



# An approach to predicting the shear strength of soil-rock mixture based on rock block proportion

Zhenping Zhang<sup>1,2</sup> · Qian Sheng<sup>1,2</sup> · Xiaodong Fu<sup>1,2</sup> · Yongqiang Zhou<sup>1,2</sup> · Juehao Huang<sup>1,3,4,5</sup> · Yuxiang Du<sup>1,2</sup>

Received: 17 April 2019 / Accepted: 25 October 2019 / Published online: 20 December 2019  
© Springer-Verlag GmbH Germany, part of Springer Nature 2019

## Abstract

Soil-rock mixture (SRM) shows complicated mechanical behaviors due to their complex compositions and structures, leading to challenging instability problems during the construction process. Typical SRM are composed of rocks with high strength and fine grained soils, and the mechanical characteristic is largely controlled by the rock block proportion (RBP) and component properties. It is noted that the rock sizes of natural SRM make it difficult for laboratory or in situ tests. There are few studies on empirical formulas to predict the mechanical characteristics of SRM. In this study, the nonlinear relationship between SRM shear strength and RBP was investigated, and an empirical formula predicting the shear strength of mixtures consisted of strong rocks and a weak soil matrix was proposed. For this purpose, a database of shear strength and uniaxial compressive strength (UCS) of SRM with different RBPs was built firstly on the basis of the laboratory test results from previous literatures. In order to focus on the interactions of rock blocks and soil matrix in SRM, a RBP range of 30–90% was set as the applicable range of the empirical formula and both of the compositions are held to provide shear resistance in the applicable range. Subsequently, a nonlinear equation to calculate the shear strength of SRM with RBP range of 30–90% was proposed using regression analysis considering the strengths of components and soil-rock contact faces. Several representative properties of rocks and soil matrix, such as RBP, UCS of the matrix ( $UCS_m$ ), and the friction angle of the blocks ( $\varphi_{\text{block}}$ ), were chosen as the input parameters based on the mechanical properties of SRM. An additional parameter “A” was used to describe the connect strengths of the soil-rock contact faces. In addition, uniaxial compression tests and large-scale direct shear tests were performed on the Taoyuan SRM samples. The test results and other measured data from the database were used to compare with the corresponding estimated values. The results demonstrated that the empirical approach could predict the shear strength with  $R^2 = 0.75$  and can be considered a practical tool in engineering designs when mechanical tests are not available.

**Keywords** Soil-rock mixture · Empirical formula · Strength characteristics · Rock block proportion

---

✉ Qian Sheng  
shengqian@whrsm.ac.cn

Zhenping Zhang  
zhangzhenping16@mails.ucas.edu.cn

Xiaodong Fu  
xdfu@whrsm.ac.cn

Yongqiang Zhou  
yqzhou@whrsm.ac.cn

Juehao Huang  
jhhuang@whrsm.ac.cn

Yuxiang Du  
duyuxiang171@mails.ucas.edu.cn

<sup>1</sup> State Key Laboratory of Geomechanics and Geotechnical Engineering, Institute of Rock and Soil Mechanics, Chinese Academy of Sciences, Wuhan 430071, China

<sup>2</sup> School of Engineering Science, University of Chinese Academy of Sciences, Beijing 100049, China

<sup>3</sup> University of Chinese Academy of Sciences, Beijing 100049, China

<sup>4</sup> Hubei Key Laboratory of Geo-Environmental Engineering, Wuhan 430071, China

<sup>5</sup> The Soft Soil Research Center, State Key Laboratory of Geomechanics and Geotechnical Engineering, Ningbo University of Technology, Ningbo 315211, China

## Introduction

Soil and rock mixture (SRM) is one of typical anisotropy geo-materials existing in many construction projects, such as tunnels, dams, and slopes. The complex shear behavior of the SRM will cause challenging problems, such as instability problems and project delays, during design and construction of structures in or on them. Determining the strength parameters of geo-materials is a crucial prerequisite for evaluating the safety of engineering works in the design stage. The strength parameters could be characterized through mechanical tests and numerical methods. The complex geology environments add difficulties to in situ tests. Laboratory tests of SRM put forward a higher requirement for the dimension of test equipment. The disturbance of sampling and limitations of specimen size reduce the reliability of laboratory tests, and unrepresentative mechanical characteristics may be obtained. In contrast, the empirical method is much easier and more suitable to obtain the objective parameters in engineering projects. The empirical formula is one of common tools to obtain the material properties based on the test results or other data from previous studies with a reference of classification systems and engineering experience. In recent studies, the strength predictions of geo-materials have been one of the most attractive questions, and most of these predictions are focused on jointed rock masses. There have been fewer attempts to study the strength predictions of special complex geo-materials composed of rock blocks and soils. The strength of chaotic mixtures is controlled by the compositions and internal structures, which brings about the uncertainty to the mechanical properties and unknown parameters used in the empirical method. Because of the heterogeneous components and difficulties encountered in mechanical mechanism analyses, the prediction of the strengths of the mixed geological rock blocks, such as *mélange*, weathered fault rock, coarse pyroclastic rock, and agglomerate, is still a creative research to be explored.

Laboratory tests on SRM show that these types of inhomogeneous mixtures are consisted of high-strength rock blocks (e.g., sand, gravels, boulders, and bobbles) and fine grained particles. The considerable strength or stiffness contrast between rock blocks and soils is a remarkable feature of SRM. Lindquist (1994) suggested that the criterion ratio of stiffness between two components should be 2.0 and that the recommended criterion ratio of the uniaxial compressive strength (UCS) is 1.5. Different contents of rock blocks (or soils) would change the structure inside and load transfer mechanism because of the strength contrast between rock blocks and soils, resulting in different mechanical characteristics. Thus, rock block proportion (RBP) is a key factor to the shear strength of the mixture. Over the past few years, a number of tests have been devoted to characterizing the influence of RBP on strength and deformation. Zhao et al. (2016) adopted five types of composite soil samples for uniaxial compressive tests

and found that the disturbance of specimen integrity and increase of weak soil-rock contact faces attributed to the increase of RBP leads to a decrease in uniaxial compressive strength (UCS). Similar conclusions were obtained by Kahraman and Alber (2006). Several researchers have investigated the effect of RBP on the SRM shear strength. Donaghe and Torrey (1979) found that the friction angle of SRM increased with an increasing RBP by performing triaxial compression tests. Wang et al. (2018) suggested that the strength of geo-material with 50% RBP was much higher than those of other contents. The results also indicated that the interlock effect could improve the strength of soil-rock contact faces and the overall material. Holtz and Willard (1956) focused on the shear resistance of gravel-clay mixtures and the shear strength showed an incremental trend with RBP varying from 0 to 65%. Gao et al. (2014) found that the internal friction angle and cohesion were strongly affected before the critical RBP by weight threshold of 70%. The large-scale direct shear tests results have revealed that when RBP changed from 25 to 70%, the rock blocks inside could enhance the strength of the entire material and improve its geo-mechanical advantage (Lindquist and Goodman 1994; Xu et al. 2011). Vallejo and Zhou (1994) noted that the shear strength of mixture of kaolinite clay and sand was governed by clay matrix when the sand content is less than 50%, and the sand fraction would control the shear behavior while the sand content was more than 80%. Laboratory tests conducted on mixtures of glass beads by Vallejo (2001) showed that when the concentration of the large beads was between 40 and 70%, the shear strength was controlled by the large beads partially. Dupla et al. (2007) indicated that the volumetric fraction of gravel had an impact on the material failure characteristics of coarse-grained soils. The laboratory investigations performed by Wickland (Wickland et al. 2006) showed that a desirable RBP could fill the void space in the waste rocks based on particle density theory and produced the maximum density mixture and strength. These studies above show that the increment of RBP would have different influences on strength at different stages, and there are obvious RBP thresholds in strength variations. On the one hand, rock blocks could provide friction and strength for the overall material. On the other hand, rock content increasing would lead to the increase of soil-rock contact faces. In SRM, the contact faces are known to be the one of important factors controlling failure behavior. The weak soil-rock contact face is prone to break and accelerate the crack propagations inside the material under external loading, finally leading to the destruction of the entire specimen via a tortuous shear band. The strength of contact faces could be related to the combination of the two components. Hence, the mechanical characteristics of the SRM are closely related with the properties and proportions of the two components.

It should be noted that the large-dimensional equipment with loading system is necessary for mechanical tests of

SRM contained a large gravel particles. Empirical method could provide strength parameters more conveniently, using a few parameters based on material classifications or simple mechanical test results. The most important aspect of fitting the empirical formula is to select the appropriate input parameters. The practicability of the predicted formula increases when the input parameters are easy to obtain. The existing prediction equations are mainly applicable for jointed rock masses based on strength failure criteria proposed by Hoek and Brown in 1980 (Tian et al. 2018; Ulusay and Erguler 2012; Catane et al. 2008). Different kinds of rock mass classifications have been developed considering other influence factors, such as weathering degree (Basu et al. 2009; Dasaka and Zhang 2012), surface morphology (Usefzadeh et al. 2013; Singh and Basu 2016), and discontinuities. The Geological Strength Index (GSI) proposed by Hoek has also been continually improved (Hong et al. 2017). More attentions need to be paid on the predictions of strength of SRM, in order to generalize an empirical formula similar to those for jointed rock blocks mixed with soil.

