F) 0~2 cm, medium (M) 2~4 cm, and coarse (R) 4~6 cm. Figure 1 shows that the medium particle broken rock is placed in the steel mold.

(3) *Test Blocks Making Equipment.* The self-developed pressure-grouting device is used in the laboratory as shown in Figure 2. The device is composed of three parts, which are the sealing steel mold, the sealing steel pressure pail containing grout, and the air compressor. The test procedures are the following:

- (i) Placing broken rock into the sealing steel mold
- (ii) Pouring grout mixing cement with water into the pressure pail

TABLE 1: Test schemes and groups.

Scheme number	Broken rock particle size	Group number (5 test blocks in each group)					Test purpose
		Water cement ratio					
		0.7	0.9	1.1	1.3	2.5	
I	—	C07	C09	C11	C13	—	To test the shear strength of grout concretion
II	F	—	—	—	—	OF	To test the shear strength of the original broken rock
	M	—	—	—	—	OM	
	R	—	—	—	—	OR	
III	F	F07	F09	F11	F13	—	To test the shear strength of fine particle broken rock grouting reinforced concretion
IV	M	M07	M09	M11	M13	—	To test the shear strength of medium particle broken rock grouting reinforced concretion
V	R	R07	R09	R11	R13	—	To test the shear strength of coarse particle broken rock grouting reinforced concretion

FIGURE 2: Grouting test device.

- (iii) Starting the air compressor to increase pressure of the pail and grouting into the sealing steel mold filled broken rock
- (iv) Discharging excess grout from the outlet and maintaining 0.3 MPa of the grouting pressure for 5 minutes
- (v) Demolding for the test blocks after condensation hardening for 24 hours

The test block is the cube of $150 \times 150 \times 150 \text{ mm}^3$. The typical test blocks are shown in Figure 3.

(4) *Test Schemes.* The purpose of this experiment is to analyze the influence of Initial Rock Mass Quality and Grouting Quality on the strength of broken rock grouting reinforced concretion. From the research results of literature [25, 26], strength growth rate of broken rock mass is closely related

FIGURE 3: Typical test blocks.

to the strength of the original rock and the strength of grout concretions. The five test schemes are developed for this experiment. All 95 test blocks are made in 19 groups (each group has five), as shown in Table 1.

- (i) In scheme I, there are 4 groups to test the shear strength of pure cement grout concretion under different water cement ratio.
- (ii) In scheme II, there are 3 groups to test the shear strength of pure broken rock using a very high water cement ratio grouting reinforced concretion, which considered grout cementing action is very small.
- (iii) In scheme III, there are 4 groups to test the shear strength of fine particle broken rock grouting reinforced concretion under different water cement ratio.
- (iv) In scheme IV, there are 4 groups to test the shear strength of medium particle broken rock grouting

TABLE 2: Normal stress and peak shear strength.

Scheme number	Group number	Normal or Shear	Stress/MPa					
			Test block number					
			1	2	3	4	5	
I	C07	σ_n	1.333	2.444	3.111	3.778	4.444	
		τ_m	3.504	5.182	5.782	6.493	7.124	
	C09	σ_n	1.111	1.778	2.444	3.111	3.778	
		τ_m	2.562	3.131	3.891	4.257	4.666	
	C11	σ_n	0.889	1.556	2.222	2.889	3.556	
		τ_m	2.049	2.486	2.753	3.142	3.631	
	C13	σ_n	0.889	1.332	1.776	2.220	2.664	
		τ_m	1.637	1.703	1.851	1.977	2.149	
II	OF	σ_n	0.889	1.332	1.776	2.220	2.664	
		τ_m	0.650	0.863	1.030	1.157	1.419	
	OM	σ_n	0.889	1.332	1.776	2.220	2.664	
		τ_m	1.006	1.367	1.588	1.760	1.929	
	OR	σ_n	0.889	1.332	1.776	2.220	2.664	
		τ_m	1.677	1.851	2.180	2.573	2.782	
	III	F07	σ_n	1.111	2.222	3.333	4.444	5.555
			τ_m	2.978	3.690	4.480	5.017	5.603
F09		σ_n	1.111	2.000	2.889	3.778	4.444	
		τ_m	2.553	2.880	3.697	3.848	4.352	
F11		σ_n	0.889	1.556	2.222	2.889	3.556	
		τ_m	2.153	2.477	2.953	3.275	3.530	
F13		σ_n	0.889	1.333	1.778	2.222	2.667	
		τ_m	2.045	2.314	2.603	2.757	2.963	
IV	M07	σ_n	1.333	2.222	3.111	4.000	4.889	
		τ_m	3.512	4.233	4.746	5.384	6.032	
	M09	σ_n	0.888	1.332	2.222	3.111	4.000	
		τ_m	2.688	3.067	3.557	4.256	4.736	
	M11	σ_n	0.888	1.776	2.664	3.552	4.440	
		τ_m	2.319	3.007	3.669	4.236	4.588	
	M13	σ_n	0.667	1.334	2.222	2.889	3.556	
		τ_m	2.220	2.575	3.108	3.735	3.923	
V	R07	σ_n	1.778	2.667	4.000	5.333	6.667	
		τ_m	4.438	5.034	6.140	7.074	8.769	
	R09	σ_n	1.333	2.222	3.111	4.000	4.889	
		τ_m	3.538	3.958	4.876	5.475	6.433	
	R11	σ_n	1.333	1.778	2.222	3.111	4.000	
		τ_m	3.003	3.516	3.879	4.382	5.285	
	R13	σ_n	1.556	2.000	2.444	2.889	3.333	
		τ_m	3.083	3.366	4.035	4.264	4.488	

