Study on the correlation of macro-fine parameters of surrounding rock in sand and cobble tunnel based on discrete element

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Abstract

As a mixture of soil and rock, the deformation of surrounding rock caused by tunnel excavation in the sandy cobble stratum is affected by the meso-structure of the stratum. In this paper, the mechanical and engineering characteristics of sandy cobble soil are studied in depth using a laboratory large-scale triaxial test and discrete element numerical simulation test, and the effects of main microscopic parameters on the characteristics of sandy cobble soil are analyzed, including the same specimen size, moisture content, compactness, confining pressure as well as loading method. The microscopic parameters such as contact modulus $E_c$, friction coefficient $f_r$, and stiffness bond ratio $n$ of sandy pebble soil can be obtained. Meanwhile, the accuracy and uniqueness of microscopic parameters are determined using the response surface method. Moreover, the influence of different contact modulus and friction coefficient on the macroscopic mechanical properties of sandy pebble soil is quantitatively analyzed. The obtained conclusions are drawn as follows: the elastic modulus of sandy pebble soil decreases with the increase of water content. The peak value point of the stress-strain curve of sandy pebble soil increases with the increase of contact modulus and friction coefficient, respectively. The relationship between peak stress and the two types of fine-scale parameters can both be described by logarithmic functions. Compared to fine-grained soils, sandy pebble soils start to break down at about 3.5% of the axial strain, and the sandy cobble soil can still bear a certain load after reaching the peak strength. The results can provide a reference for the calculation and discrete element simulation of tunnels in sand and pebble strata.

1. Introduction

In recent years, with the continuous development of urban population and land use, the development and utilization of urban underground space have increasingly become a new direction, such as public underground facilities such as rail transit and underground parking lots [1]. However, the geological conditions of these urban underground projects are often very complex. For example, the widely distributed sand and pebble formations in the Chengdu Plain in Southwest China have greatly affected the construction of underground projects such as subways and underground shopping malls [2]. The cobble stratum composed of gravel and loose coarse particles with a different particle sizes of sand and soil. The pores of sandy cobble soil are generally filled with sand with different particle sizes, and the content of medium and fine sand accounts for 30% ~ 50% of the total mass. The particle size of pebbles generally ranges from 2.0 cm to 8.0 cm, and the total mass content of pebbles accounts for 50% to 70% of the composite soil. The internal structure of sand and pebble soil is very complex. Therefore, factors such as void ratio, moisture content, etc., which can characterize the structural characteristics of soil, have a significant impact on the mechanical properties of soil [3]. When open excavation method is used for foundation pit excavation or shield tunnelling method is used for subway tunnel excavation in sandy cobble stratum, the geotechnical stability of the pit wall or surrounding rock is mainly affected by the mechanical
properties of sandy cobble soil, especially shear parameters [4]. In order to further explore the fine simulation method of tunnel excavation in sand and pebble strata and study the mechanical response law of surrounding rock after tunnel excavation, the shear characteristics and Structural meso-parameters of sand and pebble soils were studied by indoor triaxial tests and discrete element method.

