Experimental Study on Physical and Mechanical Properties of Mixed Filling behind Retaining Wall

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Abstract. Through large-scale triaxial shear test and particle sieving test, the particle breakage characteristics and shear strength of mixed coarse-grained fillers behind a high filling wall in Guizhou Province were studied. The effects of mudstone content, particle gradation, relative density and stress state of coarse-grained mixed fillers on particle breakage and shear strength were analyzed. With the increase of mudstone content, gradation and relative density, the particle breakage rate decreases. With the increase of confining pressure and stress, the particle breakage rate of coarse-grained soil also increases. The shear strength cohesion increases first and then decreases with the increase of mudstone content, and the internal friction angle decreases gradually. With the improvement of particle size distribution and the increase of relative density, the cohesion and internal friction angle increase. At the same time, a multivariate non-linear regression model was established to synthesize the comprehensive internal friction angle. The predicted values were in agreement with the experimental data, and the optimum mixed packing parameters were selected.

1. Introduction
Mountainous areas (including hills and plateaus) cover 6.636 million square kilometers, accounting for 69.1% of the country's total land area in China. With the development of the national economy, more and more infrastructure construction projects are located in the "mountainous areas". Restricted by the topography of mountainous areas, almost all of these projects are located in the transition zone between the mountain front accumulation area and the slope with great topographic relief. A large number of dense high-rise buildings (structures) are arranged on the narrow engineering site. The constraints of production process conditions make the problem of "high fill and deep excavation" very prominent. Because of such problems, most of the high fill slopes adopt retaining walls for support, the filler behind the wall is mostly coarse-grained filler mixed with earth rock excavated in situ. The nature of the filler behind the wall directly restricts the stability calculation and control of the retaining wall, and affects the design, safety of the retaining wall and the final engineering cost of the project. There are more in-depth studies on the properties of rockfill and coarse-grained soil at home and abroad [1] [2], but there are few studies on the physical and mechanical properties of the mixed filler behind the retaining wall. As the backfill behind the retaining wall, the "mixed filler" excavated in place in mountain area has the characteristics of local materials, low cost, good compactness, strong water permeability, large shear strength, small settlement deformation and high bearing capacity. The mixed filler excavated in mountainous area belongs to coarse-grained soil, and particle breakage is one of its important characteristics [3]. Particle breakage will change the gradation of particles before and after stress, and affect the strength and deformation characteristics of particles. Marsal et al. [4-9] studied the problem of coarse particle crushing, and achieved good research results. Based on the research of others, the main
factors of particle breakage and physical and mechanical properties of mixed filler behind retaining wall in a mountainous area are studied by large-scale triaxial shear test.

2. Test scheme
The YSZ-200 triaxial shear test equipment developed by the experimental center of Northwest Institute of Water Resources of the Ministry is used (Fig.1). The sample size is φ300 ~ 600mm, the maximum axial load is 200kN, the maximum confining pressure is 5.0MPa, the axial deformation rate is 0 ~ 30mm / min, and the maximum axial stroke is 300mm. The size of the test sample is 300 mm in diameter and 600 mm in height. The sample is compacted by three layers of ramming, and the shear rate is 0.2 mm/min.

The coal measure strata of Longtan Formation of Permian in Southwest China are selected as the test materials. The original rock lithology changes greatly, and the main components are mudstone and sandstone. In the test, the similar grading method is used to reduce the scale of raw materials (the scale ratio is 1/5). The oversized particles are replaced by particles of more than 5mm. The maximum particle size is not more than 80mm, and the content of particles of less than 5mm is unchanged. See Table 1 for the simulated grading of samples, and table 2 for the percentage content of particle size under all levels of matching. In the test scheme, the weight ratio (W mudstone / W sandstone = 3/7, 5/5, 7/3), water content (natural, saturated state), confining pressure (σ3 = 100, 200, 300, 400kPa) and drainage conditions (undrained, drained) are considered, and 8 groups of samples are configured as shown in Table 3.

