

On mechanical behaviour of clastic soils: numerical simulations and constitutive modelling

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Clastic soil is ubiquitous in practice but still poses challenge for testing and modelling. Conventional laboratory tests with small sizes are not suitable for the large grain size, while large-scale tests and field tests are often too costly and time-consuming. In this paper, the mechanical behaviour of clastic soil is described by a hypoplastic constitutive model based on numerical simulations considering both the soil matrix and breccia. The numerical simulations indicate that the breccia content, gradation, and the overconsolidation ratio have significant influence on the strength, deformation, and failure pattern of the clastic soil. The coarse particles are found to change the stress path developed in the soil matrix and consequently influence the critical state of the soil matrix. Moreover, the coarse particle distribution based on computed tomography scan seems to have larger impact than the breccia content for the failure pattern of the clastic soils.

KEYWORDS: Constitutive relations; Failure; Finite-element modelling; Landslides; Shear strength

INTRODUCTION

Most reactivated slow-moving landslides in the Three Gorges Reservoir (TGR) area of China are characterized by pre-existing shear surfaces (Miao et al., 2014; Wang et al., 2020d, 2018a). For these landslides, the localized sliding on a main shear surface is the prevailing type of movement (Maio et al., 2015). The localization within the shear zones usually gives rise to clastic formations. These clastic formations mainly consist of fine-grained soils composed of sand, silt and clay, and a fraction of coarse particles crushed from underlying bedrocks or floating rocks from overlying colluviums of landslides. Generally, the clastic soils in shear zones are characterized by weak shear strength, componential inhomogeneity, high overconsolidation, and particle orientation. These properties make the physical and mechanical properties of the shear zone soils significantly different from the soils in other parts of landslides, such as the sliding masses. Moreover, it is reported that the majority of viscoplastic deformations in a reactivated slow-moving landslide often occurs at the shear zone where the soil is subjected to a shear stress larger or close to the residual strength of the soil (Maio & Scaringi, 2016). Therefore, a proper understanding of the strength and deformation behaviour and failure mechanism of the clastic soil in the shear zone is the key point in the prediction of the kinematics and stability of reactivated slow-moving landslides.

During the last decades, great effort has been devoted to the investigation of the properties of soil from shear zones of landslides by performing conventional laboratory tests. Such studies focus mainly on the shear strength (Maio & Scaringi, 2016; Maio et al., 2015; Wen et al., 2007) and creep characteristics (Li et al., 2019; Wang et al., 2018a, 2020d). However, most of these investigations referred only to breccia-free soils, and the mechanical properties of the soil matrix are often used in the stability analysis of slow-moving landslides. Although some in-situ studies involve intact samples with coarse contents (Tan et al., 2018; Wang et al., 2020d), they usually take the clastic soil sample as continuous media. The effects of coarse particles on the overall mechanical behaviour

are often neglected. A few studies consider the effect of coarse particles by using large-scale apparatus in the laboratory (Li & Aydin, 2013; Li et al., 2012). The test results show that the residual strength tends to increase with the fraction of breccia contents in shear zone soils, and the particle size distribution plays an important role in the enhancement of the shear resistance. Consequently, the mechanical behaviour of shear zone materials at the residual state can be significantly influenced by the coarse particles (Wen & Jiang, 2017).

In fact, the clastic soils in shear zones have similar properties, such as inhomogeneity and microstructure, to the so-called rock-soil mixtures if the particle orientation and pre-existing localization are not considered. All previous studies have indicated that the volume content, size and shape of the coarse particles, and the composition of fine grains have great influences on the mechanical properties and failure mechanism of the rock-soil mixtures (Chang & Phantachang, 2016; Xu et al., 2008; Wang et al., 2018e; Meng et al., 2002). Analogously, these factors are bound to effect the mechanical behaviour of shear zone soils. Although large-scale laboratory tests and field tests are able to uncover some basic behaviours of this material, on the one hand, they are often too costly and time-consuming, and on the other hand, only regular testing conditions can be realized. As a result, continuum numerical approaches, such as the finite element method (FEM) and finite difference method (FDM), are increasingly used to investigate the mechanical behaviours of complex geomaterials under various testing conditions. There are many numerical studies referring to the rock-soil mixtures in the literature. Some specific topics, such as the influence of breccia on the distribution of stress fields, shearing localization, and strength enhancement in soil-rock mixtures, have been investigated with the help of digital image processing methods in the early stage through the FEM approach (Yue et al., 2003). Later, this method was enhanced by the computed tomography (CT) technique to obtain the real spatial distribution of coarse particles in geomaterials to study the mechanical behaviour of rock-soil mixtures (Jiang et al., 2016; Li et al., 2016; Xu et al., 2008).

In the aforementioned numerical studies, the mechanical and deformation response of the soil are usually modelled by Mohr-Coulomb failure criteria, which gives rise to an ideal elastoplastic response in soil. Obviously, the simulations are not able to capture the nonlinear stress response, critical state, and consolidation history of the clastic soil in the shear zone.

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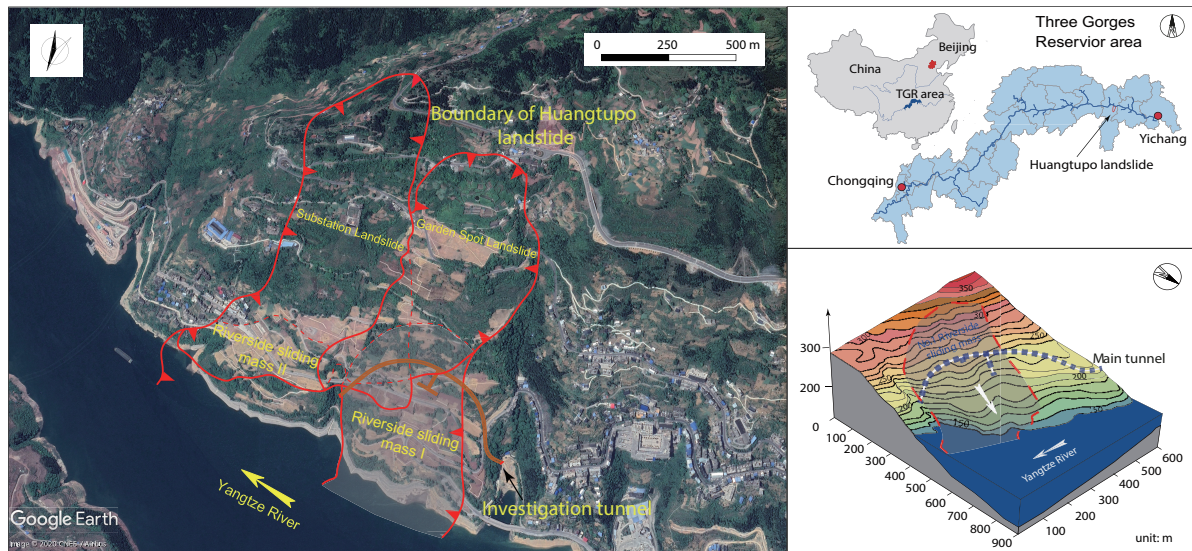


Fig. 1. Plan view of the Huangtupo landslide with 3D topographic model of the No. 1 Riverside sliding mass, modified after (Tang et al. , 2015; Wang et al. , 2018)

Therefore, a more advanced model incorporating both the critical state and overconsolidation is required to serve this purpose. As an alternative to the prevailing plasticity theory, hypoplastic models have gained much attention in recent years (Fuentes et al. , 2019; Mašín , 2012, 2014; Wang et al. , 2018c,b; Wang & Wu , 2020a; Wang et al. , 2020c). Unlike elastoplastic models, hypoplastic models are able to describe the critical state and overconsolidation behaviour without using the yield surface, plastic potential, decomposition into elastic and plastic parts, and a loading criterion (Wu et al. , 2017). Therefore, hypoplastic models are usually characterized by simple formulations and less model parameters. In particular, the hypoplastic model has been successfully applied to simulate an inhomogeneous lumpy soil in numerical tests (Shi & Herle , 2017).

The aim of this study is to present a numerical approach for modelling the clastic soil by using a novel hypoplastic model. The effects of two important factors, namely, the overconsolidation ratio (OCR) and breccia content, on the mechanical behaviour of clastic soil are investigated. Specifically, the general mechanical behaviours of clastic soil are investigated through a series of 3D triaxial compression tests on artificial clastic samples, and the failure mechanism of clastic soil is studied by performing 2D plane strain tests considering the real distribution of coarse particles in the clastic soil. In these simulations, the soil matrix is modelled by the hypoplastic model, while the mechanical behaviour of the accommodated breccia is assumed to be elastic. In the following, except where specified otherwise, effective stress is used.

CLASTIC SOIL IN THE SHEAR ZONE OF A LANDSLIDE

General description of the landslide

The Huangtupo landslide is a well-documented reactivated slow-moving landslide in the TGR area of China. It is located in the south bank of the valley of the Yangtze River with the bedrock dipping towards the valley (Fig. 1). The landslides occur in the so-called Badong formation (Middle Triassic T_2b^2 , T_2b^3) which is characterized by alternating hard and soft layers composed of mudstone, pelitic siltstone, and argillaceous limestone (Tang et al. , 2015; Wang et al. , 2014). These stratified strata are prone to large-scale landslides, and clastic

soil is commonly generated due to shear localization on the shear surface.

The landslide covers a total area of $1.35 \times 10^6 \text{ m}^2$, with a volume of nearly $6.93 \times 10^7 \text{ m}^3$. On the surface, the landslide is divided by a valley into two groups of sliding masses. Each group is composed of two sliding masses, one on the top of the other. The foreside sliding masses, the No. 1 and No. 2 Riverside sliding masses, are adjacent to the Yangtze River, with their toes being partially submerged in the river. The No. 1 Riverside sliding mass is still active, with a very slow to extremely slow rate of displacement (Wang et al. , 2018a; Cui et al. , 2020). The large size and complex structure of the landslide make it an ideal study object to better understand the mechanism of similar slides in the TGR area. To make this landslide a benchmark, a large-scale investigation tunnel beneath the No. 1 Riverside sliding masses was constructed in 2012 (Tang et al. , 2015).