Currently, scholars have conducted a great many kinds of research on the mechanical properties of sandy pebble soil using triaxial compressive tests and numerical simulations. Jaroslav concluded that the shear dilation of sandy pebble soil is mainly caused by the occlusion between particles by studying the stress-strain characteristics of non-viscous sandy pebble soil [5]. Seed presented the empirical formula for determining the maximal dynamic shear modulus and the curve of the dynamic shear modulus ratio as well as the damping ratio with the shear strain of the coarse gravel soil using the cyclic triaxial test [6]. Considering the influence of confining pressure, Rollins presented the empirical formulae of dynamic shear modulus ratio, damping ratio, and shear strain [7]. Hardin B O and Kalinski M E [8] tested the relationship between shear modulus and shear strain of prepared sandy pebble material under small deformation conditions, and they estimated the shear modulus of elasticity using the large indoor triaxial test. Pottyondya and Candall [9] simulated rock microscopic strength by setting the bond relation of particle contact. Y. H. Wang and S. C. Leung [10] explored the mechanical behavior of artificial sand consolidation based on a triaxial test and numerical simulation, the results show that the strength and dilation effect of Portland cement sand is more obvious with the increase of cement content. Du Haiming et al [11] analyzed silty sandy soil containing ice freezing at different moisture content and under confining pressure using a triaxial cyclic loading and unloading test. The obtained results show that with the increase of moisture content, the plasticity of frozen soil increases at first and then decreases. Ma Lin et al [12] analyzed frozen sandy soil subjected to confining pressure at different temperatures using the triaxial shear test, they found that considerable particle failure occurs in the process of the triaxial shear test at the temperature of $-0.5 \ ^{\circ}C,-1 \ ^{\circ}C,-2 \ ^{\circ}C,-5 \ ^{\circ}C$ and confining pressure of 0.5 MPa, 2 MPa, 5 MPa, 10 MPa respectively. In addition, the particle failure rate increases with the increase of confining pressure, which leads to a decrease in the shear strength of frozen sandy soil. Xu Gang, Pan Shu, Ji Meixiu, and Hao Yuan [13] introduced a new analytical finishing method to study the dynamic strength of coarse-grained soil. Wang Ruheng [14] and Wu Huaizhong [15] analyzed the effects of confining pressure, consolidation ratio and vibration frequency on the dynamic strength of sandy pebbles with different saturation using the indoor triaxial test. Wang Jinguang [16] designed an HHC-CA model based on the cellular automata method combined with a triaxial test in which coarse-grained soil with different grain initial structures is prepared and then the triaxial numerical simulation of coarse-grained soil was conducted by using FLAC$^{3D}$. On the basis of the triaxial consolidation drainage test, Geng [17] analyzed the particle flow model and the stress-strain relationship in triaxial test, as well as the effect of microscopic parameters on the strength of coarse-grained soil using PFC$^{3D}$. Zhou [18] performed an indoor biaxial test to simulate the formation and development of a shear zone in the sand, and discussed the change situation in the macroscopic properties of specimens by changing the meso-structural parameters including particle size and friction coefficient based on the theory of particle flow. Li Dayong [19] found that numerical tests can replace indoor tests to some extent while simulating the direct shear test using. Qin Shanglin et al [20] presented that the stress-strain curve shows weak strain-softening or strain hardening under low confining pressure, and its shape mainly depends upon the magnitude of confining pressure in large-scale triaxial tests. Sha Mand [21] developed a new method and calculation formula to modify the results of samples of different sizes by studying the shear strength of gravel-mixed soils with poor grain size distribution. Shi Zhenming [22] proposed that the triaxial test on coarse-grained soil under high pressure should be further studied, including complex stress states, dynamic load, numerical simulations as well as different stress paths based on a large number of triaxial tests. Wang Jinguang [23] studied the shear strength and particle failure rule of coarse-grained soil with different coarse grain content using a large direct shear test. However, the determination of microscopic parameters and the influence of microscopic parameters on the macroscopic mechanical properties of sandy pebble soil have not been systematically studied in depth. Zhao Zhikai [24] using the applicability and limitation of the MHCS model for fine sand containing hydrate are verified by comparing with the simulation and experimental results for drained and undrained triaxial tests. Zeng Kai(feng) [25] conducted a series of large-scale triaxial consolidation drainage tests on reinforced and unreinforced calcareous sand in order to study the mechanical properties of reinforced calcareous sand. The testing variables included confining pressures, types of reinforcing materials and the number of reinforcement layers. However, the previous research mainly used the triaxial test to study the macroscopic mechanical properties of soil samples, and the research on the microscopic parameters were not accurate enough, and the influence of the microscopic parameters on the macroscopic mechanical properties of sandy pebble soil was not studied.

The stress-strain curve of sandy pebble soil is obtained using both the indoor triaxial compressive test and the discrete element method in this paper. Additionally, a cylindrical discrete element model of sandy pebble soil is also established by PFC$^{3D}$. By comparing the triaxial test results with the ones obtained from the discrete element numerical simulation, the microscopic parameters of sandy pebble soil, including contact modulus $E_c$,
friction coefficient $f_i$ and bond stiffness ratio $n$ are obtained. In this paper, the empirical method and the response surface method are used to determine the mesoscopic parameters of sand and gravel soil for the first time, so that the numerical simulation of the obtained mesoscopic parameters is more in line with the actual situation of the soil layer. On this basis, the influence of different friction coefficients and different contact modules on the macroscopic mechanical properties of sandy pebble soil is also analyzed. Then the relationship between microscopic parameters and macroscopic mechanical parameters is obtained through a large number of numerical simulation experiments. The results can be used to provide the theoretical basis for the stability analysis and safety evaluation in real engineering, such as subway stations and underground tunnels in sand and cobble soils.