reinforced concrete under different water cement ratio.

- (v) In scheme V, there are 4 groups to test the shear strength of coarse particle broken rock grouting reinforced concrete under different water cement ratio.

(5) *Shear Strength Test.* The shear strength test can be carried out on the blocks cured for 28 days using the YZW100 direct shear apparatus.

2.2. Test Results. There are five test blocks in each group. Each test block is placed horizontally in direct shear apparatus. A constant vertical stress (or normal stress σ_n) is applied

on block first, then a horizontal stress (or shear stress τ) is applied, and then shear displacement increases with shear stress varying until failure. The shear stress-displacement curve can be obtained. A constant normal stress is applied gradually from small to large on each block in each group. To prevent the normal stress crushing blocks, each group maximal normal stress must be less than UCS. The typical shear stress-displacement curves are shown in Figure 4.

From shear stress-displacement curve of each test block, the peak shear strength τ_m and the corresponding normal stress σ_n are obtained, as Table 2 shows.

Suppose the shear strength of test block meets the Mohr-Coulomb strength criterion. Then the relation between the

FIGURE 4: Typical shear stress-displacement curves.

TABLE 3: Each group's shear strength parameter.

Scheme number	Group number	Shear strength parameter		R^2 correlation coefficient
		c/MPa	$f = \tan \varphi$	
I	C07	2.150	1.147	0.9867
	C09	1.746	0.800	0.9815
	C11	1.539	0.573	0.9917
	C13	1.344	0.292	0.9819
II	OF	0.291	0.413	0.9884
	OM	0.634	0.504	0.9702
	OR	1.039	0.661	0.9850
III	F07	2.381	0.592	0.9945
	F09	1.930	0.540	0.9642
	F11	1.694	0.533	0.9857
	F13	1.625	0.513	0.9874
IV	M07	2.616	0.696	0.9958
	M09	2.140	0.658	0.9958
	M11	1.834	0.649	0.9865
	M13	1.782	0.623	0.9880
V	R07	2.759	0.864	0.9833
	R09	2.299	0.817	0.9862
	R11	1.999	0.809	0.9872
	R13	1.930	0.792	0.9697

peak shear strength (τ_m) and normal stress (σ_n) on each group test should satisfy the following:

$$\tau_m = \sigma_n \tan \varphi + c = \sigma_n f + c, \quad (1)$$

where c is the cohesion; φ is the internal friction angle; f is the internal friction coefficient; σ_n is the normal stress acting on the failure plane; and τ_m is the maximum shear stress acting on the failure plane.

From Table 2, we can draw the strength test curve of each group; see Figure 5. The shear strength parameters of the each group test can be obtained using linear regression method; see Table 3.

3. Strength Growth of Postgrouting Broken Rock Mass

In this paper, the following assumptions apply for broken rock mass:

- The rock mass is cut by several structural features, which form fractured massive, cataclastic, or scattered textures.
- Rock mass strength is determined using the RMR classification system.
- The broken rock mass is class IV or V, and the RMR's value is less than 25.
- Rock mass strength satisfies the Mohr-Coulomb strength criterion for low stress levels.
- Grouting improves the rock mass quality, leading to an increase in RMR (ΔRMR).