Characteristics of shear zone soil

The excavation of the Badong field testing tunnel exposed a main shear zone consisting of three lithologies, namely, brown-yellow gravel soil in the slide mass, yellow and light-grey clay with breccia along the shear zone, and bluish-brown and brownish-yellow argillaceous limestone formed in the subjacent stable bedrock. According to previous laboratory tests (Chen et al. , 2009), the examined soil can be classified as silt-clay with low to medium plasticity. The basic physical properties of the clastic soil are listed in Table 1.

Table 1. Basic properties of the clastic soil in the shear zone of the Huangtupo landslide

Index	Range	Average
Natural water content, w (%)	14.8 – 23.0	18.3
Natural unit weight, γ (kN/cm ³)	19.5 – 21.1	20.6
Void ratio, e	0.44 – 0.71	0.54
Degree of saturation, S_r (%)	84.1 – 94.7	88.2
Plasticity index, I_P (%)	8.6 – 24.9	13.2
Liquid index, I_e	0.1 – 0.31	0.23

Some intact samples were taken from the shear zone to determine the grain size distribution. Figure 2(a) shows the

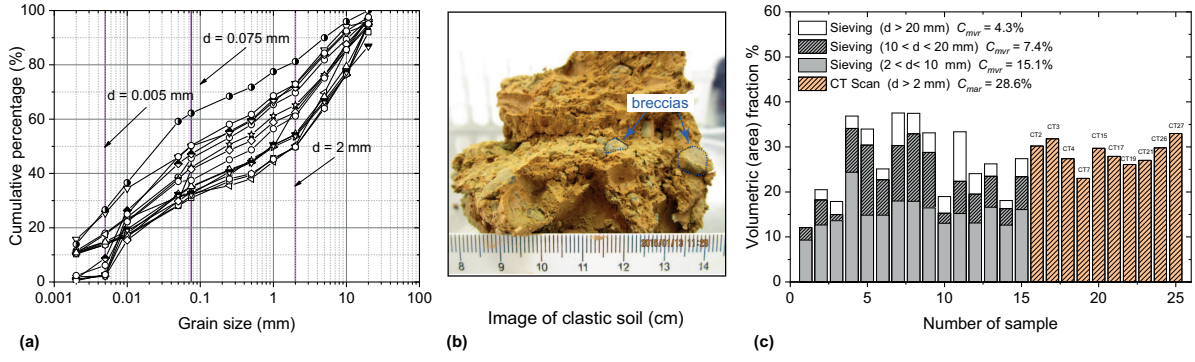


Fig. 2. Coarse particles in the clastic soil: (a) grain size distribution; (b) image of the clastic soil, and (c) volumetric and area fraction of coarse particles obtained from sieving method and analysing of CT images, respectively

grain size distribution of 15 randomly chosen samples. The soil typically comprises fine-grained soil (silt + clay) with a significant fraction of coarse particles. Specifically, the breccia ($d > 2$ mm) constitute approximately 18.8 to 50.3 wt% of the clastic soil. Figure 2(b) shows the image of an intact sample, where the breccia are clearly distinguished from the soil matrix. The breccias are typically subrounded to angular and irregularly shaped, with diameters concentrating in the range of 2 to 10 mm. Numerous experimental studies provided evidence that the interaction of particles in coarse-graded soil, with 25 and 50 wt% coarse particles, remains complex (Jiang et al., 2016; Xu et al., 2008), and the mechanical behaviour can be significantly influenced by the coarse portion.

The volumetric proportion of coarse particles in the clastic samples can be obtained according to:

$$C_v = \frac{\rho_s}{\rho_b(1/C_w - 1) + \rho_s} \quad (1)$$

where C_v and C_w denote the volumetric and weight fraction of coarse particles, respectively; $\rho_s = 1.65 \text{ kg/cm}^3$ and $\rho_b = 2.7 \text{ kg/cm}^3$, corresponding to the dry densities of the fine and coarse particles in the clastic soil, respectively. Figure 2(c) shows the volumetric and area fractions of coarse particles with diameter larger than 2 mm obtained by sieving the samples and analysing 2D CT-image of the samples presented in Figure 4. The sieving and CT scanning methods give rise to average volume and area fractions of $C_{mv} = 26.8\%$ and $C_{ma} = 28.6\%$, respectively. Since coarse particles in CT images are presented by the largest scanning sections of the particles, the breccia content obtained from analysing 2D images is generally greater than that measured from the sieving method. It is worth noting that the fraction of particles with diameters larger than 10 mm can be obtained from the 15 sieving samples. An inspection of this result suggest that the breccias with $d \in [10 \text{ } 20]$ mm distribute rather unevenly in the soils. In contrast, the spatial distribution of particles with $d \in [2 \text{ } 10]$ mm is more uniform. The average volume fraction of breccias in such range is $C_{mv} = 15.1\%$. Obviously, there is a fraction of oversized breccias larger than 20 mm in the shear zone material, which may influence the failure mechanism under shearing.

HYPOPLASTIC CONSTITUTIVE MODEL

In this work, a novel hypoplastic model proposed by Wang & Wu (2020b) is used to simulate the soil matrix of the clastic soil. The model includes a structure tensor to account for the history dependence. The model is formulated as follows:

$$\dot{\mathbf{T}} = f_s \left[(\text{tr} \dot{\mathbf{T}}) \mathbf{D} + f_v (\text{tr} \mathbf{D}) \dot{\mathbf{T}} + a^2 \frac{\text{tr}(\dot{\mathbf{T}} \mathbf{D})}{\text{tr} \dot{\mathbf{T}}} \dot{\mathbf{T}} + a(\dot{\mathbf{T}} + \dot{\mathbf{T}}^*) \|\mathbf{D}\| \right] \quad (2)$$

where $\dot{\mathbf{T}}$ is the Jaumann stress rate tensor; $\mathbf{D}$ is the strain rate (stretching) tensor; $\|\mathbf{D}\|$ stands for the Euclidean norm of the stretching tensor; $\dot{\mathbf{T}} = \mathbf{T} + \mathbf{S}$ and $\dot{\mathbf{T}} = \mathbf{T} - \mathbf{S}$ are used to account for consolidation history with $\mathbf{T}$ and $\mathbf{S}$ being the Cauchy stress tensor and structure tensor, respectively. A simple formulation is adopted for the structure tensor:

$$\mathbf{S} = \alpha \ln\left(\frac{1}{R}\right) \mathbf{T} \quad (3)$$

where α is a model parameter, and R is the stress ratio representing the overconsolidation, which is defined as:

$$R = \frac{p_e^+}{p_e} \quad (4)$$

in which p_e^+ and p_e are the preconsolidation pressure in the Modified Camclay (MCC) model and the Hvorslev equivalent pressure, respectively. The formulations of p_e^+ and p_e are given as follows:

$$p_e^+ = p \left[1 + \left(\frac{q}{Mp} \right)^2 \right], \quad p_e = \exp \left[\frac{N - \ln(1 + e)}{\lambda^*} \right] \quad (5)$$

where the inclination M corresponds to the stress ratio at the critical state by $M = 6 \sin \phi_c / (3 - \sin \phi_c)$, with ϕ_c being the critical state friction angle; N denotes the logarithmic value of the specific volume at the reference stress $p_r = 1 \text{ kPa}$; e is the void ratio of the soil; and the index λ^* is the slope of the normal compression line in the double logarithmic $\ln(1 + e) - \ln p$ plane.

The multipliers f_s and f_v account for the soil stiffness and volumetric response of the soil, respectively, which are defined as:

$$f_s = -\frac{2}{3r_i \lambda^*}, \quad f_v = \frac{3}{2} r_i - \frac{1}{3} (3 + a^2 - \sqrt{3}a) \quad (6)$$

where r_i is the ratio of the bulk modulus in the isotropic compression and the shear modulus in the undrained shear test on the isotropic consolidated sample; it has a similar physical meaning to Poisson's ratio. The material constant a corresponds to the limit stress at the critical state. It reads:

$$a = \frac{\sqrt{3}\eta(3 - \sin \phi_c)}{2\sqrt{2}\sin \phi_c} \quad (7)$$

where the factor η is adopted to incorporate the Matsuoka-Nakai failure criterion, giving:

$$\eta = \frac{2I_1}{3\sqrt{(I_1 I_2 - I_3)(I_1^2 - 3I_2)/(I_1 I_2 - 9I_3) - \sqrt{(I_1^2 - 3I_2)}}} \quad (8)$$

in which I_1 , I_2 , and I_3 are stress invariants.

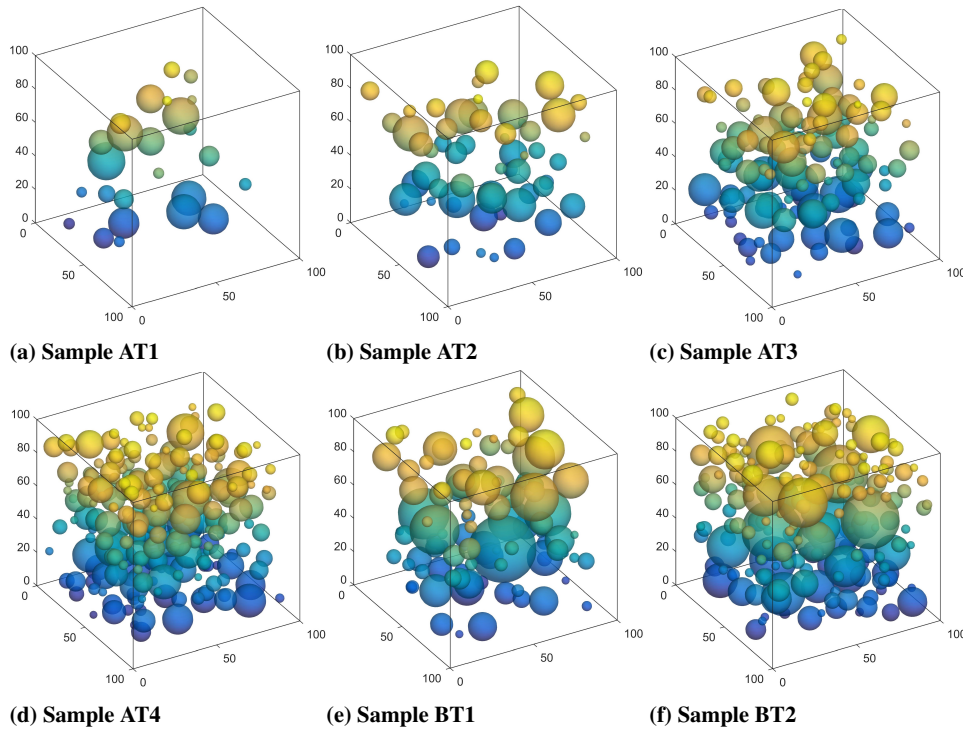


Fig. 3. Artificial clastic samples with different volume fractions of breccia (a) 2.7%, (b) 5.5%, (c) 10.5%, (d) 15.0% for group A with $d \in [2 \ 10]$ mm and (e) 15.0% (f) 22.0% for group B with $d \in [2 \ 20]$ mm

The constitutive model (2) possess a simple formulation and is characterized by the following features: (1) history dependence can be considered by the structure tensor; (2) the Matsuoka-Nakai failure surface is included by a simple formulation; and (3) loading and unloading behaviour of clays can be described without adopting an additional loading criterion. Those advantages make this model a stand-alone tool for modelling clastic soils.