2. Experimental design

2.1. Instruments used in the indoor triaxial test

To perform an indoor triaxial compressive test on the properties of sandy pebble soil specimen, YLSZ30–3 stress-typed large-scale triaxial test machine with a pressure chamber of $\Phi 300 \times 600$ mm is used and as shown in figure 1. Due to the limitation in the size of the pressure chamber, the largest particle size in sandy pebble soil specimen is set within 60 mm. The test is performed in a strain control room with an axial strain rate of 1.5 mm min$^{-1}$. The test is stopped when the axial strain reaches 20%. The experimental sampling depth was between 5 m and 15 m. To better simulate the experimental environment on-site, the confining pressure of the triaxial test was set as 100 kpa, 200 kpa, 300 kpa according to the effective stress theory, and the axial stress path is shown in figure 2.

1. Loading column (The loading speed is set within 1.5 mm min$^{-1}$); 2. Ergometer; 3. Pressure chamber containing sandy pebble soil; 4. Confining pressure control room (Water is used to exert confining pressure); 5. Water inlet; 6. Top plate; 7. Bottom plate; 8. Water outlet; 9. Machine base; 10. Water pressure java script tube; 11. Confining pressure control room; 12. Load control room.

2.2. Indoor tri-axial test scheme and results

In this test, the sandy pebble soil in several construction sites on Chengdu Metro lines was sampled as the research object. Due to the limitation of the grain size of sandy pebble soil, the original grain gradation of sandy pebble soil needs to be treated. There are generally three methods for treating oversized particles in sandy pebble soil, that is, elimination method, the equivalent substitution method, and similar gradation method. In this test, the equivalent substitution method is adopted to treat oversized particles in sandy pebble soil.

The equivalent substitution method is employed to replace the oversized particles in coarse-grained soil with the maximum allowable particle size $d_{max}$ less than 5 mm by the content weighted average, and then the particle content of each particle size is calculated as per the following formulae.
Where \( P_5 \) denotes the coarse material content, \( P_{5i} \) means the specific fraction content of \( d > 5 \) mm after treatment, \( P_{05i} \) refers to the particle size content corresponding to \( P_{5i} \) before treatment, \( P_0 \) indicates the oversized particle content, %.

The advantage of performing equivalent substitution is that the gradation of coarse-grained material percentage remains unchanged after replacement or substitution, and the content and property of the fine particles also remain unchanged. In this test, the natural gradation of sandy pebble soil is converted into the simulated gradation based on the equivalent substitution method, and groups of particle gradation are obtained by calculation and as shown in table 1. The process of specific sampling and its preparation are shown in figures 3(a) and (b) respectively. In figure 3(b), the outer sleeve of the specimen is made of rubber, while the outermost steel barrel serves as temporary support.

After the test, the ballooning phenomenon occurs in the middle of the specimen, as shown in figures 4(a) and (b), respectively. The stress-strain curves obtained from the conventional triaxial test under different confining
pressures possess no obvious peak value. Additionally, a degree of strain hardening phenomenon occurs. The obtained stress-strain curves are illustrated in figure 5. The Mohr’s stress circles revealing the failure of the specimen are shown in figure 6.

It is seen from figure 4 that the stress-strain curve of sandy pebble soil is not a completely smooth one, and the undulation phenomenon still occurs on the curve, but its overall trend is quite clear. This is because the triaxial shear test of sandy pebble soil is mainly based on point contact, and the stress concentration phenomenon appears at the contact point. In some local zones in the specimen, shear stress is much greater than the shear strength of the sandy pebble, which makes pebble particles broken, so the undulation phenomenon appears in its deformation curve.

It is seen from figure 5 that with the increase of confining pressure, the tangent slope and linearity of the stress-strain curve of sandy pebble soil increase to a certain extent, and the initial tangent modulus of sandy pebble soil also increases gradually with the increasing of confining pressure. At the initial stage of the curve, the slope of the stress-strain curve tends to decrease with the increase of water content, that is, the elastic modulus of sandy pebble soil decreases with the increase of water content.