The Mohr-Coulomb criterion is one of the most trusted and widely used linear strength criteria for soils and rocks. Jaeger et al. [27] define the criterion as follows:

$$\sigma_1 = \frac{1 + \sin \varphi}{1 - \sin \varphi} \sigma_3 + \frac{2c \cos \varphi}{1 - \sin \varphi}, \quad (2)$$

where σ_1 is the maximum principal stress in failure and σ_3 is the minimum principal stress in failure.

Expressions of the relationships between rock mass strength parameters are determined as follows:

$$c = \frac{1}{2} \sqrt{S_c S_t}, \quad (3)$$

where S_c is the uniaxial compressive strength (UCS) and S_t is the absolute value of uniaxial tensile strength (AUTS).

$$f = \tan \varphi = \frac{S_c - S_t}{2\sqrt{S_c S_t}} \quad (4)$$

$$\alpha = \frac{S_t}{S_c} = \frac{1 - \sin \varphi}{1 + \sin \varphi}, \quad (5)$$

where α is the ratio of the AUTS to the UCS.

The growth rates of each factor are determined as follows.

(i) The growth rate of UCS is expressed as

$$\xi_c = \frac{\Delta S_c}{S_{cb}} = \frac{S_{ca} - S_{cb}}{S_{cb}}, \quad (6)$$

where ξ_c is the UCS growth rate; ΔS_c is the growth value of UCS; S_{cb} is the UCS of pregrouting broken rock; S_{ca} is the UCS of post-grouting broken rock.

FIGURE 5: Shear strength curves.

(ii) The absolute value of UTS growth rate is expressed as

$$\xi_t = \frac{\Delta S_t}{S_{tb}} = \frac{S_{ta} - S_{tb}}{S_{tb}}, \quad (7)$$

where ξ_t is the AUTS growth rate; ΔS_t is the growth value of AUTS; S_{tb} is the AUTS of pregrouting broken rock; S_{ta} is the AUTS of postgrouting broken rock.

(iii) The growth rate of cohesion is written as

$$\xi_{coh} = \frac{\Delta c}{c_b} = \frac{c_a - c_b}{c_b}, \quad (8)$$

where ξ_{coh} is the cohesion growth rate; Δc is the growth value of the cohesion; c_b is the cohesion of pregrouting broken rock; c_a is the cohesion of post-grouting broken rock.

(iv) The growth rate of internal friction coefficient is defined as

$$\xi_f = \frac{\Delta f}{f_b} = \frac{f_a - f_b}{f_b}, \quad (9)$$

where ξ_f is the growth rate of internal friction coefficient; Δf is the growth value of internal friction coefficient; f_b is internal friction coefficient of pregrouting broken rock; f_a is internal friction coefficient of postgrouting broken rock.

By substituting (3) into (8), the relationships between the growth rate of cohesion and the growth rates of UCS and AUTS can be determined:

$$\xi_{coh} = \frac{\sqrt{S_{ca}S_{ta}}}{\sqrt{S_{cb}S_{tb}}} - 1 = \sqrt{(\xi_c + 1)(\xi_t + 1)} - 1. \quad (10)$$

Equation (10) can also be expressed as

$$\xi_t = \frac{(\xi_{coh} + 1)^2}{\xi_c + 1} - 1. \quad (11)$$

Similarly, by substituting (4) into (9), the relationships between the growth rate of internal friction coefficient and the growth rates of cohesion, UCS, and AUTS can be determined:

$$\begin{aligned} \xi_f &= \frac{S_{ca} - S_{ta}}{S_{cb} - S_{tb}} \cdot \sqrt{\frac{S_{cb}S_{tb}}{S_{ca}S_{ta}}} - 1 \\ &= (\xi_c + 1) \frac{1 - \alpha_a}{1 - \alpha_b} \cdot \frac{1}{\sqrt{(\xi_c + 1)(\xi_t + 1)}} - 1 \\ &= \frac{k(\xi_c + 1)}{\xi_{coh} + 1} - 1, \end{aligned} \quad (12)$$

where α_b is the ratio of the AUTS to the UCS of pregrouting broken rock; α_a is the ratio of the AUTS to the UCS of postgrouting broken rock; and

$$k = \frac{1 - \alpha_a}{1 - \alpha_b} = \frac{\sin \varphi_a (1 + \sin \varphi_b)}{\sin \varphi_b (1 + \sin \varphi_a)}, \quad (13)$$

where φ_b is internal friction angle of pregrouting broken rock; φ_a is internal friction angle of postgrouting broken rock; k is within 1.01~1.50 generally.