NUMERICAL SIMULATION OF CLASTIC SOIL

Clastic soil model

Next, we proceed to obtain the geometry model for the clastic soil. According to the grain size distribution of the clastic soil, the shear zone soil may have a coarse particle distribution similar to that of artificial soil-rock mixtures. Therefore, in the simulation, we will first use an artificial clastic soil with breccias size up to 20 mm to obtain the general mechanical behaviour of the soil.

In the 3D simulation, two groups of artificial samples with different particle gradations, namely group A for $d \in [2 \ 10]$ mm and group B for $d \in [2 \ 20]$ mm, are generated. The breccias are represented by spheres with different sizes. To avoid scale effect in the numerical simulation, the width of the numerical sample is as large as 5 times of the maximum diameter of the breccia (Coli et al., 2011). In this way, the clastic soil sample can be represented by a cubic ($100 \times 100 \times 100$ mm) containing a random distribution of spheres. To generate the geometry model, a *Python* script containing the positions and diameters of the coarse particles is used as an input for the preprocessor of Abaqus/Standard 6.14. The particles are created with the following requirements: (1) the diameter of all particles should be within 2 - 10 mm and 2 - 20 mm for samples in group A and B, respectively; (2) all particles cannot contact each other in the space; and (3) the particle boundary should maintain a minimum distance from the boundary of the representative volume. The minimum distance is the radius

of the corresponding particle. The positions of the breccia are characterized by their centres and diameters. With this method, six numerical samples with six different breccia contents are generated, as shown in Fig. 3. Note that this method also allows us to generated numerical samples with real 3D shapes of breccias by importing their geometric data. Bearing in mind that breccia shape may also influence the mechanical behaviour of the clastic soil. Details can be referred to some experimental and numerical studies (Liu and Lehane, 2012; Zhou et al., 2017).

Experimental studies show that the failure mechanism of the clastic soil is directly correlated to the spatial distribution of coarse particles in the soil (Xu et al., 2008). To consider this factor, the CT technique can be adopted to visualize the real internal structure of the clastic soil. In this study, we make use of the numerical samples by Jiang et al. (2016) and Li et al. (2019), which reference some CT images of the clastic soil from the shear zone of the Huangtupo landslide. Figure 4 shows 10 typical digital images of the clastic samples. The physical samples were originally taken from the investigation tunnel of the landslide by using sampling device in cylinder shape. With the help of a CT scanner, the spatial distribution of breccias in these samples can be obtained. The digital samples are reconstructed from 2D CT-images by using digital image processing technique Xu et al. (2008). To make it easier for readers to compare this study to the literature (Jiang et al., 2016), the same sample names are adopted. The dimension of the sample is 100 mm in width and 200 mm in height. The sample contains breccia content varying from 23.0% to 33.0%. In particular, the samples CT2, CT17, CT19, and CT27 contain some oversized breccias larger than 20 mm, leaving the remaining samples with a relatively uniform breccia distribution. These oversized particles, possessing different shapes and orientation, are unevenly distributed in the soils. These features may lead to scattered results in both experiments and numerical simulations. In addition, the oversized breccias may bring some scale effects into the numerical results. It is

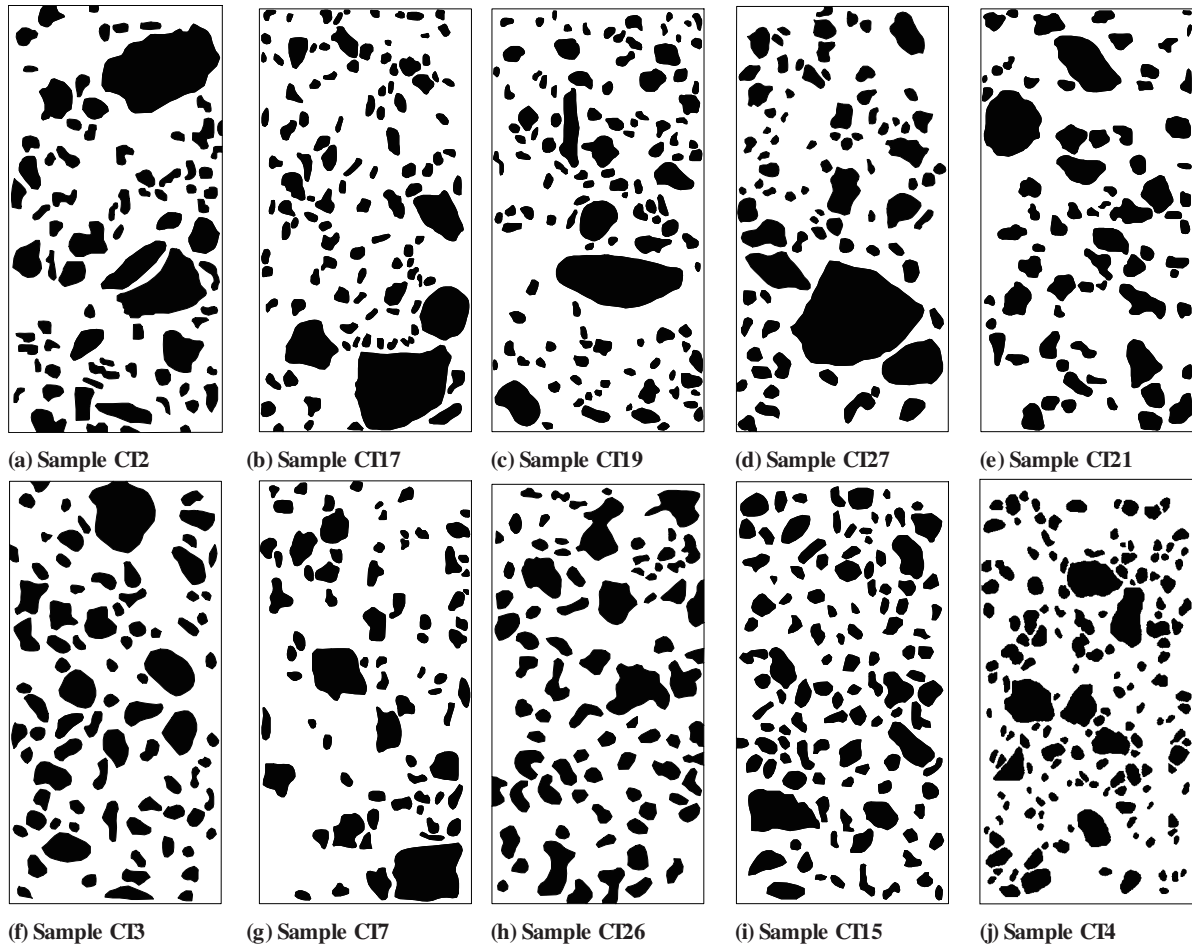


Fig. 4. Digital samples ($W = 100$ mm and $H = 200$ mm) for plane strain test on the clastic soil from the shear zone of Huangtupo landslide. The sample CT4 is referred from (Li et al., 2019), and the others are from (Jiang et al., 2016)

reported that the compressibility and shear strength of clastic soils decrease with the increase of the specimen size (Zhang et al., 2015). One may generate larger numerical samples with the same size of breccias using random distribution theory, but it may spoil some natural characteristics, e.g. particles orientation caused by inherent shearing, of the samples. Note that some numerical samples by Jiang et al. (2016) include breccias that are located at the boundaries. This situation is unrealistic, so the breccia particles at the boundaries are removed in the regenerated FEM samples.

Numerical test program

To investigate the strength, deformation, and failure mechanism of the clastic soil, three groups of tests, namely, the isotropic compression test, triaxial compression test, and plane strain test, are carried out. Specifically, the isotropic compression test and triaxial compression test are used to gain the strength and deformation behaviour of the soil, while the failure mechanism is investigated through plane strain tests. This type of test is widely used in bifurcation analysis because shear localization occurs more easily under the plane strain condition.

For the samples in the isotropic compression test (ICT) and triaxial compression test (TCT), both the soil matrix and the breccia are discretized with a four-node tetrahedral element with three degrees of freedom at a single node. The FEM meshes and boundary conditions of sample AT4 are shown in Figs. 5 (a) and 5(b). A displacement constraint is applied on the bottom surface of the model. In addition, the left and back surfaces are fixed along the X and Z directions, respectively.

A confining pressure is applied on the front and right surfaces during the test.

On the other hand, the samples for plane strain tests (PST) are discretized with four-node plane strain elements. The elements representing the breccias can be determined by making use of the digital images, as shown in Fig. 5(c). The numerical test is performed by applying a displacement boundary condition on the top surface of the numerical sample while keeping the confining pressure constant. An additional boundary condition at the bottom is applied to constrain the vertical displacement. In addition, the middle of the bottom boundary should be constrained to prevent the overall motion along the horizontal direction. For both tests, the compression is conducted by applying a displacement boundary (U_2) upon the upper boundary. Generally, a total of 30 mm of shearing displacement is required to bring the clastic sample to a critical state.

Table 2. Test program on clastic samples with different breccia content

Test	Sample ID	Breccia content ^a	OCR
ICT	AT2-4	5.5 - 15.0%	1
TCT	AT1-4, BT1-2	2.7 - 22.0%	2, 5, 10
PST	all CT samples	23.0 - 33.0%	2, 5, 10

^a Note: breccia content denotes volume and area content for 3D and 2D samples, respectively. For details of the breccia contents in 2D samples, please refer to Fig. 2c.