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The shear parameters of the specimen obtained from Mohr’s circle while the specimen is broken are shown in table 2. As is seen from table 2 that the internal friction angle increases with the decrease of moisture content while the density of sandy pebble soil remains unchanged. But the cohesion increases with the decrease of moisture content. This is because the decrease in water content increases the strength of sandy cobble soil, then the cohesion and internal friction angle will increase with the decrease of water content.

3. Results and discussion

3.1. Determination of microscopic parameters of sandy pebble soil

3.1.1. Setup of sandy pebble soil model and application of its parameters

To simulate the mechanical behavior of sandy pebble soil under conventional triaxial compression with the numerical method, PFC3D is employed in this paper and the following procedures in the numerical simulation are observed to obtain the microscopic parameters of sandy pebble soil.

The parallel bond model can be simplified into two springs appearing in the particle contact surface, which have constant directional stiffness and tangential stiffness. They are different from the simplified spring of the...
Figure 5. Stress-strain curve of samples.

Figure 6. Mohr’s circle of samples.
contact bond model in that they do not have stiffness. The model is shown in Figure 7. The model boundary is formed by setting walls to constrain particle units. FISH language is used to control the movement rate of the wall to apply an axial force to the model.

(1) Preparation of soil sample. First of all, a cylindrical container is simulated as the same one generally used in indoor triaxial compression, which is composed of two bottom surfaces and a cylindrical wall. The size of the container is 0.3 m in diameter and 0.6 m in height. The target ball, namely the large particle in the model, is then generated inside the cylinder. Secondly, the generated particle information, including particle center coordinates and particle radius, is extracted, and stored in the array. Then, the stored large particle information is used as the data source to form large particles with small particles and classified as a clump. Finally, the created numerical triaxial specimen is then subject to particle gradation in an indoor test, and the ‘floating’ particles that have less than three points of contact with adjacent particles are eliminated, and the sample reaches the initial equilibrium state through the preceding circulation.

(2) Application of confining pressure: In the numerical test in this paper, three different groups of confining pressures at 100 kPa, 200 kPa and 300 kPa are applied to each group of specimens respectively. The confining pressure is achieved by applying an external force that is perpendicular to the boundary particle, and the confining pressure is kept unchanged during the whole process of loading. This procedure aims to apply force on the side wall of the sample and allow the confining pressure to reach the predetermined value, then apply the contact force on the boundary particles, and finally delete the displacement constraint of the boundary particles to effectuate the application of confining pressure.

(3) Vertical displacement load on specimen: Vertical displacement load is controlled by the descent speed of boundary particles. In the process of displacement loading, too fast loading is not allowed since it affects the convergence of the triaxial experimental model and the mechanical properties of sandy pebble soil are related to the loading rate.
The numerically triaxial compressive test for sandy pebble soil specimen is simulated using PFC3D, and the particle composition of the numerical specimen in the triaxial test is simulated by using Fish language embedded in PFC3D. Moreover, the number of particles generated in PFC3D nearly amounts to 40,000. The numerical simulation diagrams of sandy pebble soil specimen are shown in figure 6. It can be seen that different colors represent particles of different sizes, where large pebbles are composed of several particles by CLUMP command in PFC. On this basis, by adjusting such microscopic parameters as contact modulus $E_c$, friction coefficient $f_r$, and stiffness bond ratio, the stress–strain curve obtained from the numerical simulation is consistent with that derived from indoor experiment. The microscopic parameters such as contact modulus $E_c$, friction coefficient $f_r$, as well as stiffness bond ratio $n$ of sandy pebble soil obtained from numerically triaxial compressive simulation are shown in table 3, and the comparison between the stress–strain curves obtained from the indoor test and discrete element method is shown in figure 8.

As shown in figures 9(a)–(c), with the increase of confining pressure, the tangent slope and linearity of the stress–strain curve of sandy pebble soil increase to a certain extent and the initial tangent modulus of sandy pebble soil also increases gradually with the increase of confining pressure. At the initial stage of the curve, the slope of the stress–strain curve tends to decrease with the increase of water content, that is, the elastic modulus of sandy pebble soil decreases with the increase of water content. The stress-strain curves obtained by numerical simulation are consistent with those obtained by a laboratory test, indicating that the mesoscopic parameters obtained by the empirical method have certain accuracy and can well simulate laboratory tests.