FIGURE 6: The relation between the growth rate of UCS and the relative UCS.

4. The Growth Rate of UCS of Post- to Pregrouting Broken Rock

The UCS of broken rock mass postgrouting reinforcement increases. Xu et al. [25] collected 28 days of UCS test data for broken rock mass pre- and postgrouting reinforcement, and analysed variation laws of the growth rate of UCS (ξ_c) with the relative UCS of the initial rock mass (η). The relative UCS is a dimensionless parameter that is a ratio of the initial rock mass UCS (S_{cb}) to the UCS of hardened cement grout concretion after 28 days (q_c). It was proposed as

$$\eta = \frac{S_{cb}}{q_c}, \quad (14)$$

where η is the relative UCS of initial rock mass UCS, that is, the ratio of the initial rock mass UCS to the grout concretion UCS; q_c is the UCS of hardened cement grout concretion after 28 days.

The test data in Table 3 are made being dimensionless using (6) and (14). And the results are shown in Table 4 and Figure 6. In Table 4, the UCS is calculated with Mohr-Coulomb failure criterion (1). Table 4 shows the change of UCS of the broken rock pre- and postgrouting reinforcement and the cement grout concretion for 28 days with broken rock particle sizes and grout water cement ratio. Figure 6 shows that the UCS growth rate of the broken rock postgrouting

TABLE 4: Test results of growth rate of UCS on broken rock grouting concretion.

Broken rock type/water cement ratio of grout	UCS			η	ξ_c
	Broken rock grouting concretion S_{ca}/MPa	Cement grout concretion q_c/MPa	Original Broken rock S_{cb}/MPa		
	$2c \cos \varphi / (1 - \sin \varphi)$	$2c \cos \varphi / (1 - \sin \varphi)$	$2c \cos \varphi / (1 - \sin \varphi)$	S_{cb}/q_c	$(S_{ca} - S_{cb})/S_{cb}$
F/0.7	8.351	11.477	0.870	0.076	8.599
F/0.9	6.471	7.265	0.870	0.120	6.437
F/1.1	5.644	5.311	0.870	0.164	5.487
F/1.3	5.320	3.585	0.870	0.243	5.114
M/0.7	10.014	11.477	2.059	0.179	3.864
M/0.9	7.940	7.265	2.059	0.283	2.856
M/1.1	6.752	5.311	2.059	0.388	2.279
M/1.3	6.420	3.585	2.059	0.574	2.118
R/0.7	12.059	11.477	3.864	0.337	2.120
R/0.9	9.694	7.265	3.864	0.532	1.509
R/1.1	8.378	5.311	3.864	0.728	1.168
R/1.3	7.981	3.585	3.864	1.078	1.065

reinforcement varies with the relative UCS of the original broken rock.

From Figure 6, we find that a correlation between UCS growth rate (ξ_c) for broken rock postgrouting reinforcement and relative UCS (η) for broken rock is obvious. The relationship can be expressed as follows using fitting method.

$$\xi_c = 1.14\eta^{-0.8} \quad (15)$$

(Correlation coefficient $R^2 = 0.9309$).

5. The Growth Rate of Post- to Pregroutings Broken Rock Mass Shear Strength

The empirical formula for estimating the internal friction angle of broken rock mass with RMR was presented by Aydan and Kawamoto [22] as follows:

$$\varphi_b = \frac{\pi(20 + 0.5\text{RMR})}{180} = \frac{\pi}{9} + \frac{\pi}{360}\text{RMR} \quad (\text{radian}). \quad (16)$$

Then the internal friction angle of the rock mass postgrouting reinforcement is

$$\begin{aligned} \varphi_a &= \frac{\pi}{9} + \frac{\pi}{360}(\text{RMR} + \Delta\text{RMR}) \\ &= \varphi_b + \frac{\pi}{360}\Delta\text{RMR} \quad (\text{radian}). \end{aligned} \quad (17)$$

The growth rate of internal friction coefficient for post- to pregrouting broken rock mass can be determined from (9), (16), and (17):

$$\xi_f = \frac{\tan \varphi_a - \tan \varphi_b}{\tan \varphi_b} = \frac{\tan(\varphi_b + \pi\Delta\text{RMR}/360)}{\tan \varphi_b} - 1. \quad (18)$$