According to the fundamental laws proposed by Weibel in 1980, the value of RBP could be collected through convenient observations such as linear (such as scanlines or drill core) or areal (outcrops, geological maps, and image analysis) measurements. Therefore, the RBP of a debris or colluvium slope could also be measured. The test data collected from other SRM studies have showed that there is a nonlinear relationship between shear strength and RBP. Kalender et al. (2014) introduced an empirical approach that obtained the strength of bimrock for contents from 0 to 100% based on the model test data of Lindquist (1994), Alstinsoy (2006), and Coskun (2010). However, the interlock effect between rock blocks in SRM with a high RBP is obvious, and the material could be addressed as a jointed rock mass mixed with soil. The shear resistance is almost provided by the rock blocks. The mixture with a low RBP is composed of the soil matrix with a few gravel particles floating inside, and the mechanical properties are partial to the soil matrix. Thus, the formula to predict shear strength of SRM with the entire RBP range is not necessary and may lead to a loss of accuracy. A suitable range of RBP is necessary to choose to identify the shear strength supported by rock blocks and soil matrix both.

In this study, an empirical approach to estimate SRM shear strength based on its RBP was proposed on the basis of the mechanical properties of rock blocks and soil matrix. First, the strength data of laboratory and in situ tests were collected from previous studies to build a SRM strength parameter database. Second, an applicable range of RBP was set to study the SRM strength properties provided by rock blocks and soil matrix both. Then, a nonlinear formula was suggested for the estimation of the shear strength of SRM with different RBPs. The properties of components and soil-rock contact faces were also considered in determining the input parameters of

formula to describe its influence on the overall geo-material. Several uniaxial compression tests and direct shear tests were performed, and the prediction accuracy of the empirical approach was validated with laboratory tests and data from the database, which proved its applicability and practicability.

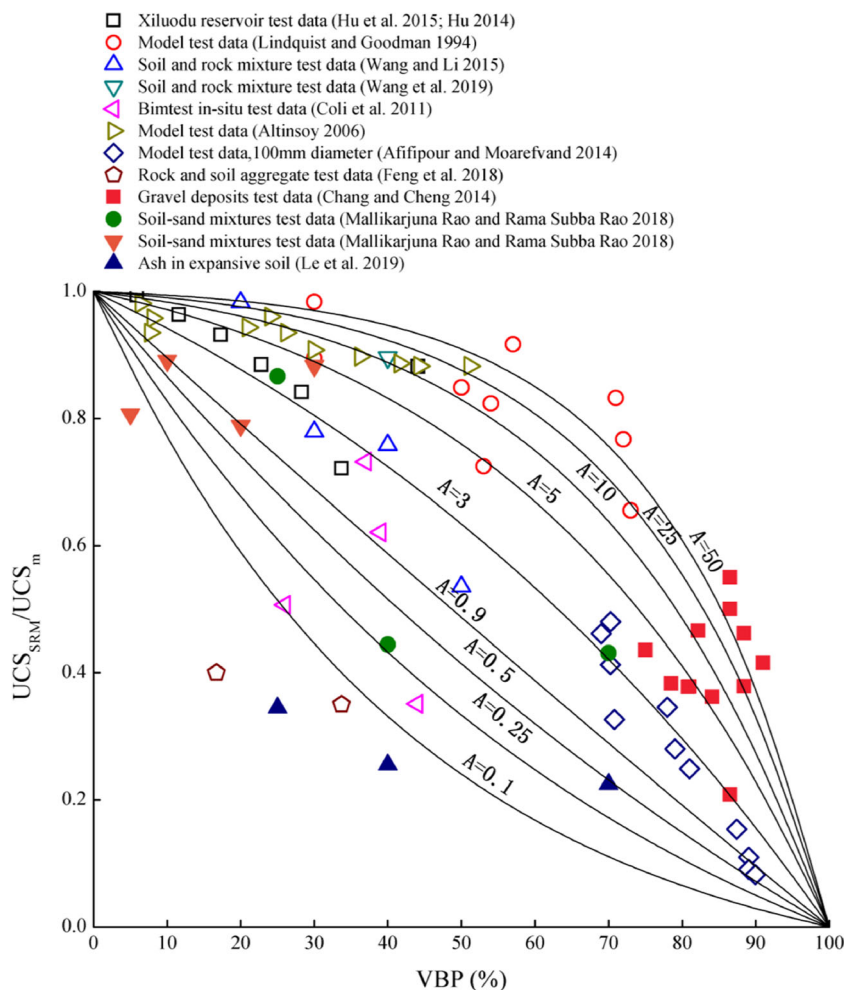
## Mechanical behavior and database of SRM

The empirical approach to predict SRM shear strength in this paper was established on a large number of test data as the foundation to ensure accuracy and universal applicability. The literature review presents that SRM mechanical behaviors show complex variation trends under the affection of several factors and RBP is the most remarkable influence factor. Most of laboratory and in situ tests also focused on the effect of RBP on the mechanical properties and failure mechanism of SRM, which is the easiest variable to control in mechanical tests. Therefore, an SRM database of the shear strength and UCS of SRM with different RBPs is expected to be built in this study.

The SRM samples in database were mostly collected from the large-scale accumulated bodies in southwestern China. The test data from other regions such as shale-limestone chaotic mixtures from Italy (Coli et al. 2011) and gravel deposits around central Taiwan, China (Chang and Cheng 2014) were also involved in database. In fact, most of the SRM samples share the similar fabric, characterized by a fine-grained matrix, often constituted by silty sandy or clay, containing floating rock blocks. It is noted that the types of matrix are similar for natural SRM samples in databases based on the geology summary. The matrix of mixtures is mainly composed of gray-clay or sand, with the value of UCS smaller than 30 MPa. The rock fragments are mainly 2.0–60 mm in size, and are composed mostly of irregular-shaped strongly weathered limestone, mudstone, and sandstone. The tests of volcanic soils and artificial aggregates were also selected. The connection strengths (bonds) between soil matrix and rock blocks are relatively low, which bring about the initial failure occurrences of contact faces under loading. Considering the scale limit of test apparatus to avoid scale effect, the maximum particle size SRM material in this paper was smaller than 60 mm. A summary of information about the UCS and shear strength database established in this study is given as follows.

The UCS values of the soil matrix and SRM with different RBPs are both collected for the analysis. The normalized UCS value was proposed as the ratio of the UCS value of SRM and soil matrix, which could decrease the strength gaps due to the soil matrix properties. Figure 1 shows the normalized UCS values of SRM samples with different volumetric block proportions (VBPs), and it is clear that UCS decreases nonlinearly as the VBP increases. However, Sonmez et al. (2006) presented an opposite trend of welded volcanic

**Fig. 1** Normalized UCS<sub>SRM</sub> with different VBPs

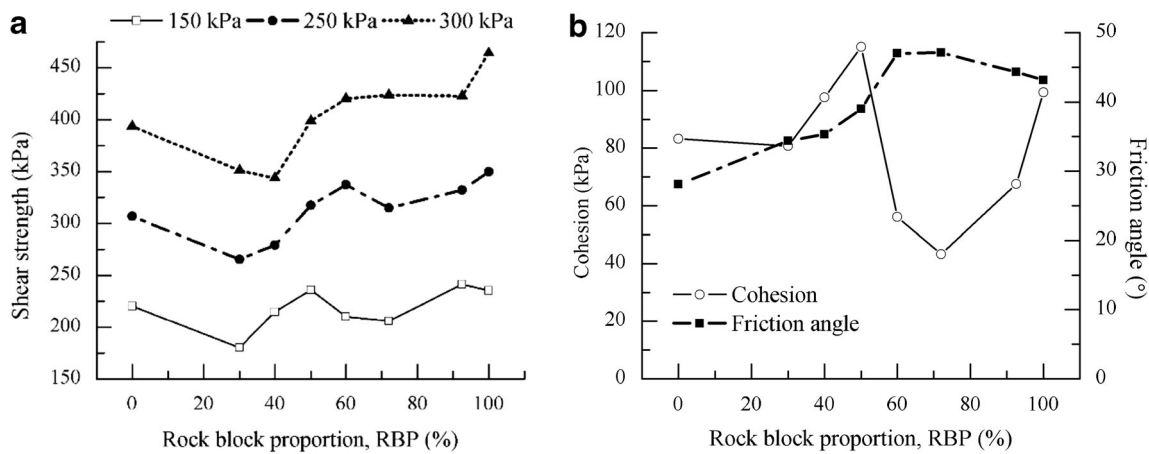


bimrocks with an additional VBP. The difference could be explained by the source of strength of welded bimrocks. The connect strength of soil-rock contact faces in welded bimrock is much stronger than that of natural SRM, and VBP increasing provides a strength supply for the overall material. In this study, the natural SRM samples were chosen and the discrepancies in UCS<sub>SRM</sub> descending degree between different SRM samples could be found in Fig. 1. In addition, the failure morphology for SRM specimens was also discussed, which suggested that the number of main cracks and secondary cracks surrounding the rock blocks increase.

The information acquired from the uniaxial compression tests above revealed that the normalized UCS<sub>SRM</sub> is relevant with VBP and component properties. The failure patterns with different VBPs show that the strength of weak contact faces could control crack propagations and affect the shear resistance of overall geo-material. In summary, UCS<sub>SRM</sub> has a non-negligible relationship with mechanical characteristics of components, such as different shapes, roughnesses, and friction (repose) angles of rock blocks and other mechanical parameters of soil matrix. The shape and roundness of rock blocks are relative qualitative factors. In order to reduce

uncertain variables and human factors in calculating UCS<sub>SRM</sub>, the shape and roundness of the rock blocks could be replaced with the friction or repose angles.