Table 1. Simulated grading of sample

| Mass percentage content of different particle groups (mm)% | d_60 | d_30 | d_10 | Cc  | Cu  |
|----------------------------------------------------------|------|------|------|-----|-----|
| 80~60 60~40 40~20 20~10 10~5 <5                         |      |      |      |     |     |
| 1 2.9 4.4 8.2 13.5 34.1 36.9                              | 14.00| 12.00| 3.00 | 3.40| 4.70|
| 2 2.9 3.4 12.1 21.6 23.6 36.4                              | 7.90 | 3.95 | 1.12 | 1.76| 7.05|
| 3 1.2 0.8 2.9 79.9 4.6 10.6                                | 10.00| 3.80 | 0.70 | 2.06| 14.29|

Note: D represents the diameter of the sample
Table 2. Test scheme

| number | W<sub>mudstone</sub>/W<sub>sandstone</sub> | Gradation | Relative density | Water content |
|--------|---------------------------------|-----------|-----------------|--------------|
| JP1    | 3:7                             | common    | 0.65            | nature       |
| JP2-1  | 5:5                             | common    | 0.65            | nature       |
| JP2-2  | 5:5                             | common    | 0.65            | saturated    |
| JP3-1  | 7:3                             | common    | 0.55            | nature       |
| JP3-2  | 7:3                             | common    | 0.65            | nature       |
| JP3-3  | 7:3                             | common    | 0.80            | nature       |
| JP4    | 3:7                             | good      | 0.65            | nature       |
| JP5    | 3:7                             | bad       | 0.65            | nature       |
| JP2-3  | 3:7                             | good      | 0.90            | saturated    |

Table 3. Particle size distribution and fracture rate of the sample after shearing under different proportion of mudstone and sandstone

| Proportion of mudstone and sandstone | Test condition | confining pressure /kPa | Mass fraction of granular soil in each particle size group /% | Bg/% |
|-------------------------------------|---------------|--------------------------|-------------------------------------------------|------|
| 3:7 (JP1)                           | Before test   | 2.9 4.4 8.2 13.5 34.1   | 36.9 | 3.7 (JP2-1)                           | After test   | 1.2 3.5 5.6 8.5 22.7 | 55.3 | 21.6 |
|                                    | 100           | 2.4 4.0 6.3 9.5 18.7    | 58.5 | 200           | 1.9 3.2 5.5 8.6 18.4 | 62.4 | 25.5 |
|                                    | 300           | 1.4 3.8 6.0 8.0 17.5    | 63.3 | 400           | 0.8 3.7 5.4 8.0 19.4 | 62.7 | 26.8 |
|                                    | After test    | 1.2 2.6 5.7 8.8 21.4    | 60.3 | 200           | 1.2 2.4 5.6 8.2 21.4 | 61.2 | 24.3 |
|                                    | 300           | 1.2 2.4 5.5 7.7 21.7    | 61.5 | 400           | 1.2 2.4 5.4 8.4 24.0 | 58.1 | 22.2 |
| 5:5 (JP2-1)                         | Before test   | 2.9 4.4 8.2 13.5 34.1   | 36.9 | 3.7 (JP2-1)                           | After test   | 1.2 3.5 5.6 8.5 22.7 | 55.3 | 21.6 |
|                                    | 100           | 2.4 4.0 6.3 9.5 18.7    | 58.5 | 200           | 1.9 3.2 5.5 8.6 18.4 | 62.4 | 25.5 |
|                                    | 300           | 1.4 3.8 6.0 8.0 17.5    | 63.3 | 400           | 0.8 3.7 5.4 8.0 19.4 | 62.7 | 26.8 |
|                                    | After test    | 1.2 2.6 5.7 8.8 21.4    | 60.3 | 200           | 1.2 2.4 5.6 8.2 21.4 | 61.2 | 24.3 |
|                                    | 300           | 1.2 2.4 5.5 7.7 21.7    | 61.5 | 400           | 1.2 2.4 5.4 8.4 24.0 | 58.1 | 22.2 |

3. Analysis of test results

3.1. Analysis of mixed fillers particle breakage rate

Because the particle size of the mixed filler behind the wall is large, it is easy to break under the action of external load, and its breaking degree has a great influence on the mechanical properties of the mixed filler. The crushing degree is usually expressed by the particle crushing rate, and the commonly used expression method is the coarseness coefficient KC proposed by Si Hongyang [2], which is expressed by the ratio of the average particle size d to the boundary particle size d<sub>c</sub> multiplied by the mass fraction of the coarse material; the difference of the control particle size d<sub>60</sub>-d<sub>60f</sub> is proposed by the Chinese Academy of water resources [4]. Hardin [5] proposed that the crushing amount B<sub>f</sub> and the relative crushing rate B<sub>r</sub> should be used to express; Marsal [6] proposed that the particle crushing rate B<sub>g</sub>, which is defined as the sum of the positive values of the difference between the content of each coarse particle group before and after the test, that is, for the soil samples with the same grading, the absolute value of the sum of the difference between the content of each coarse particle group <span class="math" id="eq:1">\Delta W_k</span>, is the crushing rate B<sub>g</sub>, and the numerical range is 0-1, that is

\[
B_g = \sum \Delta W_k = W_{ki} - W_{kf}
\]

In the formula: W<sub>ki</sub> is content of a certain particle group on the grading curve before the test, W<sub>kf</sub> is
content of the same particle group on the grading curve after the test.