In addition, the growth rate of cohesion for post- to pregrouting broken rock mass is calculated from (12):

$$\xi_{\text{coh}} = \frac{k(\xi_c + 1)}{\xi_f + 1} - 1. \quad (19)$$

6. The Expression of the Growth Value of RMR for Post- to Pregroutings Broken Rock Mass

The empirical formula for predicting the UCS of broken rock mass with RMR was presented by Masada and Han [23]:

$$S_{cb} = 0.0016\text{RMR}^{2.5} \quad (\text{MPa}). \quad (20)$$

The UCS of broken rock mass postgrouting is as follows.

$$S_{ca} = 0.0016(\text{RMR} + \Delta\text{RMR})^{2.5} \quad (\text{MPa}). \quad (21)$$

Based on (6), (20), and (21), the expression of the growth rate of UCS for post- to pregrouting broken rock mass is as follows:

$$\begin{aligned} \xi_c &= \frac{S_{ca} - S_{cb}}{S_{cb}} = \frac{(\text{RMR} + \Delta\text{RMR})^{2.5}}{\text{RMR}^{2.5}} - 1 \\ &= \left(1 + \frac{\Delta\text{RMR}}{\text{RMR}}\right)^{2.5} - 1. \end{aligned} \quad (22)$$

So, the growth value of RMR for post- to pregrouting broken rock mass from (22) and (15) is determined as follows:

$$\begin{aligned} \Delta\text{RMR} &= [(\xi_c + 1)^{0.4} - 1] \text{RMR} \\ &= \left[(1.14\eta^{-0.8} + 1)^{0.4} - 1\right] \text{RMR}. \end{aligned} \quad (23)$$

FIGURE 7: The influence of RMR of pregrouting broken rock mass and UCS of grout concretion (q_c) on the growth rate of internal friction coefficient for post- to pregrouting broken rock mass.

FIGURE 8: The influence of RMR of pregrouting broken mass and UCS of grout concretion (q_c) on the growth rate of cohesion for post- to pregrouting broken rock mass.

7. Analyses of Strength Growth Rates for Post- to Pregrouting Broken Rock Mass

The final strength and strength improvement of broken rock mass postgrouting reinforcement are dependent on a range of factors including grouting materials, grouting pressure, grouting duration, extent of fracturing, and the permeation coefficient. The most important factors are grouting quality and pregrouting rock mass quality. The grouting quality may be characterised by the UCS of grout concretion (q_c), while the initial rock mass quality may be characterised by RMR. When q_c is from 5 MPa to 40 MPa, the RMR ranges from 10 to 40.

7.1. Influences of RMR and UCS of Grout Concretion on Shear Strength Growth Rates. The UCS growth rate for pre- to postgrouting broken rock mass can be determined using (15) and (20) when RMR and q_c are known. Δ RMR can be determined from (23). The growth rate of internal friction coefficient and the growth rate of cohesion which can be determined from (18) and (19) are shown in Figures 7 and 8.

These figures demonstrate that initial rock and grouting quality strongly influence the growth rate of shear strength parameters for grouted broken rock mass.

The effectiveness of grouting reinforcement has an inverse relationship with rock mass quality. Detailed results are as follows:

- (i) The growth rates of shear strength parameters decrease relatively sharply with initial RMR increase and gradually slow to a constant rate across higher RMR values.
- (ii) The growth rates of shear strength parameters increase with the increase of UCS of grout concretion after 28 days (q_c). This trend is clearer for low values of RMR.

- (iii) The growth rate of cohesion is much higher than that of internal friction coefficient under the same conditions. The growth rate of internal friction coefficient is below 0.4 for RMR greater than 10 and q_c less than 14 while the growth rate of cohesion is greater than 1 generally for RMR less than 20 and q_c greater than 2 MPa.

7.2. Influences of RMR and UCS of Grout Concretion on Growth Rates of UCS and AUTS. The growth of UCS and AUTS for broken rock mass postgrouting can be determined from (11) and (22) and seen in Figures 9 and 10. These figures demonstrate that initial rock mass and grouting quality also have a great influence on the growth rates of UCS and AUTS for pre- to postgrouting broken rock mass. These growth rate values are much higher when the rock mass quality is poor and decrease with an increase of RMR. Detailed conclusions are as follows.