The shear strength of SRM in database has an obvious nonlinear association with the changes in RBP. The laboratory shear test data performed by Hu in 2014 were shown as an example in the study to analyze the variations of SRM shear strength and mechanical parameters of SRM. The RBP was set as rock content data, which is defined as the ratio of the rock block mass to the total mass of SRM. Figure 2 reflects that the shear strength increases in a similar wave trend with the increase of RBP under different normal stresses. It is noted that the increase stage starts at around RBP = 30% when the rock block started to carry the shear force with soil matrix together. The corresponding RBP at the end of strength-ascending stage is positively correlated with the normal stress. A higher normal stress such as 300 kPa results in an increase of the compaction and friction between rock blocks and soil matrix effectively. Therefore, the shear strength kept rising with RBP larger than 40% under 300 kPa. It is found that there is a decline period after the RBP is larger than 50~60% under 150 kPa and 250 kPa. The reason for the decline period is that



**Fig. 2** The relation between **a** shear strength and **b** mechanical properties and the RBP (Hu 2014)

the replacement of soil matrix with rock blocks would increase the number of soil-rock contact faces and reduce the cohesion of SRM provided by the soil matrix. Similar test conclusions were presented by Xu et al. (2008) and Chang and Phantachang (2016).

Figure 2 b reveals that 30% and 70% are two key RBP for SRM and there are clear turning points in strength parameters of SRM when the RBP was between 30 and 70%. For the low-RBP (RBP = ~30%) and high-RBP (RBP = ~70%) SRM, the increment of RBP had a slight impact on the cohesion and friction angle. The rock blocks inside the SRM change from floating into framework-forming during the variation of RBP, which results in the substitution of rock blocks for soil matrix and improves the friction angle of the overall SRM. However, a large number of soil-rock contact faces are also created in the process. The weak strength of contact faces would accelerate the initiation and propagation of cracks. Therefore, RBP has positive and negative impacts on the shear strength. In addition, the differences in the strength among different SRM samples demonstrate that the properties of components also have a decisive effect on the mechanical properties and should be considered in predicting the strength of SRM.

### An empirical approach to predicting the shear strength of SRM

According to the variation characteristics of strength behaviors of SRM discussed above, there are obvious RBP thresholds for the shear strength dominator material. Soil matrix with different RBPs may form different soil skeleton structures and micro-porosities and hence present different stress-strain behaviors. For the low-RBP mixtures (RBP is less than 25–30%), a small number of rock blocks are floating in the matrix discretely. Shear resistance of the geo-material is mainly attributed to the soil matrix, and slight change could be

found under the various RBP. When the RBP increases over than a high bound (70–90%), SRM would transfer into heavily broken rock mass mixtures, with angular or rounded rock pieces inside. An applicable range of RBP as 30–90% was set in this study to describe the SRM strength provided by rock blocks and soil matrix simultaneously. The RBP range was proposed based on the variation regularity of shear strength in the database and mechanical mechanisms from the literature reviews. The mechanical properties of the geo-material with RBP lower than 30% have many similarities to the pure soil matrix, and the shear strength could be calculated based on the Mohr-Coulomb criterion. The material strength with more than 90% RBP could be obtained based on Hoek-Brown criterion (Hoek et al. 2002). Therefore, the empirical approach is only for SRM with RBP between 30 and 90%, which focuses on the coupled influence of the rock blocks and soil matrix on the mechanical properties of SRM. In this approach, SRM with all range of RBP could be calculated without large-scale laboratory and in situ tests.

The structures of natural SRM are usually complicated and anisotropy. The angular rock blocks of SRM with low RBP are floating in the soil matrix and poorly interlocked. The behavior of SRM with a high RBP is similar to the behavior of block/disturbed or disintegrated jointed rocks, and the rock skeleton mostly controls the shear resistance (Khalili et al. 2010). The mechanical characteristics of the latter are usually modeled using the Hoek-Brown criterion, which has gained a wide acceptance in estimating the strength characteristics of rock masses (Hoek et al. 2002; Hoek and Brown 2018). Hoek-Brown failure criterion is one of the most common criterions to estimate the mechanical behavior of typical rock masses. Both intact rocks and discontinuities are characterized based on Geological Strength Index (GSI). The nonlinear Hoek-Brown criterion provides a reference for the empirical formula of SRM. In this study, UCS values of SRM with various VBP is also necessary for the calculation by reference to the Hoek-Brown criterion.

### The empirical formula to predict UCS<sub>SRM</sub>

Kalender et al. (2014) developed a preliminary approach to predict the UCS of bimrock based on laboratory results and other studies. The formula is as follows:

$$UCS_{bim}/UCS_m = \frac{A - A \frac{VBP}{100}}{A - 1} \tag{1}$$

In Eq. 1, UCS<sub>m</sub> and UCS<sub>bim</sub> are the uniaxial compressive strength of the soil matrix and bimrock, respectively. The parameter “A” is an important variable controlling the declining trend of strength with the addition of VBP. To keep the formula calculable, “A” is selected to be 0.999 instead of 1. Figure 1 shows that the curve with a higher “A” has a smaller decreasing degree during the preliminary and middle increasing periods of VBP. As can be seen, a high value of “A” means that the influence of VBP increasing on UCS<sub>SRM</sub> is small, and the contact faces could offer a part of strength for the overall material. Thus, parameter “A” could be defined as a variable to represent the strength of the soil-rock contact faces, which could also be called as the connect strength between the rock blocks and soil matrix. The strength of contact faces in unwelded SRM is weak due to the strength contrast between components and is highly dependent on the composition properties, such as the type of matrix and the angularity and roundness of rocks. In this paper, the compressive strength of the matrix and the roundness (or repose angle) of rocks are used to determine corresponding parameter “A.”

### The empirical formula to predicting the shear strength of SRM

According to the literature reviews above, the variation trend of SRM shear strength is mainly controlled by RBP, and the strength of soil matrix and rock block also affects the shear strength to a certain extent. Therefore, the influence factors mentioned above should be involved in the prediction formula. More than 100 groups of SRM mechanical test data were used to establish the empirical formula. Even though other SRM properties, such as particle distribution and density, have some associations with SRM strength, the relevant test data is limited and inaccurate results might be obtained. To improve the practicability of the formula and reduce unknown input parameters as much as possible, the formula to predict the shear strength behavior of SRM with different RBPs was studied. Properties of soil matrix and rock blocks were involved during deriving the formula to exhibit the influence of both components.

Considering the similarity between SRM and jointed rock masses, the shear strength failure criterion for jointed rock masses could be utilized as a reference for SRM. Traditional Hoek-Brown criterion is expressed in terms of the major and

minor principle stresses, and the Mohr envelope is related to normal stress and shear strength. Therefore, a statistical curve fitting process was used to derive an equivalent Mohr envelope defined by the equation (Hoek and Brown 1997):

$$\tau = A\sigma_{ci} \left( \frac{\sigma}{\sigma_{ci}} - T \right)^B \tag{2}$$

where *A* and *B* are material constants,  $\sigma_{ci}$  is the uniaxial compressive strength of the intact rock pieces. *T* could be defined by the equation:

$$T = \left( m_b - \sqrt{m_b^2 + 4s} \right) / 2 \tag{3}$$

where  $m_b$  is the value of the Hoek-Brown constant *m* for the rock mass, *s* is a constant which depend upon the characteristics of the rock mass. In order to use the Hoek-Brown criterion for estimating the stress of jointed mass, three necessary properties of rock mass have to be obtained: (1) UCS of intact rock pieces; (2) the value of Hoek-Brown constant *m*; (3) the value of GSI.

The empirical formula in the paper was evaluated from the Eq. 2 given by:

$$\tau_{SRM} = D\sigma_{c(SRM)} \left( C \frac{\sigma}{\sigma_{c(SRM)}} - T \right)^B \tag{4}$$

where  $\sigma_{c(SRM)}$  is the value of UCS<sub>SRM</sub> and *C*, *B*, and *T* are parameters used to describe the influence of RBP on strength and assumed to be related with RBP to reduce calculated cost. The parameter values could be obtained by Eqs. 5 to 7:

$$C = \begin{cases} 4736.2\gamma^2 - 3237.7\gamma + 552.3 & 0.3 \leq \gamma < 0.5 \\ 6007.8\gamma^2 - 7744.8\gamma + 2487.9 & 0.5 \leq \gamma < 0.7 \\ 1928.4\gamma^2 - 2980.3\gamma + 1151.6 & 0.7 \leq \gamma \leq 0.9 \end{cases} \tag{5}$$

$$B = \begin{cases} -1.94\gamma + 1.59 & 0.3 \leq \gamma < 0.5 \\ -24.96\gamma^2 + 32.19\gamma - 9.24 & 0.5 \leq \gamma < 0.7 \\ -53.06\gamma^2 + 83.8\gamma - 31.59 & 0.7 \leq \gamma \leq 0.9 \end{cases} \tag{6}$$

$$T = \begin{cases} 846\gamma^2 - 674.6\gamma + 124.47 & 0.3 \leq \gamma < 0.5 \\ -126.9\gamma^2 + 132.08\gamma - 35.64 & 0.5 \leq \gamma < 0.7 \\ -232.18\gamma^2 + 376.47\gamma - 155.13 & 0.7 \leq \gamma \leq 0.9 \end{cases} \tag{7}$$

The  $\gamma$  in Eq. 5–7 is defined as  $\gamma = RBP/100$ . The functions of *C*, *B*, and *T* are all piecewise functions and the segmental regions of RBP are the same. The strength parameters in database show an apparent extremely high or low value at around 30% and 70% RBP, which are in accordance with piecewise threshold. The calculation range of  $\gamma$  is set from 0.3 to 0.9, in which both rock blocks and soil matrix would participate in the shear strength of SRM.