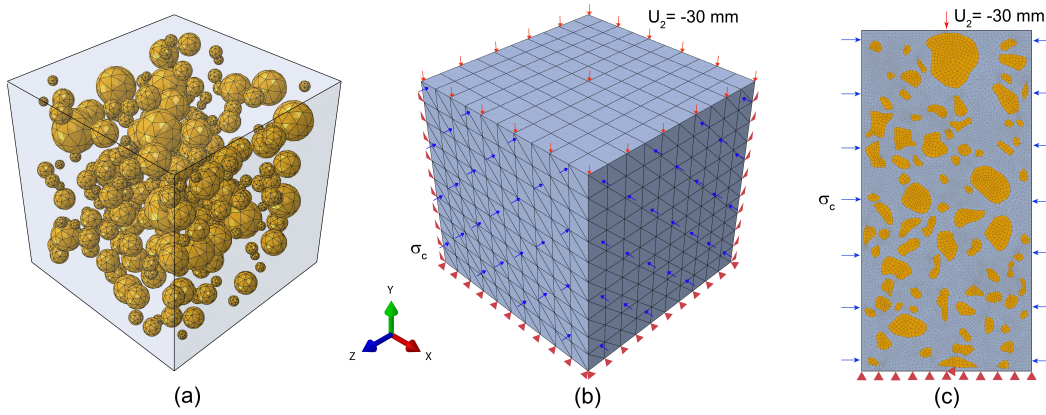


Fig. 5. FEM mesh and boundary condition of clastic samples: (a, b) AT4 sample with 2,007,215 elements for triaxial compression test and (c) CT3 sample with 98,720 elements for plane strain test

The overall test program is listed in Table 2. To consider the influence of OCR, three different OCRs are tested. Before shearing, the soil sample is isotropically consolidated with different preconsolidation pressures and then unloaded to 100 kPa to achieve desirable OCRs. Note that no interface elements are applied between the soil matrix and the breccias, and the excess pore water pressure is not considered in the simulations. It is assumed that the loading velocity is sufficiently slow so that a fully drained process is guaranteed.

Material parameters

To calibrate the hypoplastic model, some laboratory tests, namely, isotropic compression and drained triaxial compression tests, were carried out. Remoulded shear zone soil without coarse particles was used to reconstitute the specimens.

In the isotropic compression test, the specimen is isotropically consolidated to 4000 kPa and then unloaded to 10 kPa. The experimental and numerical results are compared and presented in Fig. 6. It can be seen that the compression behaviour of the homogeneous soil is properly predicted by this model. Moreover, an excellent agreement in the prediction of the unloading behaviour is achieved in the simulations.

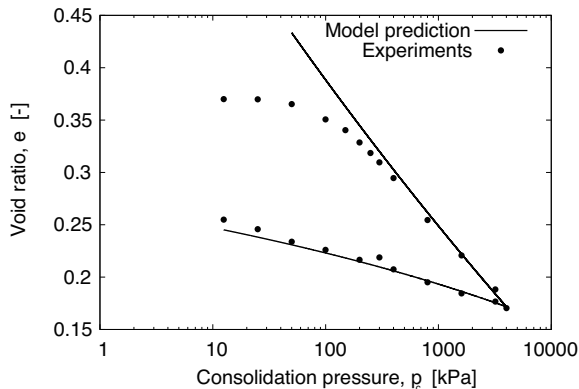
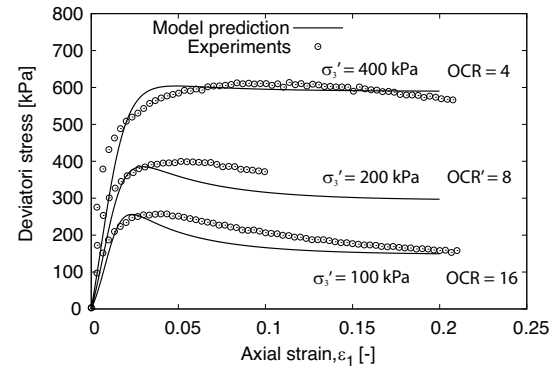
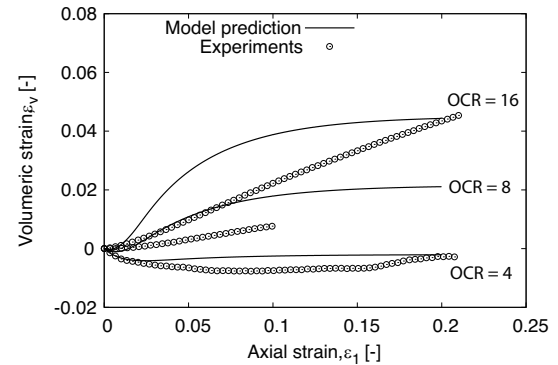


Fig. 6. Comparison between experiments and model prediction on isotropic normal compression test of breccia-free shear zone soil

In the triaxial tests, the specimens are isotropically consolidated to 1600 kPa and unloaded to 100 kPa, 200 kPa, and 400 kPa to produce difference OCRs. The experiments and prediction of the triaxial tests are presented in Fig. 7. Apparent strain softening and shear dilation are observed in the experiments. Basically, the stress-strain behaviour can be properly modelled by the hypoplastic model, while the prediction of volume change is less satisfactory. This result is



(a)



(b)

Fig. 7. Comparison between experiments and model prediction on drained triaxial compression test of breccia-free shear zone soil

related to the values of compression index λ^* and the OCR parameters α , which control both compressibility and shear dilatancy. A better prediction in volumetric variation can be achieved by choosing different parameters for λ^* and α , but this may leads to a worse prediction of the shear strength behaviour at the same time. The constitutive model shows much better performance in predicting the mechanical behaviour of structure soils (Wang & Wu, 2020b).

The calibrated parameters listed in Table 3 are used to model the soil matrix in the clastic soil. Compared to the fine particles in the clastic soil, the breccia has a much higher shear resistance. Previous laboratory investigation of the same soil has shown that the shear resistance of the breccia is sufficiently high to prevent it from being crushed (Li et al., 2019). Therefore, the elastic model is adopted to describe the

Table 3. Material parameters used in the FE simulations

Material	Mechanical parameters	Values
Breccia	Young's modulus: E , MPa	4000
	Poisson's ratio: ν	0.3
Soil matrix	Critical friction angle: ϕ_c, I°	25.1
	Compression index: λ^*	0.46
	Specific volume: N	0.540
	Modulus ratio: r_i	1.3
	OCR parameter: α	0.6

mechanical response of the breccia in the assembly. The typical Young's modulus and Poisson's ratio for argillaceous limestone are used in the simulation.

RESULTS OF THE NUMERICAL SIMULATIONS

The clastic soil consists of the soil matrix and a number of breccias. Apparently, the mechanical response and the resulting stress states will be different in these two substances. To obtain the overall stress and strain developed in the clastic sample, a volume average method proposed by Shi & Herle (2017) is used. The method is described as:

$$\xi = \xi_b C_v + \xi_s (1 - C_v) \quad (9)$$

where ξ is a variable representing the overall outputs, such as hydrostatic stress p , deviatoric stress q , and volumetric strain ε_v . ξ_s and ξ_b are the averaged variables for the soil matrix and coarse particles, respectively, defined as:

$$\xi_s = \frac{\sum_{i=1}^{N_s} \eta_i v_i}{\sum_{i=1}^{N_s} v_i}; \quad \xi_b = \frac{\sum_{i=1}^{N_b} \eta_i v_i}{\sum_{i=1}^{N_b} v_i} \quad (10)$$

where N_s and N_b are the numbers of elements representing the soil matrix and coarse particles, respectively, and v_i is the volume(area) of the i th element.

Compression behaviour

Figure 8(a) shows the test results of isotropic compression tests on the artificial clastic samples. The results are presented in the $\varepsilon_v - \ln(p)$ plane. The compression behaviour of the soil matrix is additionally presented for comparison. Compared to the compression behaviour of homogeneous soil, the overall compression modulus is significantly enhanced by increasing the coarse content. The overall unloading behaviour is also influenced by the coarse fraction. Usually, a clastic sample with a more coarse fraction exhibits less resilience, but the compression behaviour of the soil matrix is hardly affected by coarse particles.

For a homogeneous sample, the stress path of a soil element is defined by the boundary stress conditions. However, the clastic soil exhibits different behaviour. For example, an isotropic stress state at the boundary gives rise to a local shear stress at the interfaces of coarse and fine particles. Figure 8(b) shows the stress path developed in the soil matrix and the clastic samples with different volumetric breccia contents. Basically, in the loading stage, the deviatoric stress increases linearly with the consolidation pressure. This trend is observed for both the overall clastic sample and the soil matrix. In the unloading stage, the deviatoric stress first decreases linearly to a certain value and then increases slightly until a residual value is achieved. Along with unloading, it is found that a residual deviatoric stress remains in the clastic soil, reflecting the interaction of the coarse and fine particles in the sample.

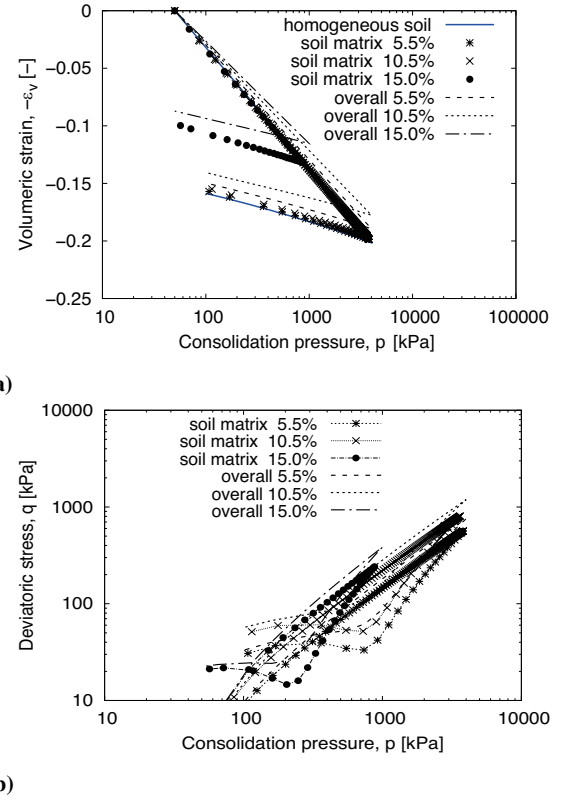


Fig. 8. Isotropic compression test on clastic samples AT2-4 with different volumetric breccia contents (a) $\varepsilon_v - \ln(p)$ and (b) stress path

On the other hand, the coarse particles play an important role in the development of the shear stress. At the same consolidation pressure, the shear stress is larger for the sample with a higher breccia content. Similar numerical results were reported by Shi & Herle (2017) for lumpy soil. This fact suggests a more intensive interaction between coarse and fine particles occurring in the sample with more breccias. Compared to the overall deviatoric stress, the value developed in the soil matrix is much smaller, in both the loading and unloading stages. However, the overall residual shear stress is nearly equal to that in the soil matrix. This result indicates that the residual shear stress mainly remains in the soil matrix.