- (i) The growth rates of UCS and AUTS rapidly decrease with an increase of RMR and gradually smooth out towards higher RMR values.
- (ii) The growth rates of UCS and AUTS increase with an increase of UCS of grout concretion after 28 days (q_c). Growth rate is much higher when the rock mass quality level is low.
- (iii) The growth rate of UCS is always greater than that of AUTS under the same conditions.

8. Verification

According to the definition of the growth rates of cohesion and internal friction angle with (8) and (9), the test results of the growth rate of the shear strength parameters for broken rock postgrouting reinforcement can be obtained from Tables 3 and 4, as shown in Table 5.

TABLE 5: Test results of growth rate of shear strength parameters for broken rock grouting reinforcement.

Broken rock type/water cement ratio of grout	η	Shear strength parameters				$\xi_{coh} = (c_a - c_b)/c_b$	$\xi_f = (f_a - f_b)/f_b$
		Broken rock grouting concretion	Original broken rock				
	S_{cb}/q_c	c_a/MPa	f_a	c_b/MPa	f_b		
F/0.7	0.076	2.381	0.592	0.291	0.413	7.182	0.433
F/0.9	0.120	1.930	0.540	0.291	0.413	5.632	0.308
F/1.1	0.164	1.694	0.533	0.291	0.413	4.8213	0.291
F/1.3	0.243	1.625	0.513	0.291	0.413	4.584	0.242
M/0.7	0.179	2.616	0.696	0.634	0.504	3.126	0.381
M/0.9	0.283	2.140	0.658	0.634	0.504	2.375	0.306
M/1.1	0.388	1.834	0.649	0.634	0.504	1.8934	0.288
M/1.3	0.574	1.782	0.623	0.634	0.504	1.811	0.236
R/0.7	0.337	2.759	0.864	1.039	0.661	1.655	0.307
R/0.9	0.532	2.299	0.817	1.039	0.661	1.213	0.236
R/1.1	0.728	1.999	0.809	1.039	0.661	0.924	0.224
R/1.3	1.078	1.930	0.792	1.039	0.661	0.858	0.198

FIGURE 9: The Influence of RMR of pregrouting broken rock mass and UCS of grout concretion (q_c) on the growth rate of AUTS for post- to pregrouting broken rock mass. FIGURE 10: The influence of RMR of pregrouting broken rock mass and UCS of grout concretion (q_c) on the growth rate of UCS for post- to pregrouting broken rock mass.

In this paper, three types of broken rock (F, M, and R) are used. Their UCS values are 0.870, 2.059, and 3.864 MPa, respectively, from Table 4. According to (20), the corresponding values of RMR on the three original broken rocks are calculated as 17.531, 12.421, and 22.551. When the UCS of the cement grout concretion was changed within 2 MPa~14 MPa (approximately corresponding to water cement ratio from 1.84 to 0.62), the relative strength for the three-type original broken rock is corresponding to 0.435~0.062, 1.030~0.147, and 1.932~0.276, respectively. We can get the growth of RMR from formula (23) for broken rock postgrouting reinforcement. Therefore, we can obtain the changing curves of the friction coefficient growth rate (see Figure 11) by formula (18) and the change curves of cohesion growth rate (see Figure 12) by formula (19) with the values of relative UCS of three-type broken rocks.

It can be shown from Figure 11 that there is a good agreement between the theoretical curves and test points

on the cohesion growth rate of broken rock postgrouting reinforcement with the relative UCS. The three theoretical curves are among the test points.

It can be seen from Figure 12 that there is also a good agreement between the theoretical curves and test points on the growth rate of the friction coefficient for the broken rock postgrouting reinforcement. The distribution law of three theoretical curves is the same as the test data. The finer the particle of broken rock, the smaller the relative error between theoretical and test on broken rock postgrouting reinforcement.

It is shown by analyzing Figures 11 and 12 intuitively that the theoretical method to estimate the strength growth rate of broken rock postgrouting reinforcement established in this paper is reasonable and provides a new analysis approach for the design of engineering grouting reinforcement.

FIGURE 11: Comparison between theoretical curves and test points on the cohesive growth rate.

FIGURE 12: Comparison between theoretical curves and test points on growth rate of internal friction coefficient.