The Geological Strength Index (GSI) is a necessary index in generalized Hoek-Brown criterion and provides a system for estimating the reduction in rock mass strength by considering the surface conditions and structures of the rock masses (Hoek et al. 2002). The previous studies have shown that the mechanical properties of contact faces have dominating influence on destruction path and crack propagation. Therefore, the GSI of SRM has been improved by taking the effect of soil-rock contact face properties into consideration. The rock fragments in SRM under natural condition are disintegrated and heavily weathered mixed with grain soils. According to the introduction above the surface condition could influence the strength of contact faces and SRM both. In this paper, the qualitative description of surface condition was replaced by the roundness and friction of rocks. Except the surface condition, the property of soil matrix is the other contributing factor of contact face mechanical properties. Moreover, the number of soil-rock contact faces is controlled by RBP. As a consequence, parameter “G” is proposed as the value of the GSI of SRM with different RBPs based on the mechanical characteristics of SRM. It is noted that the parameter is related to RBP and parameter “A,” which reflect bond (contact) strength between rock blocks and soil matrix according to the definition. A practical guide for parameter “G” is illustrated in Fig. 3, which was built on the basis of back-calculated shear strength from the database. Figure 3 reveals that the parameter “G” could be considered a compensation parameter for the strength reduction due to the contact face occurrences. Figure 1

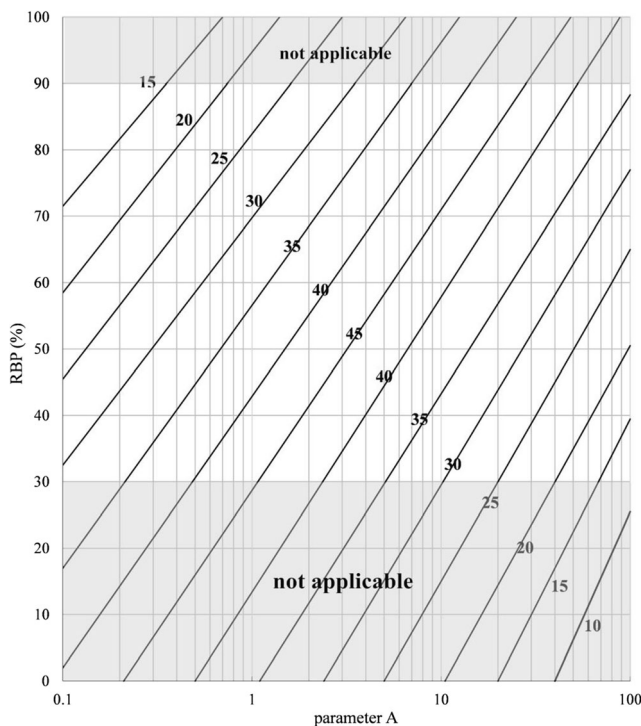


Fig. 3 A practical guide for the selection of “G” on the basis of RBP and parameter “A”

presents that the curve with a higher “A” has a faster descending degree in UCS during the high RBP period, with the corresponding a higher “G” was chosen based in Fig. 4. The value of “G” is highly dependent on the connect strength and RBP, and when contact strength is relatively high, a larger “G” should appear at the last period of RBP increasing.

The parameter “D” could be considered an adjusted parameter related to component properties, which was obtained by nonlinear fitting with the help of parameters “A” and “G,” which also reveals the combination in strength of rock block, soil matrix, and weak contact face. Figure 1 reveals that different “A” values could lead to different decrease degree of UCS and  $A = 1$  is an obvious boundary. The function of parameter “D” was divided into three parts with a range of “A” as  $0 < A < 1$ ,  $1 < A \leq 20$ , and  $20 < A$ . During the calculation in each period, parameters  $m_b$ ,  $s$ , and  $a$  were used to normalized one variable to keep the same order of magnitude. In this method, the difference due to matrix types could be solved. The calculation equations are listed in Table 1. The details of the procedure are given in Fig. 4.

### Validation of the empirical strength formula for critical RBP

The empirical approach to predict the shear strength of SRM in this study is applicable for materials with RBP ranging from 30 to 90%. As previously mentioned, the strength of mixtures with RBP less than 30% and greater than 90% could be obtained by the Mohr-Coulomb (M-C) criterion and the Hoek-Brown (HB) criterion, respectively. Hence, for the mixtures with RBP = 30% and 90%, the prediction values of the empirical approach should be coincident with the results of M-C and HB criteria to make sure the continuity and effectiveness at the calculation boundaries. Several groups of SRM with RBP of 30% and 90% from database were chosen to test, and results are represented in Fig. 5. The relation between predicted and calculated values was obtained as  $y = 1.1x$  with a correlation coefficient of  $R^2 = 0.93$  for the material with RBP = 30%, and the relation for SRM with the high RBP is  $y = 0.94x$  with  $R^2 = 0.92$ . Figure 5 reveals the practicability of the approach and the consistency with M-C and HB criterions for the collected data. Thus, the combination of the empirical approach and the M-C and HB criteria could predict the shear strength of SRM for the whole range of RBP.

### Validation evaluations of the proposed empirical strength approach for SRM

The additional laboratory uniaxial compression tests and direct shear tests were performed in this study to observe the effect on strength of different RBP. The test data were incorporated in the database to validate availability of empirical

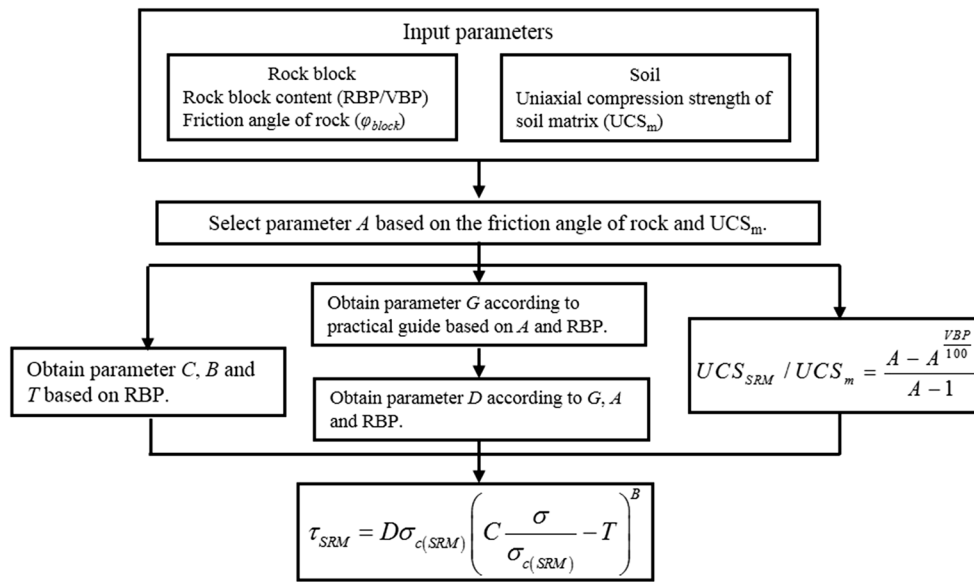


Fig. 4 The flow diagram of the empirical approach to obtain the shear strength of SRM

formula. The SRM samples were collected from a colluvium slope near the Taoyuan-Jinshajiang bridge site, which is located in Yunnan Province, China (Fig. 6). The lithological composition of the SRM in the study area is complicated (including weathered limestone, silty clay, and siltstone). The size of the specimen for the uniaxial compression test is 125 mm × 61.8 mm and the size of the specimen for the direct shear test is 500 mm × 500 mm × 400 mm. The particle size distribution of the SRM for test is shown in Fig. 7, and the diameter threshold between soil and rock particle of SRM in Taoyuan is set to 2 mm.

**Test procedures**

Seven SRM specimens were prepared within the VBP range from 0 to 30% for the uniaxial compression tests. To make specimen molding more convenient, the water content of the soil matrix was maintained at the optimal water content of 17.8%. The rock blocks with particle sizes ranging from 2 to 10 mm were selected (Fig. 8) to meet the required ratio of specimen height to maximum diameter particle size in uniaxial compression test and avoid boundary effect. The materials were mixed adequately to ensure that the rock blocks were uniformly distributed and compacted layer by layer to obtain a dense packing. The specimens were molded after 24-h

consolidation. The soil triaxial compression apparatus was used for the uniaxial compression test. A high VBP (more than 30%) would add difficulties to sample molding during the specimen preparation process. Therefore, the VBP for uniaxial compression test was limited to 30% in this study. The grain size distributions of uniaxial compression test specimens are shown in Fig. 9a. The compression tests were carried out under strain control at a constant rate of 0.368 per min. The loading stopped when the specimen was fully destroyed.