Stress-strain response

Figure 9 shows the stress-strain results of numerical triaxial compression tests in terms of overall deviatoric stress and overall volumetric strain against the overall axial strain for the artificial clastic samples. A close inspection of the overall stress-strain curves suggests that the initial stiffness increases significantly with both the OCR and the breccia content. For light overconsolidation, say $OCR = 2$, the clastic samples exhibit only strain hardening behaviour, and this mechanical response becomes more obvious with increasing breccia content. This phenomenon agrees with the experimental observations of the triaxial compression test of soil-rock mixtures (Zhao & Liu, 2019). On the other hand, both strain hardening and strain softening can be observed when the samples have experienced heavy overconsolidation. In particular, for the sample AT4 with a 15% volume fraction of breccia, the shear strength first reaches a peak and then undergoes reduction accompanied by strain hardening, which brings the sample to the critical state at a shear strength larger than the previous peak strength. It is interesting to compare the results obtained from AT4 and BT1, which have the same

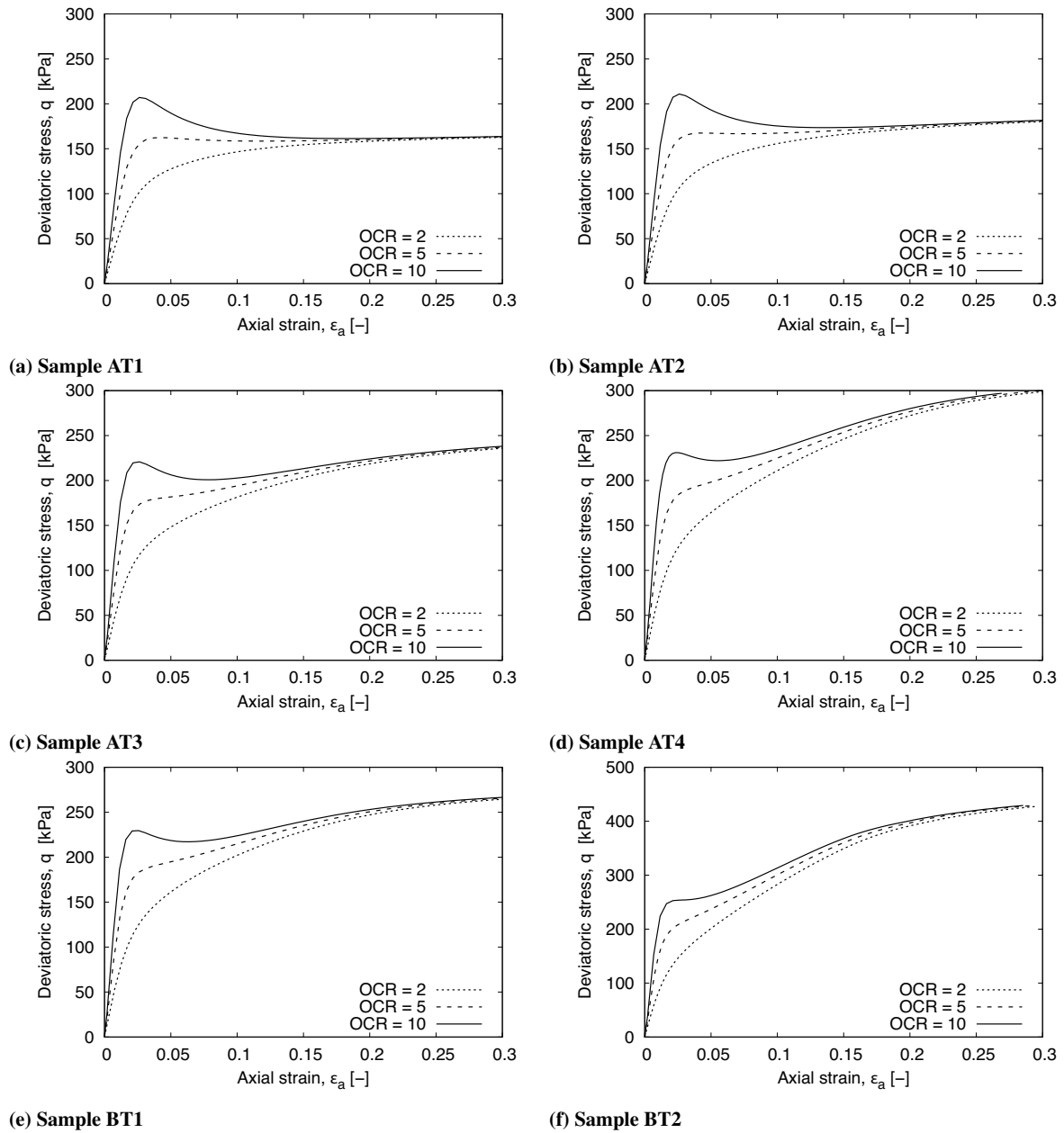


Fig. 9. Overall stress-strain results of drained triaxial compression test on artificial clastic samples

breccia content but different grain distributions. These two samples give rise to very close peak shear strength at prefailure stage, while the AT4 samples yields a higher shear strength at postfailure stage. It seems that the prefailure strength is dictated by soil matrix, while the postfailure strength is related to the breccia distribution. Obviously, with a finer gradation, the AT4 sample contains more coarse particles with a more uniform distribution than that of the BT1 sample (see Fig. 3). Therefore, the failure path in AT4 sample is more likely to be hindered by those breccias, leading to a higher shear strength. It can be expected that, with the largest breccia content, the BT2 sample yields the highest shear strength.

Likewise, the breccia content and the OCR significantly affect the volumetric behaviour of the clastic samples. Figure 10 presents the volumetric changes of the artificial clastic samples during triaxial compression. Generally, the effect of the breccia content on the volumetric behaviour is similar to that of the OCR. With a higher breccia content, the clastic sample is more prone to dilatancy. The BT2 sample contains the highest

breccia content, and thus exhibit the most pronounced dilatant volume change. Moreover, with the same breccia content, a finer gradation leads to more dilatant response. These results indicate that the presence of breccias and their nonlinear movement during deformation increase the dilatancy of the soil sample; this is also visually evident in the triaxial compression tests on soil-rock mixtures by using a CT scanner (Wang et al., 2018d). Basically, dilatation of the clastic soil includes a series of complex behaviours, such as breccia movement, soil matrix consolidation and cracking, and breccia-soil interface cracking and closure.

It is believed that the strain hardening and dilation are related to the interaction of coarse particles and the soil matrix in the sample. Simultaneously, the soil matrix may undergo strain softening and dilation due to overconsolidation. These two effects give rise to a complex mechanical response in the clastic sample. Figure 11 provides insight into this mechanism. In this figure, the mechanical behaviour of the soil matrix is plotted together with the overall response. Under triaxial compression

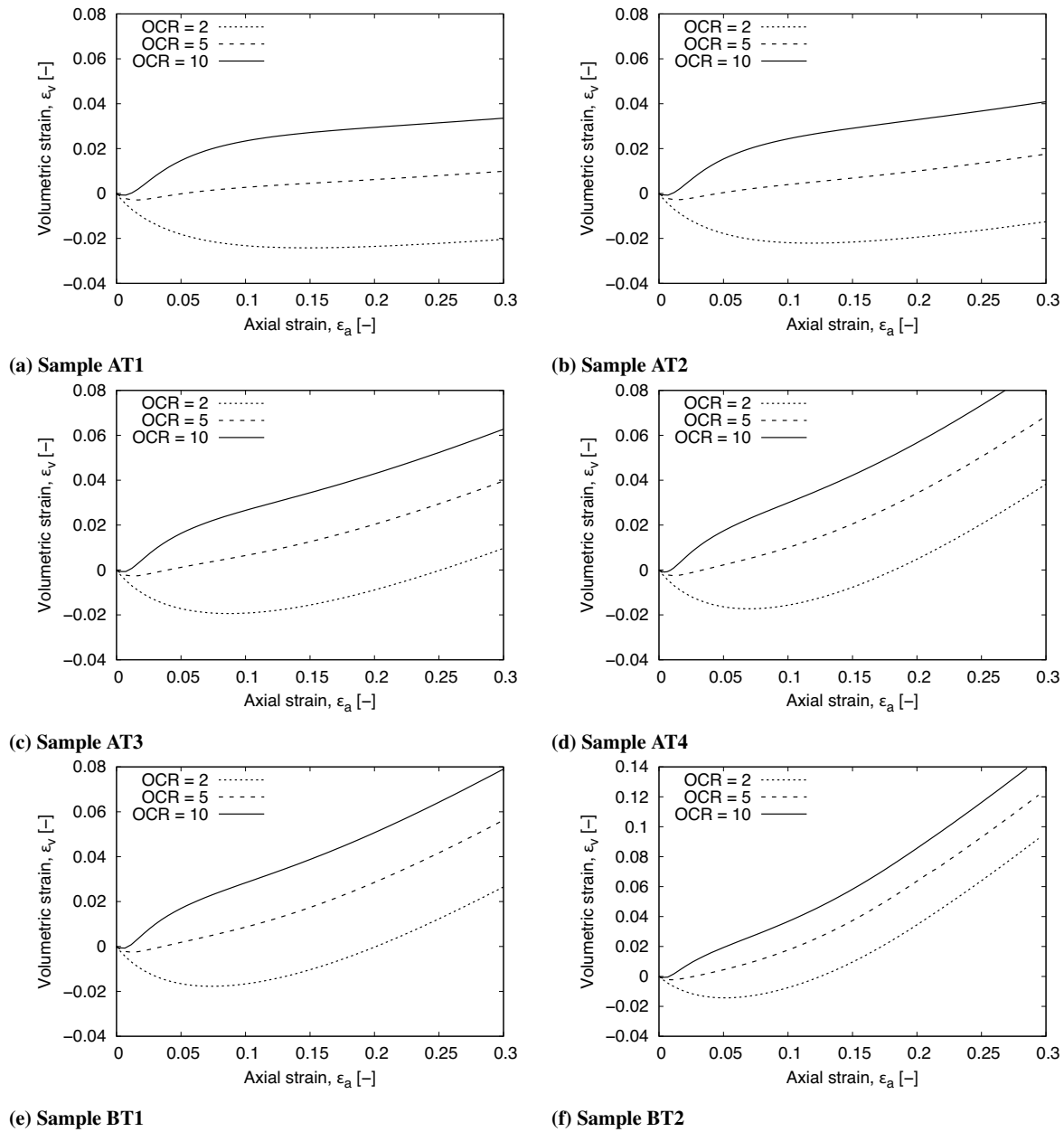


Fig. 10. Overall volume change of drained triaxial compression test on artificial clastic samples