This test adopts the calculation method of particle crushing rate proposed by Marsal. After each triaxial test, dry the sample and re-screen it to get the crushing rate. Under different shale content, gradation and relative density, the particle size distribution and crushing rate of the sample after shearing under various confining pressures are shown in Table 3.

Through the analysis of the relationship between proportion of mudstone and sandstone and the breakage rate after the shear test (Fig. 2), it can be seen that with the decrease of sandstone content, the mudstone content increases, the mudstone that is cut by sandstone decreases, and the crushing effect decreases.

Through the analysis of the relationship between particle crushing rate and gradation after shear test (Fig 3). It can be seen that as the grading changes from good to bad (the nonuniformity coefficient decreases), the filling and dense contact between particles of different sizes gradually transit to the angular contact between coarse particles, and the angular contact is easy to produce stress concentration phenomenon, and the crushing effect is significant. Poor grading will lead to a significant increase in particle breakage.

Through the analysis of the relationship between particle breakage and relative density after shear test is analyzed (Fig 4). It can be seen that with the increase of relative density, the overall performance of rock mass becomes better, the particle crushing effect is weakened, and the crushing is reduced.2.2 Analysis of shear strength of mixed filling

The large-scale triaxial shear test is carried out for the test scheme listed in Table 2. The stress-strain curve (Fig.7) and Mohr circle (Fig.8) of the sample under each test condition are drawn. The shear strength value of the test under the condition of unconsolidated drainage - internal friction angle (φu) and cohesion (Cu) are calculated. The results are shown in Table 6, and the analysis is as follows:

It can be seen from Figure 5 that the stress-strain curve of this sample shows strain hardening
phenomenon, without obvious peak value, which may be related to the low stress state of the sample. The other samples are the same as JP1.

![Mohr circle of sample JP1](image1)

**Figure 6. Mohr circle of sample JP1**

![Relationship between mudstone content and internal friction angle and cohesion](image2)

**Figure 7. Relationship between mudstone content and internal friction angle and cohesion**

| Sample number | W_mudstone/W_sandstone | relative density | $c_u$ (kPa) | $\phi_u$ (°) |
|---------------|-------------------------|-----------------|------------|--------------|
| JP1           | 3:7                     | 0.65            | 49.24      | 28.0         |
| JP2-1         | 5:5                     | 0.65            | 85.42      | 27.0         |
| JP3-1         | 7:3                     | 0.55            | 73.35      | 25.0         |
| JP3-2         | 7:3                     | 0.65            | 75.72      | 28.0         |
| JP3-3         | 7:3                     | 0.80            | 79.78      | 30.5         |
| JP4           | 3:7                     | 0.65            | 55.49      | 27.0         |
| JP5           | 3:7                     | 0.65            | 41.28      | 27.0         |

According to Fig.7, with the increase of mudstone content, the cohesion increases to a certain extent and then decreases. For the coarse-grained soil composed of sandstone and mudstone, the sandstone generally shows no viscosity, and the cohesion is mainly affected by the content of mudstone. At the same time, the bite force between particles also affects the shear strength. The larger the bite force is, the larger the shear strength is. When the bite force is greater than the ultimate strength of the particles, the particles break. As the mudstone particles break before the sandstone particles, the fine particles move each other, resulting in the decrease of the bite force between the particles and the shear failure of the soil. According to figure 9, the internal friction angle of the test sample decreases gradually under different mud and sandstone ratios, which is due to the following reasons. In the case of the same relative density and particle grading, the roughness between particles determines the internal friction angle within a certain range.

![Relationship gradation with internal friction angle and cohesion](image3)

**Figure 8. Relationship gradation with internal friction angle and cohesion**

![Relation curves of relative density with cohesion and Angle of internal friction](image4)

**Figure 9. relation curves of relative density with cohesion and Angle of internal friction**

With the gradual improvement of particle size distribution, the interparticle filling density increases,
the particle breakage rate decreases, and the cohesion and internal friction Angle increase. 