The direct shear test was performed using the large-scale direct shear equipment with a large shear box size (500 mm\*500 mm\*400 mm) and a computer-controlled data acquisition system. The maximum horizon force and displacement are 1000 kN and 90 mm, respectively. The loading force control and data collection are performed by a computerized electro-hydraulic servo control system, which records the vertical loading force and displacements, horizontal shear displacements, and the force of the upper and lower shear boxes. The dry density of the samples was maintained at 1.9 g/cm<sup>3</sup> with a water content of 1.73% (natural water content). The direct shear tests were performed with various RBP range of 0 to 100% under different normal stress (100, 200, 400, 600, and 800 kPa) and the particle size distribution curves of samples were shown in Fig. 9b. The sample was compacted after each of the three layers added to the shear box. During the

Table 1 The summary of calculation equations of *m<sub>b</sub>*, *s*, *a*, and *D* under different “*A*”

	<i>m<sub>b</sub></i>	<i>s</i>	<i>a</i>	<i>D</i>
0 < <i>A</i> < 1	<i>m<sub>b</sub></i> = exp(− <i>G</i> )	<i>s</i> = (γ <sub>G</sub> /50) <sup><i>A</i></sup>	<i>a</i> = AG/25	<i>D</i> = 0.1 <i>a</i> ( <i>sm<sub>b</sub></i> / <i>a</i> ) <sup>0.02</sup>
1 < <i>A</i> < 20	<i>m<sub>b</sub></i> = exp(− <i>G</i> )	<i>s</i> = (γ <sub>G</sub> /50) <sup><i>A</i>/20</sup>	<i>a</i> = <i>m<sub>b</sub></i> <i>s</i> <sup>2</sup>	<i>D</i> = 0.045 × <i>a</i> <sup>−0.176</sup>
20 ≤ <i>A</i>	<i>m<sub>b</sub></i> = A/10 <i>G</i>	<i>s</i> = (γ <sub>G</sub> /45) <sup>γ</sup>	<i>a</i> = 100 <i>s</i> <sup>7</sup> / <i>A</i>	<i>D</i> = 0.064( <i>m<sub>b</sub></i> <i>e<sup>a</sup></i> / <i>s</i> ) <sup>−0.286</sup>



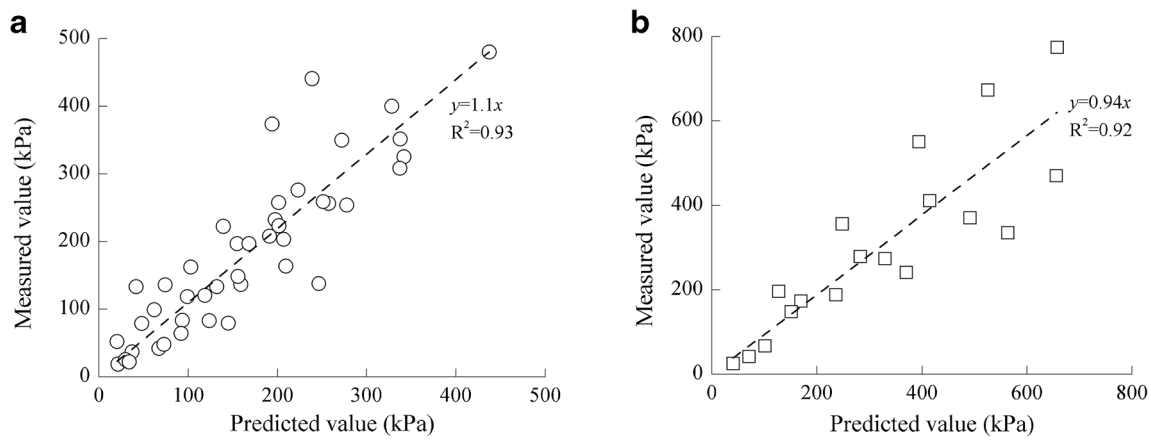


Fig. 5 Prediction performance of the empirical SRM approach with **a** RBP = 30% and **b** RBP = 90%

process of filling the shear box, each layer was compacted by a rubber hammer. Before filling the next layer, the surface was scratched to improve the homogeneity of layers and keep the rocks in random orientations. At the end of this process, a layer of fine soil was placed on the top of the sample to ensure uniform application of the loading force. After consolidation under a small stable vertical force for more than 6 h, the horizontal loading system started moving the lower shear box at a speed of 2 mm/min and stopped when the horizontal displacement reached 75 mm.

**Test results and discussions**

The stress-strain curves of SRM with different VBP are illustrated in Fig. 10, and the variation trends of curves of different specimens are approximately similar. The axial stress increases firstly after declining within the loading process and the increase of VBP would bring down the peak axial stress. The ascent rate of the curve becomes much slower for the SRM with a higher VBP, and the axial strain corresponding

to the peak axial stress keep decreasing as the VBP changes from 0 to 30%. Figure 10 shows that cracks initiate in the middle of the specimen at the peak axial stress and then propagate through the whole specimen, resulting in the destruction of SRM sample. The specimen with a higher VBP often formed more disordered cracks and tortuous failure surfaces because the random distributions of the rock blocks would obstruct cracks extending. The existence of rock blocks would destroy the integrity of SRM samples, and evolving contact faces raise the possibility of sample destruction under normal loading without confined stress. The values of  $UCS_{SRM}$  of different VBP are plotted on Fig. 11 and an exponential relation between  $UCS_{SRM}$  and VBP is proposed in Eq. 8:

$$UCS_{SRM} = 801.51e^{-0.017VBP} \tag{8}$$

Figure 12 presents the shear stress-strain curves with different RBPs as 0%, 40%, 60%, and 100% under various normal stresses. The results show that the RBP has a great effect on SRM shear strength, and under the same normal stress, the



Fig. 6 Photos of SRM in the field in Taoyuan

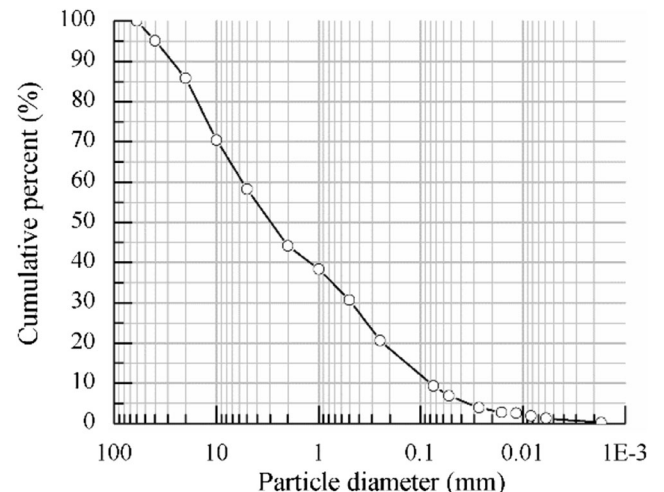
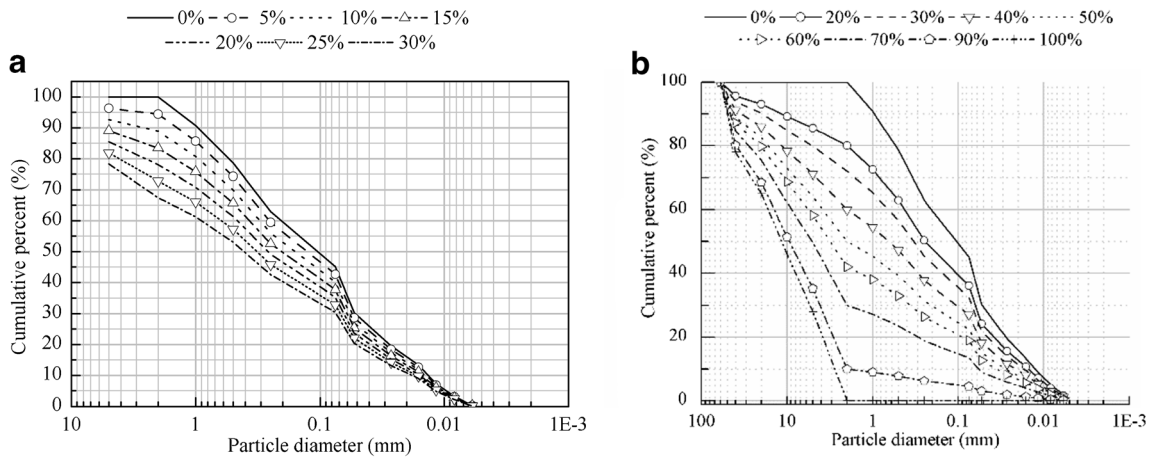
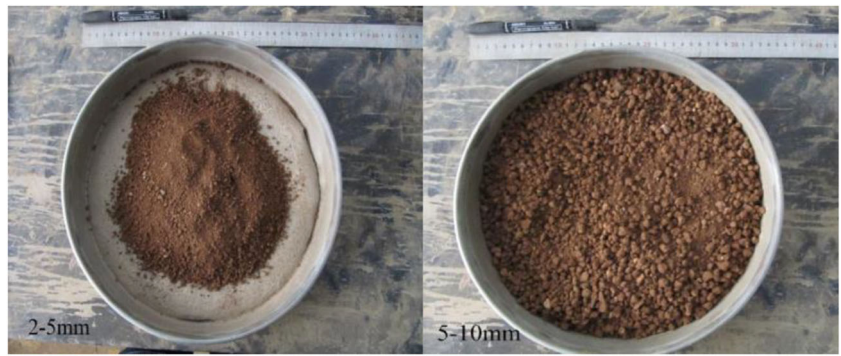


Fig. 7 Particle size distribution of natural SRM

**Fig. 8** SRM blocks for the uniaxial compression tests

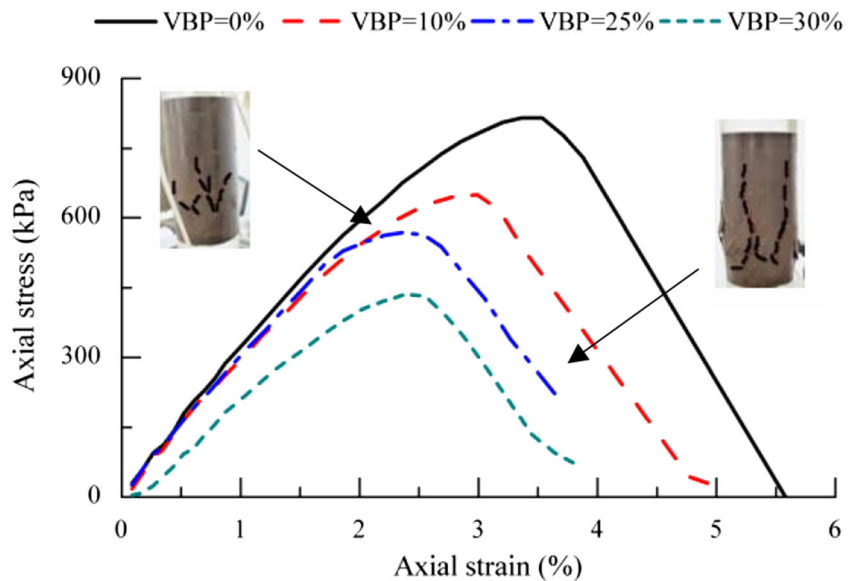


**Fig. 9** Particle size distribution of the SRM samples in **a** uniaxial compression test and **b** direct shear test

shear strength increases as the RBP increases. Under the low normal stress (Fig. 12a), the difference in peak shear strength between RBP = 100 and 60% is the most apparent to be observed as about 100 kPa. This result indicates that under a relative low normal loading, the shear strength provided by

rock blocks is obvious only in the SRM with the high RBP. The rolling and movement of blocks inside the mixture under low normal stress occurred much more easily. There are many voids in the SRM samples with RBP less than 60% under the low normal loading, which lead to a loose and unstable