loading, the shear strength of the breccia increases linearly due to its elastic essence. This effect gives rise to an increased overall shear strength at the prepeak stage. When the stress in the soil matrix increases to a state that leads to strain softening and dilation due to overconsolidation, the overall behaviour also shows softening and dilation. Furthermore, the shear strength contributed by the breccia increases continuously, which gives rise to an overall hardening behaviour in the clastic soil. However, the soil matrix is already close to the critical state, and the overall structure cannot be adapted to any further loading. Consequently, the clastic sample collapses and tends towards the critical state. On the other hand, the overall volumetric change is almost the same as the change that occurred in the soil matrix, as shown in Fig. 11 (b). This observation suggests a negligible volume change in breccias because the bulk modulus of the breccia is much larger than that of the soil matrix.

Figure 11 also compares the mechanical response of the homogeneous soil and the soil matrix. Generally, the homogeneous soil and the soil matrix show a similar

mechanical response. Although a significant increase in the overall critical shear strength is achieved in these simulations, only a slight increase in the critical shear stress is observed in the soil matrix. In other words, the critical state of the soil matrix is somehow independent of the fraction of the coarse particles. Despite this feature, significant differences are observed in the volumetric response of the homogeneous soil and the soil matrix.

Figure 12(a) shows the stress paths of the overall clastic soil and soil matrix. Basically, the overall stress path follows the boundary condition of triaxial compression test. The overall critical stress exceeds the critical state line (CSL) of the homogeneous soil with $M = 0.988$. However, the stress paths of the soil matrix diverge significantly from the overall stress path. This is because the clastic sample consists of numerous breccias, and the local shear stresses may change their directions in the vicinity of every breccia. It is observed that this divergence is increased with the grave content; this implies that the critical stress ratios of the soil matrix are related to the breccia content. To show this relation, the critical

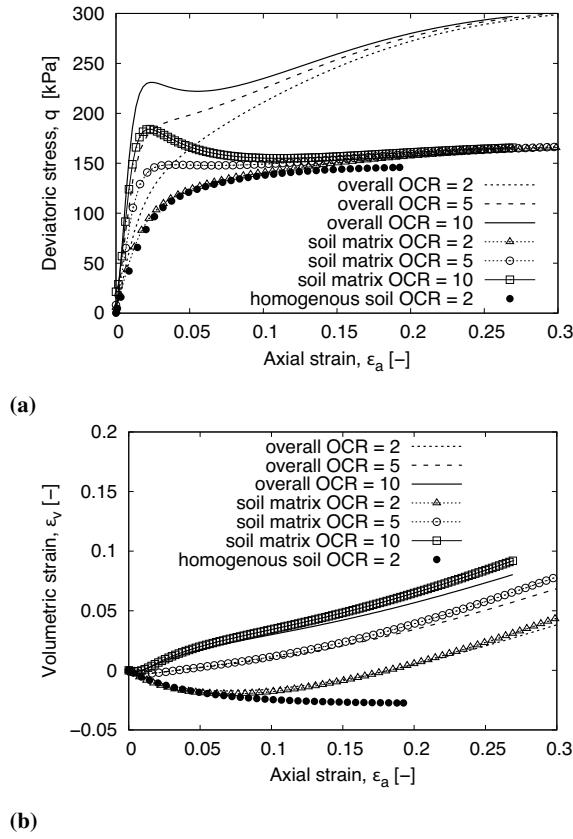


Fig. 11. Comparison of the mechanical responses between the the sample AT4 and homogenous soil (a) ϵ_a - q and (b) ϵ_a - ϵ_v

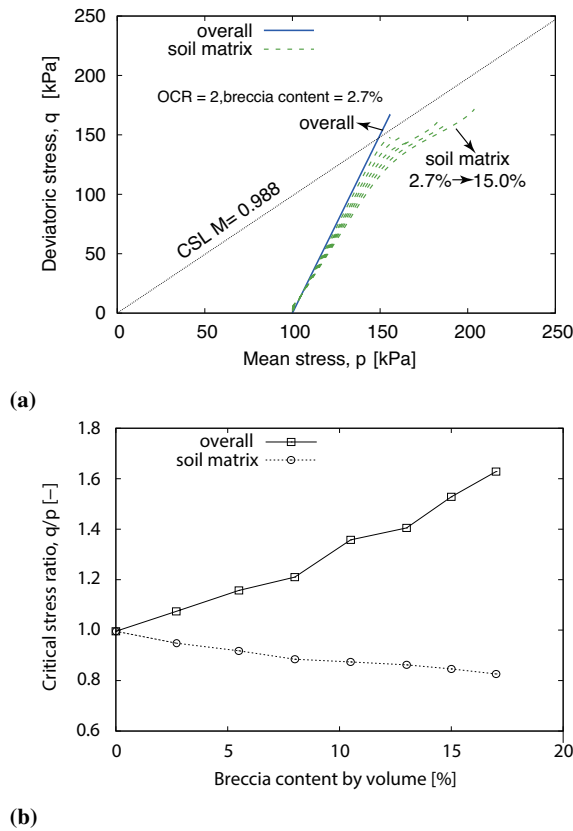


Fig. 12. Results of group A samples with $d \in [2 \ 10]$ mm: (a) stress paths of overall clastic soil and soil matrix with OCR = 2 (b) critical state stress ratio of samples with different breccia contents

state stress ratios of samples with different breccia contents are plotted in Fig. 12(b). Note that three additional simulations with breccia contents of 8.0%, 13.0% and 17.0% are added for comparison. The figure shows that the overall critical state stress ratios increase with the breccia content. In contrast, the critical state stress ratio in the soil matrix decreases gradually with the breccia content. These results imply that at a critical state, the contributions of the breccias to the shear resistance increase with the breccia content. Furthermore, the critical state is more easily achieved in the soil matrix. This type of stress distribution plays an important role in the deformation and failure behaviour of the clastic soils.

Failure mechanism

The failure mechanism of rock-soil mixtures has been extensively investigated with the help of the CT technique, which enables a direct observation of the internal structural change during shearing. It is reported that the failure process of the clastic samples includes a series of particle rearrangements, such as rock-soil interface cracking, forming macro pores, and forming shear localization, and these processes are affected by the breccia content and distribution (Wang et al., 2018e).

Figure 13 shows the localized failure at the end of shearing in the clastic samples with OCR = 2. Obviously, the shearing localizations in these samples are quite diverse, although there is little difference in the coarse content among these samples. Comparison of the shearing localization suggests that the sample may fail along multiple paths or a single path depending on the distribution of the coarse particles. Due to the highly elastic mismatch of breccias and the soil matrix, all localized failures propagate to bypass the breccia. Moreover, the primary direction of the localized failure tends to be dictated by large breccias. For example, the samples CT19 and CT27 contain several bifurcations at the upper part of the samples. Let us recall the coarse particle distribution of these two samples shown in Fig. 4. Both samples contain an amount of oversized breccias at the lower part of the sample, which may serve as obstacles to prevent the development of failure. Therefore, localization occurs mainly at the upper parts. This phenomenon indicates that the breccia distribution has a significant influence on the failure mechanism of the clastic soil.

Now, we confine our attention to the effects of OCRs on the failure mechanism of clastic samples. Figures 14 and 15 compare the localized failures in clastic samples with OCR = 5 and 10, respectively. Basically, for samples with heavy overconsolidation, the failure paths mainly follow a single-path manner. A higher OCR may change the primary failure path in the sample. For instance, the sample CT17 follows a “right-left” primary failure path at OCR = 2 and 5. When the OCR is increased to 10, however, the failure changes to follow a “left-right” path, and a bifurcation is observed at the top of the sample. A close inspection of the failure paths for all samples at different OCRs shows that all the possible failure paths have emerged at OCR = 2, but only one or two of them prevail with increasing OCRs. The investigation on 3D clastic samples shows that a sample with a higher OCR may experience strain softening along with failure. These combined effects may significantly influence the failure mechanism of the clastic samples.