According to Fig. 9, the strength parameter of the sample increases with the increase of the relative density. When the relative density is larger, the soil particles are more closely packed, and their mutual bite force is also larger, so the cohesive force and internal friction Angle of the mixed filler increase with the relative density.

4. Comprehensive prediction of internal friction Angle

The internal friction Angle and cohesion c values of the strength index of the mixed packing are converted to the comprehensive internal friction Angle \( \phi_0 \). When the comprehensive internal friction Angle is large, the filling pressure behind the retaining wall is small, which is conducive to the retaining wall stability. We can:

\[
\phi_0 = \tan^{-1}(\tan \phi + \frac{c}{\sigma})
\]

In the formula: \( \sigma \) is normal stress at the wall heel, \( \sigma = \gamma H \), where \( \gamma \) is the bulk density of the soil, and \( H \) is the vertical design height of the retaining wall.

In this test, the comprehensive internal friction Angle of the mixed packing \( \phi_0 \) by mud, sandstone proportion, relative density, particle size distribution and other factors together, the constructor function:

\[
\phi_0 = f(X_1, X_2, X_3)
\]

In the formula: \( X_1 \) is percentage of mudstone in test material (%), \( X_2 \) is relative density \( Dr \) of test material, \( X_3 \) is coarse grain content of test material (%).

\[
X_1 = [0.30, 0.50, 0.70, 0.30, 0.30, 0.30, 0.70, 0.70, 0.70];
X_2 = [0.65, 0.65, 0.65, 0.55, 0.65, 0.65, 0.65, 0.65, 0.65];
X_3 = [0.631, 0.631, 0.631, 0.631, 0.631, 0.631, 0.994, 0.631, 0.636];
\phi_0 = [33.7, 34.2, 34.3, 31.8, 34.3, 36.6, 35.7, 33.7, 38.0]
\]

MATLAB software was used to establish a multivariate nonlinear regression model [11] to reflect the relationship between the comprehensive internal friction Angle \( \phi_0 \), ratio of mud and sandstone \( X_1 \), relative density \( X_2 \) and coarse grain content \( X_3 \), and to fit the data function. At the same time, the significance of the regression model was detected. According to the predicted value of the regression function at \((X_1, X_2, X_3)\), residual analysis was performed and residual and confidence interval were drawn.

By analyzing the variation rule of data, the functional relations between \( f \) and \( X_1, X_2 \) and \( X_3 \) are deduced as follows:

\[
f_1(X_1) = a + b \ln X_1; f_2(X_2) = a + b X_2; f_3(X_3) = a + b X_3 + c X_3^2
\]

Therefore, the regression function model is:

\[
Y = a + b \ln X_1 + \frac{c}{X_2} + d X_3 + e X_3^2
\]

According to MATLAB fitting, the values of each parameter are:

\( a=-1674.3, \ b=-0.1, \ c=-8.5, \ d=4648.8, \ e=-3044.3 \)

By combining the curves of the experimental value of internal friction Angle with the predicted value (Fig. 10) and the residual diagram of the regression model (Fig. 11), it can be seen that the predicted regression model is consistent with the original data, the residual of all data is close to the zero point, and the confidence interval of the residual passes the zero point.

Fig. 10 comparison of experimental and predicted values

Fig. 11 residual diagram of the regression model
It can be seen from the figure that in this test, when the ratio of mud and sandstone is 7:3, the relative density is 0.65, the coarse particle content is 63.6%, and the gradation is in good condition (Cc=2.06, Cu=14.29), the comprehensive internal friction Angle of soil filling is large, the soil pressure behind the corresponding wall is small, and the engineering safety is high.

5. conclusion
Particle crushing rate is an important index to describe coarse particle mixed packing. The results of this paper show that the higher the mudstone content in coarse particle mixed packing is, the smaller the crushing rate is. With the gradual improvement of gradation, the particle breakage rate decreased significantly. The higher the relative density, the smaller the crushing rate. The particle breakage rate increases with the increase of confining pressure and stress. The mechanical properties of coarse particle packing can be effectively analyzed by large triaxial shear test. The test results show that the stress-strain relationship of the coarse particle packing is a strain-hardened type when the coarse particle packing is mixed from 100kPa to 400kPa. The mudstone content has obvious influence on the shear strength index, the grading and the relative density have obvious influence on the strength index, and the regularity is strong.

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