**Fig. 10** The relation between axial stress and strain



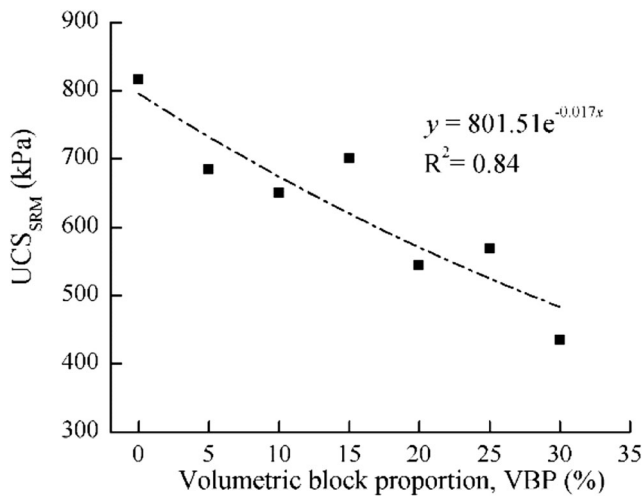


Fig. 11 The relation between  $UCS_{SRM}$  and VBP

structure inside. Under high normal stress (Fig. 12c), SRM specimen was compacted continually during the shearing process, which could improve the interaction between the two components. Therefore, the shear strength is more sensitive

to RBP and has increased obviously since the RBP increased from 0 to 40% as shown in Fig. 12c.

The curve rose approximately in a linear trend at the beginning of the shearing process, which exhibited a plastic stage with the development of shear deformation until the yield stress was reached. The strain-softening could be observed in Fig. 12a, especially in the low-RBP SRM samples. Figure 12 b and c present the occurrences of strain-hardening of high-RBP SRM under high normal stress. The rock blocks in high-RBP SRM or under the high normal stress started to interlock and erode each other after the initial yield stage, and the shear stress started to increase again.

The relationship between RBP and shear strength was also studied, and the results are presented in Fig. 13. Under the low normal stress of 200 kPa, the shear strength increased with the increases of RBP ranging from 0 to 70% and had a sudden decrease to 173.4 kPa at RBP = 90%. Under the high normal stress such as 600 kPa, the shear strength keeps increasing for the samples as RBP ranging from 40 to 100%. But there is an obvious wave trend in shear strength as RBP increases from 0 to 40% under normal stress as 400 kPa and 600 kPa. Such

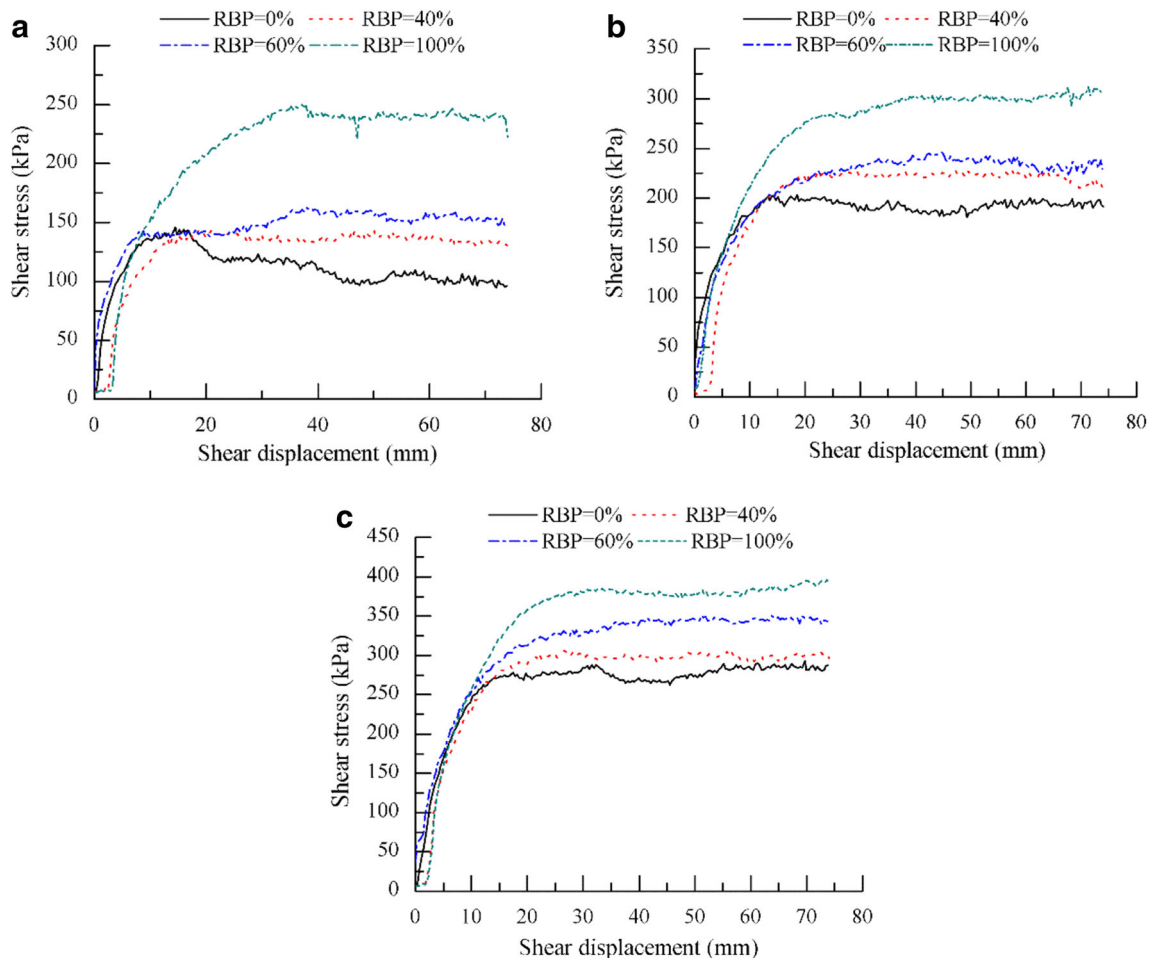


Fig. 12 The relation between shear stress and displacement under different normal stresses. a 200 kPa. b 400 kPa. c 600 kPa

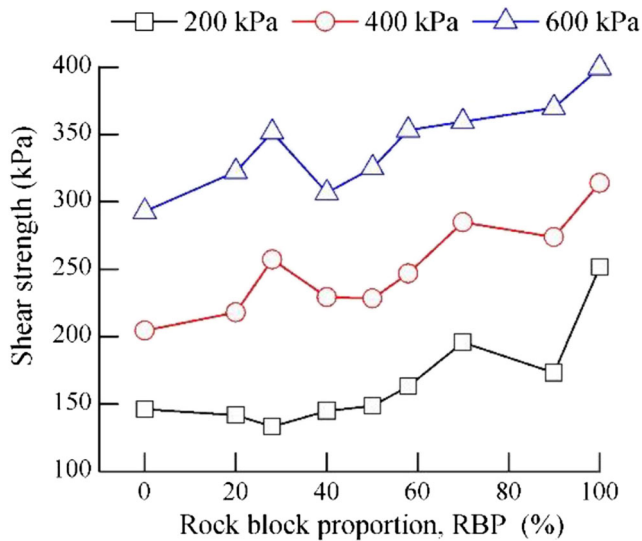


Fig. 13 The relation between shear strength and RBP

variations in strength could be explained by the functions of rock blocks in the geo-material. It is known that the shear strength of SRM is composed of the cohesion mainly provided by soil matrix and the friction supplied by component interactions. At the start stage of RBP increasing, the roundness and strength of rock blocks provided friction for the overall material, leading to the increase of strength. The increase of rock blocks took the place of the soil matrix, resulting in the decrease of cohesion. A sudden drop in shear strength could be found since the increment in friction effects provided by rock blocks is smaller than the decrease in cohesion, resulting in the loss of soil matrix. The rock blocks gradually build the skeleton of the mixture as the RBP changes from 30 to 70% and shear strength is dominated by both rock blocks and soils. The false cohesion induced by the interaction of soil matrix and rock blocks appeared, and under high normal stress, the compaction and interlock effect is more obvious. The occurrence of the second turning point of shear strength is attributed to the increase of contact faces with weak strength and the voids inside SRM samples under the low normal stress.

**Validation evaluations of predicted values and measured values**

To evaluate the validation of the empirical approach to SRM shear strength, the predicted values of SRM samples in database were compared with the corresponding laboratory test results. According to the results of uniaxial compression tests and large-scale direct shear tests, the value of  $UCS_m$  is 816.24 kPa and the friction angle of the rock block is  $28.7^\circ$ . The parameter “A” could be obtained as 0.49. Equation 1 was proposed based on several bimrock uniaxial compression tests, and there are differences in soil matrix between bimrock and SRM. According to concepts and definitions of SRM and

bimrock, the contact strength between rock blocks and soil matrix is much higher in the bimrock than the SRM. In order to validate the generality of the formula in predicting  $UCS_{SRM}$ , several measured and predicted  $UCS_{SRM}$  values from database were prepared. The predicted performance of Taoyuan SRM and other data is presented in Fig. 14. It could be found that the predicted  $UCS_{SRM}$  values according to Eq. 1 were close to the measured values. The relation between the predicted values and measured values is  $y = 0.96x$ , with  $R^2 = 0.96$ . Consequently, the formula could be used as the reference of UCS values for SRM by taking component mechanical properties and RBP into account.