The strength and deformation behaviours of samples CT27 and CT15, representing the sample with the maximum breccia content and that with the most uniform coarse particle distribution, respectively, are shown in Fig. 16. The failure process of the samples at OCR = 2 and 10 are also shown for illustration. Normally, localized failure begins to emerge either in the critical state for light overconsolidation or in the strain softening stage for heavy overconsolidation. For the

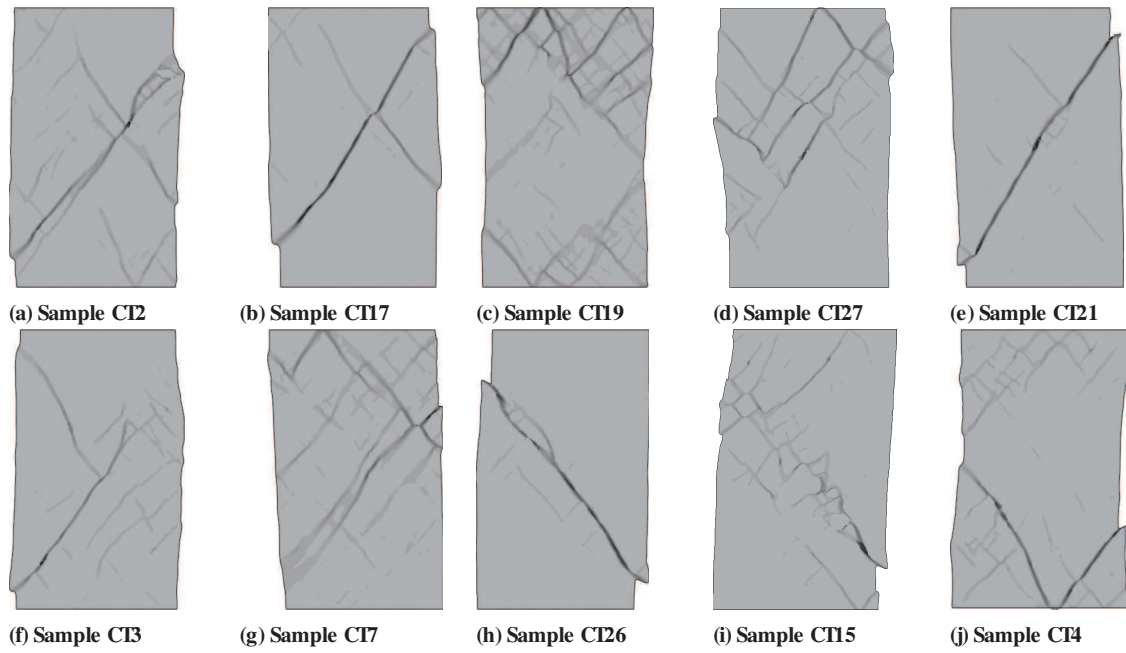


Fig. 13. Localized failure indicated by the maximum principal strain at the end of shearing in the clastic samples with OCR = 2

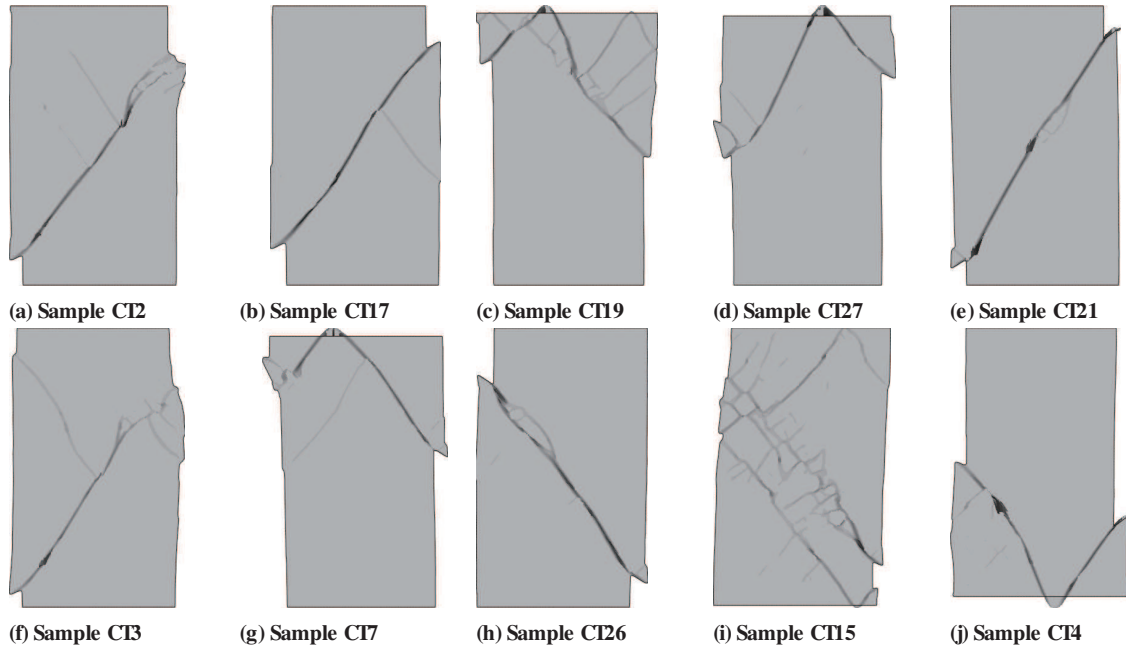


Fig. 14. Localized failure indicated by the maximum principal strain at the end of shearing in the clastic samples with OCR = 5

former, the sample may experience localized failure with very large axial deformation, e.g., approximately 10%, and plenty of secondary shear bands can be seen around the main one. For the latter, however, a prevailing shear band will be formed closely after the strain softening. It is interesting to note that with a uniform coarse particle distribution, the sample CT15 leads to several discontinuous shear localizations forming an inclined shear zone. Accordingly, the sample may first exhibit dilatancy and then experience significant contractive deformation after failure.

Comparison between the deformation behaviour of samples CT27 and CT15 reveals that the uniformity of the distribution of coarse particles in a clastic sample may play a more important role in the resistance of soil deformation than the breccia content. This may due to the significant particle rotation

occurred during the shearing in sample CT15. To show this, we can plot particle rotations in the shear zone at the end of shearing. Figure 17 shows the rotation of two particles located in shear bands of samples CT15 and CT27. It can be seen that the selected particles in sample CT15 and CT27 are rotated by approximately 5° and 1° , respectively. Since the samples with more uniform distribution of coarse particles have higher probabilities to resist shearing failure by rotating the breccias, a higher shear resistance can be obtained in such samples. This result agrees well with what we learned from the 3D simulations on samples AT4 and BT1.

The peak and critical shear strengths obtained from the plane strain tests on different samples with OCR = 5 and 10 are presented in Fig. 18. The breccia content of all samples (CT0 is referred to as a homogeneous soil sample) is also

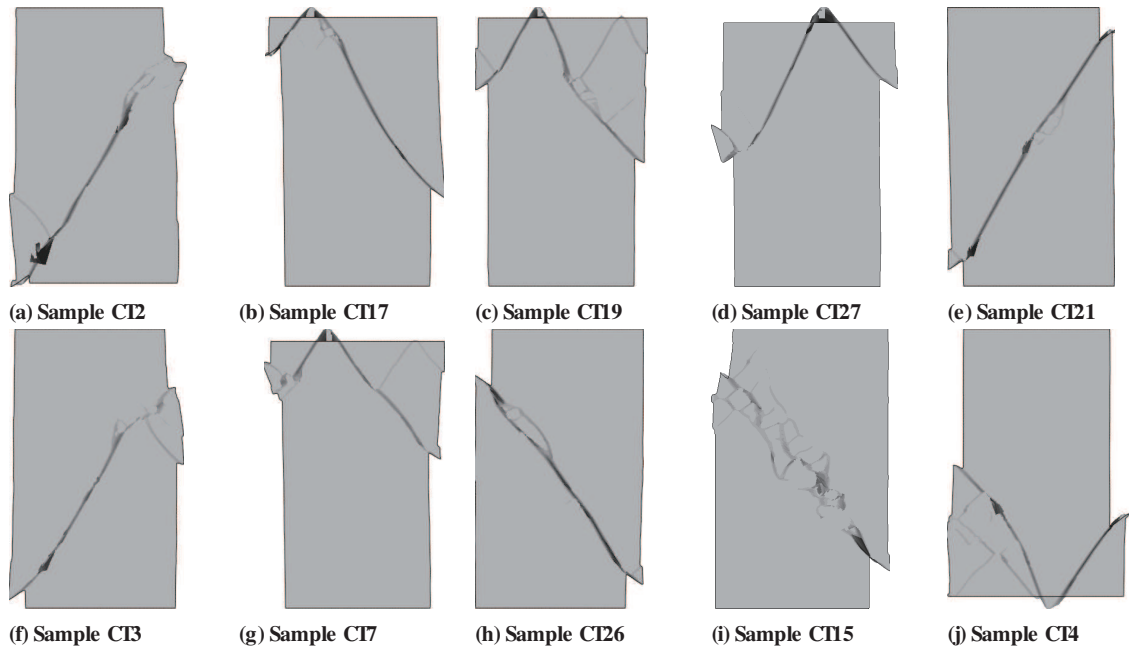


Fig. 15. Localized failure indicated by the maximum principal strain at the end of shearing in the clastic samples with OCR = 10