9. Concluding Remarks

This paper presents an investigation into quantifying strength growth estimations of post- to pregrouting broken rock. Using self-developed indoor pressure-grouting devices, we make cube test blocks of the different water cement ratio grouting into different particle sizes broken rock, having 95 of 19 groups in all. Five test schemes are developed to test the shear strength. They test, respectively, the shear

strengths of grout concretion with different water cement ratio, of original broken rock of different particle size, and of fine, medium, and coarse particle broken rock grouting reinforcement concretion under different water cement ratio. The cohesion and internal friction coefficient of each group are obtained by the direct shear test. This paper presents a quantitative calculation method for predicting the strength growth of grouted broken rock mass. Relational equations were developed to investigate the relationship between the growth rates of uniaxial compressive strength (UCS). The relational equations between the growth rates of UCS, AUTS, the internal friction coefficient, and broken rock mass cohesion are determined based on the Mohr-Coulomb strength criterion under low-level stress. Expressions for the growth rate of the internal friction coefficient and UCS for grouted broken rock and RMR progression throughout grouting (Δ RMR) have been developed using the empirical equations of Aydan [12, 13]. Using test UCS data, the empirical relational expression between the growths rate of UCS for post- to pregrouting broken rock and the UCS of grout concretion has been determined. The results show that initial rock quality and grouting quality have a significant influence on the strength growth rate of rock mass, UCS, AUTS, internal friction coefficient, and broken rock mass cohesion. The strength growth rates of grouted broken rock mass decrease with increasing of initial RMR and increase with increasing of UCS of grout concretion. The growth rate of UCS is always greater than AUTS under the same conditions. The growth rate of cohesion is much higher than that of internal friction coefficient for the same conditions. The comparative analysis shows that the theoretical results of change rules on growth rate change of the cohesion and the internal friction coefficient for broken rock postgrouting reinforcement with the relative UCS are basically the same as the test rules. These findings are important for the engineered design of broken rock mass grouting reinforcement. The RMR-based method to predict the mechanical parameters of broken rock mass grouting reinforcement and the derived equations can be used effectively for engineering design of various broken rock mass engineering and for the determination of parameters in numerical analysis, which will reduce the calculation error. Based on previous studies [23, 24], RMR and Mohr-Coulomb parameters can be converted into GSI index, Q system, and Hoek-Brown parameters by further research. The self-developed pressure-grouting device allows the grouting test to be carried out in the laboratory and is simple in structure and easy to operate. It is worth to be extended to more geotechnical grouting test.

Abbreviations

- c: Cohesion (MPa)
- φ : Internal friction angle
- f: Internal friction coefficient
- σ_n : Normal stress acting on the failure plane
- τ_m : Maximum shear stress acting on the failure plane
- σ_1 : Maximum principal stress in failure
- σ_3 : Minimum principal stress in failure
- S_c : Uniaxial compressive strength (UCS)

S_t :	Absolute value of uniaxial tensile strength (AUTS)
α :	Ratio of the AUTS to the UCS
ξ_c :	UCS growth rate
ΔS_c :	Growth value of UCS
S_{cb} :	UCS of pre-grouting broken rock
S_{ca} :	UCS of post-grouting broken rock
ξ_t :	AUTS growth rate
ΔS_t :	Growth value of AUTS
S_{tb} :	AUTS of pregrouting broken rock
S_{ta} :	AUTS of postgrouting broken rock
ξ_{coh} :	Cohesion growth rate
Δc :	Growth value of the cohesion
c_b :	Cohesion of pregrouting broken rock
c_a :	Cohesion of postgrouting broken rock
ξ_f :	Growth rate of internal friction coefficient
Δf :	Growth value of internal friction coefficient
f_b :	Internal friction coefficient of pregrouting broken rock
f_a :	Internal friction coefficient of postgrouting broken rock
α_b :	Ratio of the AUTS to the UCS of pre-grouting broken rock
α_a :	Ratio of the AUTS to the UCS of post-grouting broken rock
k :	$(1 - \alpha_a)/(1 - \alpha_b)$
φ_b :	Internal friction angle of pre-grouting broken rock
φ_a :	Internal friction angle of postgrouting broken rock
η :	Relative UCS, that is, the ratio of the initial broken rock UCS to the grout concretion UCS
q_c :	UCS of hardened cement grout concretion after 28 days
RMR:	Rock mass rating
ΔRMR :	Growth value of RMR.

Conflicts of Interest

The authors declare that they have no conflicts of interest regarding the publication of this paper.

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