In the comparison between the predicted values of the shear strength of Taoyuan SRM and other test data from database, which focused on the fitting degree of the RBP ranging from 30 to 90%, a relation of  $y = 0.98x$  was determined with the correlation coefficient of  $R^2 = 0.75$ . Figure 15 indicates that most of the predicted values were larger by about 50–100 kPa than the measured values. The error in the estimation of the shear strength of SRM demonstrated a few restrictions in the prediction under high normal stress. It should be noted that the predicted shear strength in this paper was calculated only according to RBP and part of mechanical properties of components. However, the real mechanical behaviors of SRM are complex and influenced by many other factors, such as the spatial distribution of rock blocks and the size of samples.

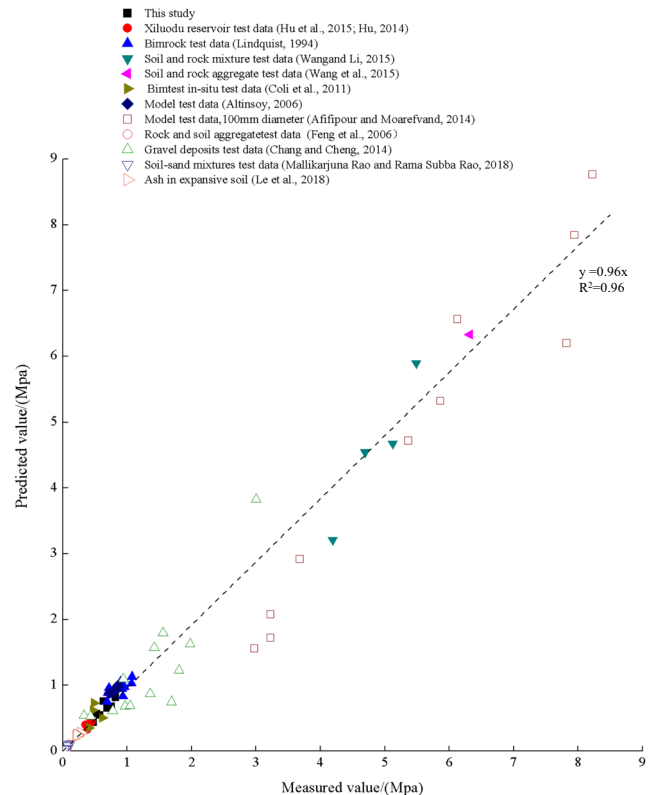
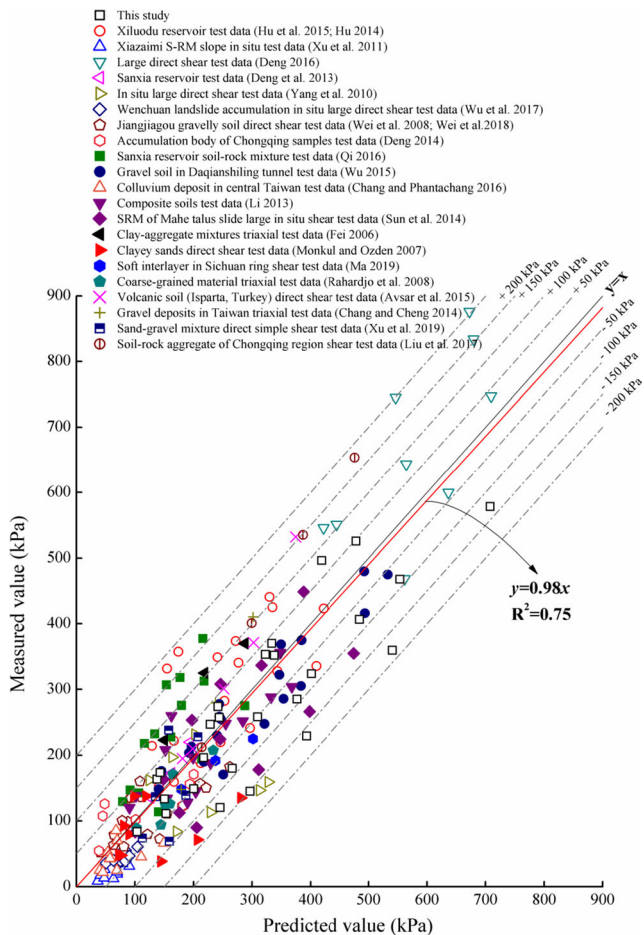


Fig. 14 Cross-correlation of measured and predicted values of  $UCS_{SRM}$



**Fig. 15** Cross-correlation of measured and predicted values of shear strength

Therefore, many factors should still be considered in the empirical approach to improve the accuracy of the results and represent the mechanical character of the SRM.

Consequently, the strength parameters and UCS values of SRM predicted by the empirical formula are much higher than the parameters in reality. The accuracy of the approach has been validated by the test data in database, and the error in estimation for strength parameters is acceptable as the reference in design stage of engineering projects.

## Conclusions

Due to the difficulties in the preparation of standard core samples and high cost of large-scale mechanical tests, the possibility of utilizing an empirical approach to predict the UCS and shear strength of SRM was investigated. For this purpose, more than 100 groups of SRM mechanical test results were collected from well-documented literatures. After systematic analysis of geological background and shear behaviors of SRM, an empirical formula to predict SRM shear strength

within a certain range of RBP was proposed with nonlinear regression of the available data. Furthermore, several groups of uniaxial compression tests and direct shear tests were performed with different RBP by using the SRM collected from Yunnan. Then, the predicted values of SRM in database were validated satisfactorily by the measured values, which proved that the approach could determine the SRM strength in a more practical and convenient way.

The previous studies show that the variation in the RBP would result in the transition of material structures. The spatial distribution of rock blocks change from floating in the matrix to rock-skeleton as the RBP increases. The strength characteristics of SRM are closely related with RBP, and the dominant component of SRM which provides the primary shear resistance for SRM would change with various RBP. This study focuses on the variations of shear strength with the RBP range of 30–90%, and the shear strength is assumed to be provided by rock blocks and soil matrix both in the selected range.

An empirical formula was proposed from this study to estimate the shear strength of natural SRM based on the experimental results in database. In order to describe the mechanical characteristics of SRM, the influence of RBP, mechanical properties of components, and soil-rock contact faces were considered in defining input parameters of the formula. The accuracy of the suggested relation was evaluated using the uniaxial compression tests and direct shear test results, with  $R^2$  values of 0.96 and 0.75, respectively, which showed the availability of empirical approach could be proved in estimating the SRM shear strength with different RBP values.

There are still improvements and simplifications to be made to the SRM empirical approach in predicting shear strength. It is necessary to study the nonlinear trend of shear strength as a function of factors other than the RBP, such as particle size, sample density, and rock orientation. Moreover, future studies should focus on the interactions between the rocks and soils and the uncertainties to better understand the impact of the rock block content on the strength behaviors of SRM.

**Funding information** The authors gratefully acknowledge the support given by National Key R&D Program of China (2018YFC0809400), the National Natural Science Foundation of China (No. 51779250, No. U1402231, No. 51679232, No. 51609240, No. 51909259), and the Traffic Science, Technology and Education Project of Yunnan Province (2017) No. 33.

## References

- Afifipour M, Moarefvand P (2014) Mechanical behavior of bimrock shaving high rock block proportion. *Int J Rock Mech Min Sci* 65: 40–48
- Altinsoy H (2006) A Physical based model investigation for determination of shear strength of block in matrix rocks (MSc Thesis).