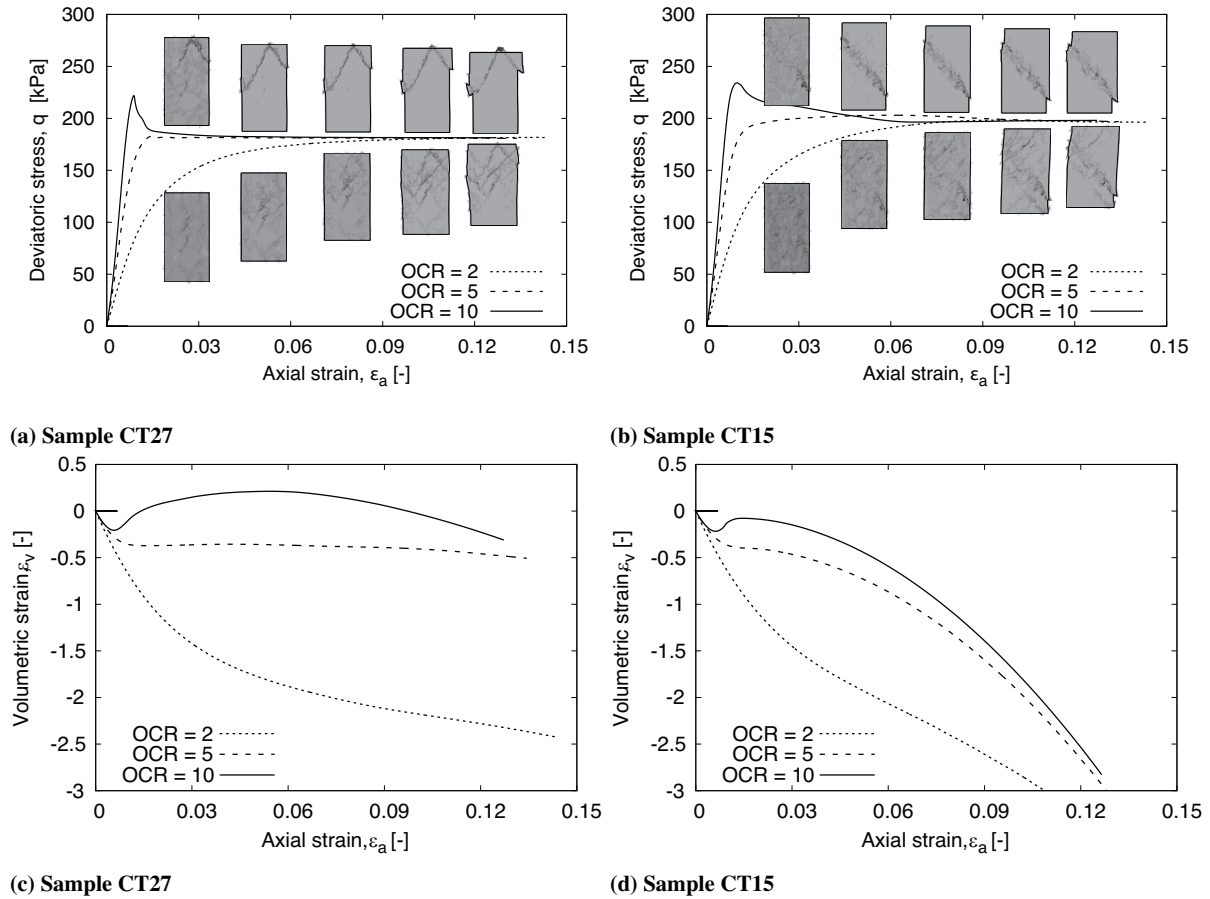


Fig. 16. Numerical results of samples CT27 and CT15 with different OCRs together with the development of shear bands indicated by the maximum principal strain in plane strain tests

shown for comparison. Overall, the shear strength does not increase correspondingly with the breccia content in the plane strain tests. In contrast, most of the clastic samples give rise to peak and critical strengths that are smaller than those obtained from homogeneous soil samples. It is speculated that both the

breccia content and particle distribution significantly influence the mechanical behaviour of clastic samples, while the coarse particle distribution may prevail over the breccia content in controlling the failure mechanism of clastic samples in the plane strain test.

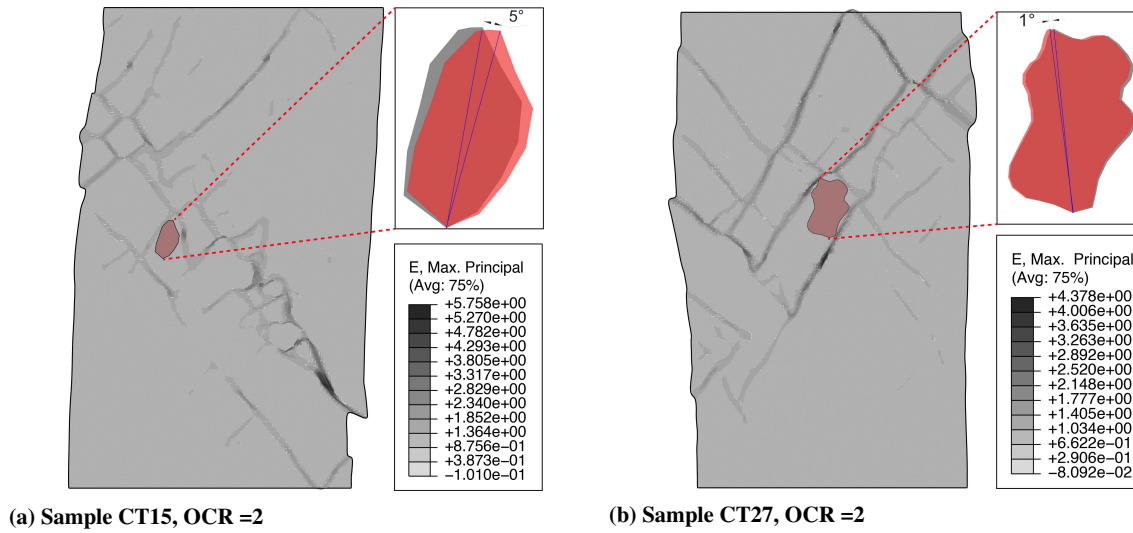


Fig. 17. Rotation of breccias located in shear bands at the end of the plan strain tests on clastic samples CT15 and CT27

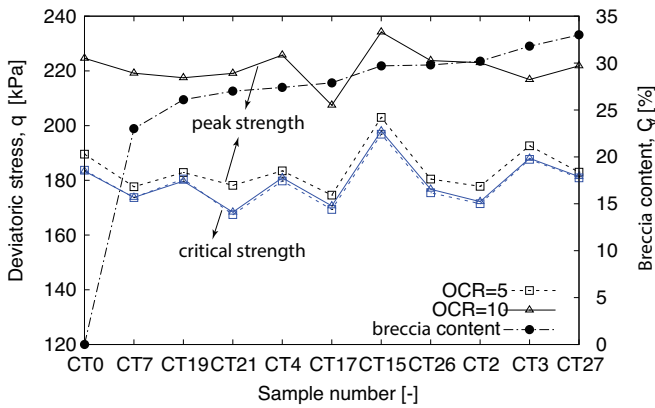


Fig. 18. Peak and critical shear strength of the clastic samples in plane strain tests

DISCUSSION AND CONCLUSION

In this paper, we present a numerical study on the mechanical behaviour of clastic soil from the shear zone of a slow-moving landslide in TGR area. The presence of breccias and the consolidation history on the strength, deformation, and failure mechanism of the clastic soil are investigated by using a novel hypoplastic model.

Generally, the presence of breccias is able to increase the bulk modulus of a clastic sample. Unlike pure soil without breccias, local shear stresses can be developed in the clastic sample under isotropic compression, and the shear stress increases with the breccia content. Moreover, a residual shear stress is found to remain in the soil matrix after unloading, reflecting the interaction of the coarse and fine particles in the sample. These results suggest that coarse particles in shear zone might be able to increase the stiffness of the shear zone, resulting less deformation under the self-weight of the landslide. Therefore, some laboratory tests on reconstituted shear-zone soil with only fine particles may underestimate the deformability of clastic soils. This is particularly significant for predicting the displacement of some slow-moving landslides, for which the shear zones usually account for the majority of the landslide motion (Maio et al. , 2015; Wang et al. , 2018a, 2020d)

On the other hand, both the breccia content and consolidation history can significantly influence the strength and deformation

behaviour of clastic samples under triaxial loading. For light overconsolidation, the clastic samples exhibit only strain hardening behaviour, but the sample with a higher breccia content may experience dilatancy. For heavy overconsolidation, the sample with a relatively high breccia content may first experience strain softening and then strain hardening. In the drained triaxial compression tests, with increasing breccia content, the critical stress ratio of the clastic sample increases, while it decreases in the soil matrix. The influence of the breccia content on the overall mechanical behaviour is realized through changing the stress state developed in the soil matrix.

Based on the results obtained from the plane strain tests on 2D sample with real grain distribution, we found that the distribution of coarse particles has a more important role than the breccia content in controlling the failure mechanism of the clastic soil. A clastic sample with a uniform coarse particle distribution may render a higher shear resistance to prevent the development of failure. This is also confirmed by the 3D simulation with the same breccia content but different grain distributions. In addition, the consolidation history may change the failure path in a clastic sample. It is worth noting that the oversized particles may serve as obstacles to change the localized failure in the soil matrix. This result is similar to those obtained from numerical investigations on the stability and development of failure localization of slopes with rock blocks (Gao et al. , 2018; Liu et al. , 2020).

In this work, we only show the local effect of breccias and overconsolidation on the strength and deformation of clastic soils. A further interesting area is their global effects on the slow motion, e.g. creep, of landslides along the pre-existing shear zones. We speculated that, for such landslides, they may increase the strength of the shear zones, improving the stability of the landslides, but cannot change the shear localization. In addition, even small displacements along the sliding surface could generate pore pressure due to the reduction in volume, leading to a decrease in shear resistance (Bontemps et al. , 2020). The large fraction of breccias in the shear zone may influence the effect of pore pressure in the clastic soil. In this regard, we also need to consider the solid-fluid coupling effect in the shear zone.

DATA AVAILABILITY

The datasets generated during and/or analysed during the current study are available from the corresponding author on reasonable request.

ACKNOWLEDGMENTS

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NOTATION

$C_{v,w}$	volumetric (C_v) and weight (C_w) fraction of breccia
C_{mv}	average volumetric fraction
C_{ma}	average area fraction
d	diameter of breccia
$\mathbf{D}$	strain rate (stretching) tensor
e	void ratio of the soil
E	Young's modulus
f_s	stiffness factor for model (2)
f_v	volumetric factor for model (2)
I_P	plasticity index
I_e	liquid index
I_i	stress invariants ($i = 1, 2, 3$)
M	stress ratio at critical state for MCC model
N	logarithmic value of the specific volume at $p_r = 1$ kPa
$N_{s,b}$	number of soil (N_s) and breccia (N_b) elements
OCR	overconsolidation ratio
p	the mean stress pressure
p_e	the Hvorslev equivalent pressure
p_e^+	preconsolidation pressure in the MCC model
q	the deviatoric stress
r_i	the ratio of initial bulk and shear moduli
R	stress ratio representing overconsolidation
S_r	degree of saturation
$\mathbf{S}$	structure tensor for consolidation history
$\mathbf{T}$	Cauchy stress tensor
v_i	the volume or area of the i th element
w	natural water content
α	parameter for model (2)
γ	natural unit weight
ϕ_c	critical state friction angle
$\rho_{s,b}$	dry densities of soil matrix (ρ_s) and breccia (ρ_b)
λ^*	slope of the normal compression line
ν	Poisson's ratio
$\xi_{s,b}$	averaged variables for soil matrix (ξ_s) and breccia (ξ_b)
$\varepsilon_{v,a}$	volumetric strain ε_v and axial strain ε_a

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