- Hacettepe University, Geological Engineering Department, Ankara, Turkey (91 pp)
- Avsar E, Ulusay R, Mutluturk M (2015) An experimental investigation of the mechanical behavior and microstructural features of a volcanic soil (Isparta, Turkey) and stability of cut slopes. *Eng Geol* 189:68–63
- Basu A, Celestino TB, Bortolucci AA (2009) Evaluation of rock mechanical behaviors under uniaxial compression with reference to assessed weathering grades. *Rock Mech Rock Eng* 42(1):73–93
- Catane SG, Cabria HB, Zarco MAH, Saturay RM Jr, Mirasol-Rober A (2008) The 17 February 2006 Guinsaugon rock slide-debris avalanche, Southern Leyte, Philippines: deposit characteristics and failure mechanism. *Bull Eng Geol Environ* 67:305–320
- Chang KT, Cheng MC (2014) Estimation of the shear strength of gravel deposits based on field investigated geological factors. *Eng Geol* 171:70–80
- Chang WJ, Phantachang T (2016) Effects of gravel content on shear resistance of gravelly soils. *Eng Geol* 207:78–90
- Coli N, Berry P, Boldini D (2011) In situ non-conventional shear tests for the mechanical characterisation of a bimrock (BimTest). *Int J Rock Mech Min Sci* 48:95–102
- Coli N, Berry P, Boldini D, Bruno R (2012) The contribution of geostatistics to the characterization of some bimrock properties. *Eng Geol* 137–138:53–63
- Coskun A (2010) Development of an empirical approach to overcome the problems for boundary condition between bimrocks and jointed Rock Masses (MSc Thesis). Hacettepe University, Geological Engineering Department, Ankara, Turkey
- Dasaka SM, Zhang LM (2012) Spatial variability of in situ weathered soil. *Geotechnique* 62:374–384
- Deng XC (2014) The tunnel excavation and its slope stability in the accumulation body. Chongqing Jiaotong University, Chongqing
- Deng HF (2016) Test research on the shear strength characteristics of rock-soil aggregate mixture. East China University of Technology, Jiangxi
- Deng HF, Yuan XF, Li JL, Luo Q, Zhu M (2013) Research on failure characteristics and determination method for shear strength of earth-rock aggregate in direct shear tests. *Chin J Rock Mech Eng* 32:4065–4072
- Donaghe RT, Torrey VH (1979) Scalping and replacement effects on strength parameters of earth-rock mixtures. *Des Parameters Geotech Eng Lond* 2:29–34
- Dupla JC, Pedro LS, Canou J, Dormieux L (2007) Mechanical behavior of coarse-grained soils reference. *Bulletin de Liaison des Laboratoires des Ponts et Chaussées* 268–269:31–57
- Fei K (2016) Experimental study of the mechanical behavior of clay-aggregate mixtures. *Eng Geol* 210:1–9
- Feng C, Li ZG, Li SH (2018) Study on uniaxial compression characteristics of brittle rock and soil aggregate. *Chin J Comput Mech* 35:356–363
- Gao W, Hu R, Oyediran IA, Li ZQ, Zhang XY (2014) Geomechanical characterization of zhangmu soil-rock mixture deposit. *Geotech Eng* 32:1329
- Hoek E, Brown ET (1980) Underground excavations in rock. *Inst. Min. Metall.* Stephen Austin and Sons, London
- Hoek E, Brown ET (1997) Practical estimates of rock mass strength. *Int J Rock Mech Min Sci* 34:1165–1186
- Hoek E, Marinos P, Benissi M (2002) Applicability of the geological strength index (GSI) classification for very weak and sheared rock masses. The case of the Athens Schist Formation. *Bull Eng Geol Environ* 57(2):151–160
- Holtz WG, Willard M (1956) Triaxial shear characteristics of clayey gravel soils. *J Soil Mech Found Div* 820(SM1):1–22
- Hong K, Han E, Kang K (2017) Determination of geological strength index of jointed rock mass based on image processing. *J Rock Mech Geotech Eng* 9(4):702–708
- Hu W (2014) Experimental study on shear strength of soil-rock-mixture in Xiluodu reservoir. *Inst. Rock and Soil Mechanics, Chinese Academy of Sciences, Wuhan, China* (in Chinese)
- Hu W, Min H, Chen J, Sheng Q (2015) Study on the effect of gravel on unconfined mechanical properties of soil-rock mixture. *J Yangtze River Sci Res Inst* 11:55–61
- Iannacchione AT (1997) Shear strength saturated clays with floating rock particles. University of Pittsburgh, Oakland
- Kahraman S, Alber M (2006) Estimating unconfined compressive strength and elastic modulus of a fault breccia mixture of weak blocks and strong matrix. *Int J Rock Mech Min Sci* 43:1277–1287
- Kalender A, Sonmez H, Medley E, Tunusluoglu C, Kasapoglu KE (2014) An approach to predicting the overall strengths of unwelded bimrocks and bimsoils. *Eng Geol* 183:65–79
- Khalili A, Wijewickreme D, Wilson GW (2010) Mechanical response of highly gap-graded mixtures of waste rock and tailings. Part I: monotonic shear response. *Can Geotech J* 47:552–565
- Le TM, Dang LC, Khabbaz H (2019) Combined effects of bottom ash and lime on behaviour of expansive soil. In: McCartney J, Hoyos L (eds) *Recent advancements on expansive soils. GeoMEast 2018. Sustainable Civil Infrastructures*. Springer, Cham
- Li YR (2013) Effects of particle shape and size distribution on the shear strength behavior of composite soils. *Bull Eng Geol Environ* 72:371–381
- Lindquist ES, Goodman RE (1994) The strength and deformation properties of a physical model mélange. *Proc. 1st North American Rock Mech. Symp., Austin, Texas*, pp 843–850
- Liu XR, Tu YL, Wang P, Zhong ZL, Tang WB, Du LB (2017) Particle breakage of soil-rock aggregate based on large-scale direct shear tests. *Chin J Geotech Eng* 39:1425–1434
- Ma C, Zhan HB, Zhang T, Yao WM (2019) Investigation on shear behavior of soft interlayers by ring shear tests. *Eng Geol* 254:34–42
- Mallikarjuna Rao K, Rama Subba Rao GV (2018) Influence of coarse fraction on characteristics of expansive soil-sand mixtures. *Int J Geosyn Ground Eng* 4:19. <https://doi.org/10.1007/s40891-018-0136-8>
- Medley EW, Goodman RE (1994) Estimating the block volumetric proportions of melanges and similar block-in-matrix rocks (bimrocks). *Proc. 1st North American Rock Mechanics Symposium, Austin*
- Monkul MM, Ozden G (2007) Compressional behavior of clayey sand and transition fines content. *Eng Geol* 89:195–205
- Qi D (2016) Experimental study on the mechanical properties of soil-rock mixture. North China University of Water Resources and Electric Power, Henan (in Chinese)
- Raharadjo H, Indrawan IGB, Lenog EC, Yong WK (2008) Effects of coarse-grained material on hydraulic properties and shear strength of top soil. *Eng Geol* 101:165–173
- Raymond LA (1984) Classification of melanges. In: Raymond LA, Boulder LA (eds) *Melanges: their nature, origin and significance*. Geol. Soc. of America, pp 7–20
- Singh HK, Basu A (2016) Shear behaviors of ‘real’ natural un-matching joints of granite with equivalent joint roughness coefficients. *Eng Geol* 211:120–134
- Sun SR, Xu PL, Wu LM, Wei JH, Fu WG, Liu J, Kangungo DP (2014) Strength parameter identification and application of soil-rock mixture for steep-walled talus slopes in southwestern China. *Bull Eng Geol Environ* 73:123–140
- Tian YC, Liu QS, Ma H, Liu Q, Deng PH (2018) New peak shear strength model for cement filled rock joints. *Eng Geol* 233:269–280
- Ulusay R, Erguler ZA (2012) Needle penetration test: evaluation of its performance and possible uses in predicting strength of weak and soft rocks. *Eng Geol* 149–150:47–56
- Usefzadeh A, Yousefzadeh H, Salari-Rad H, Sharifzadeh M (2013) Empirical and mathematical formulation of the shear behavior of rock joints. *Eng Geol* 164:243–252

- Vallejo LE (2001) Interpretation of the limits in shear strength in binary granular mixtures. *Can Geotech J* 38:1097–1104
- Vallejo LE, Zhou Y (1994) The mechanical properties of simulated soil-rock mixtures. In: *Proceedings of the 13th International Conference on Soil Mechanics and Foundation Engineering*, New Delhi, India. pp 365–368
- Wang Y, Li X (2015) Experimental study on cracking damage characteristics of a soil and rock mixture by UPV testing. *Bull Eng Geol Environ* 74:775–788
- Wang Y, Li CH, Hu YZ (2018) X-ray computed tomography (CT) observations of crack damage evolution in soil-rock mixture during uniaxial deformation. *Arab J Geosci* 11:199
- Wang Y, Li CH, Hu YZ (2019) 3D image visualization of meso-structural changes in a bimsoil under uniaxial compression using X-ray computed tomography (CT). *Eng Geol* 248:61–69
- Wei HZ, Wang R, Hu MJ, Zhao HY, Meng QS (2008) Strength behavior of gravelly soil with different coarse-grained contents in Jiangjiagou Ravine. *Rock Soil Mech* 29(1):48–51
- Wei HZ, Xv WJ, Wei CF, Meng QS (2018) Influence of water content and shear rate on the mechanical behavior of soil-rock mixtures. *Sci China Technol Sci* 51(8):1127–1136
- Weibel ER (1980) *Stereological Method*, vol 2. Academic Press, London
- Wickland BE, Wilson GW, Wijewickreme D, Klein B (2006) Design and evaluation of mixtures of mine waste rock and tails. *Can Geotech J* 43:928–945
- Wu R (2015) Study on the strength experiment of gravel soil in the slope of Daqianshiling Tunnel. China University of Geosciences, Wuhan
- Wu RA, Zhang YS, Wang XL, Yao X, Yang ZH, Du GL (2017) In situ direct shearing test on landslide accumulation body intensity of Wenchuan earthquake region. *J Geom* 23:105–114
- Xu WJ, Xv Q, Hu RL (2011) Study on the shear strength of soil-rock mixture by large scale direct shear test. *Int J Rock Mech Min Sci* 48(8):1235–1247
- Xu DS, Tang JY, Zou Y, Rui R, Liu HB (2019) Macro and micro investigation of gravel content on simple shear behavior of sand-gravel mixture. *Constr Build Mater* 221:730–744
- Yang JH, Dong JY, Liu HD, Huang ZQ, Jiang T (2010) In situ direct shear tests on the large scale accumulation body and three dimensional stability analysis of the slope. *J China Coal Soc* 35:392–396
- Zhang YS, Guo CB, Yao X, Qu YX, Zhou NJ (2013) Engineering geological characterization of clayey diatomaceous earth deposits encountered in highway projects in the Tengchong region, Yunnan, China. *Eng Geol* 167(17):98–104
- Zhang HY, Xv WJ, Yu YZ (2016) Triaxial tests of soil-rock mixtures with different rock block distributions. *Soils Found* 56(1):44–56
- Zhao YL, Gao Y, Zhang YL, Wang Y (2016) Effect of fines on the mechanical properties of composite soil stabilizer-stabilized gravel soil. *Constr Build Mater* 126(15):701–710