It can be seen from figure 9(d) that, at the elastic stage, the specimen has no deformation basically, and the shear surface of the specimen is basically formed when the stress peak is reached. With the increase of strain, the complete shear surface is finally formed. This is because the early pressure is small, which is not enough to cause the specimen to break, but with the increase of axial pressure, the specimen will break beyond its bearing capacity, and the greater the pressure, the larger the rupture surface.

The stress-strain curve of the numerical model is well consistent with the experimental curve, even though the volatile of the numerical simulation curve is less than indoor test curve. It means that the microscopic parameters of the discrete element determined thereby can well embody the macroscopic mechanical properties of sandy pebble soil. It can be seen that when the confining pressure is constant in the triaxial test, the stress peak value of the sand and pebble surrounding rock increases gradually with the increase of the coarse grain content, but the initial modulus is different. This is mainly because the basic elements of discrete elements are rigid spheres that cannot be broken, the occlusal force between particles is weaker than that of pebbles, and the particle arrangement is relatively simple. Even if contact bonding is chosen to increase the cohesion between particles, it still leads to bond cracking with progressively higher pressure loads. As a result, there is a certain difference in the strength characteristics of the actual sand and pebble surrounding rock, which is also a problem.
3.1.2. Calibration of microscopic parameters by using the response surface method

Initial calibration of microscopic parameters of sandy pebble soil can only be conducted by using a set of microscopic parameters. This makes numerical simulation results more consistent with experimental ones. However, both the rationality and reliability of the selection of microscopic parameters are still open problems to be addressed. To solve this problem, the response surface method is used to determine the microscopic parameters of sandy pebble soil via back analysis.

Since 1951, the response surface method proposed by Box and Wilson has been widely used in scientific studies. The response surface method is used to develop, improve, and optimize the statistical and mathematical properties. The response in the response surface is also regarded as sensitivity, according to the properties...
response test regression for high-dimensional surface, it provides a method for constructing the approximate model. Selection of response surface function is typically used in the linear or quadratic polynomial. A batch of PFC$^{3D}$ numerical experiments is used as samples, and the quadratic polynomial without cross terms is selected as the response surface function. To conduct the inversion of sandy pebble soil microscopic parameters and the functional relationship among principal stresses, expressions are presented as follows.

$$s_k(x) = a + \sum_{i=1}^{3} b_i x_i + \sum_{i=1}^{3} c_i x_i^2$$  \hspace{1cm} (3)

$$\mathbf{x} = [\bar{x}_1, \bar{x}_2, \bar{x}_3]^T = [\bar{E}_c, \bar{k}_n/\bar{k}_s, \bar{f}^T]$$  \hspace{1cm} (4)

$$\bar{E}_c = \frac{E_c}{\bar{E}_c} = \frac{k_n/k_s}{\bar{k}_n/\bar{k}_s} \quad \bar{f} = \frac{f}{\bar{f}}$$  \hspace{1cm} (5)

Where $s_k$ denotes the deviator stress of the load step $k$; $a$, $b_i$ and $c_i$ indicate the undetermined coefficients of the polynomial; $x$ represents the dimensionless independent variable vector; $E_c$, $k_n/k_s$, and $f$ refer to dimensionless contact modulus, dimensionless stiffness ratio and dimensionless friction coefficient respectively; $E_c$, $k_n/k_s$, and $f$ are related to the empirical estimations of the microscopic parameters of sandy pebble soil. Taking the first load step as an example, the mathematical operation of equation (3) yields series equations listed from equation (6) to equation (12).

$$s_1^k(x) = s(E_c, \bar{k}_n/\bar{E}_c, \bar{f})$$  \hspace{1cm} (6)

$$s_2^k(x) = s(E_c + \Delta E_c, \bar{k}_n/\bar{E}_c, \bar{f})$$  \hspace{1cm} (7)

$$s_3^k(x) = s(E_c - \Delta E_c, \bar{k}_n/\bar{E}_c, \bar{f})$$  \hspace{1cm} (8)

$$s_4^k(x) = s(E_c, \bar{k}_n/\bar{E}_c, \bar{f} + \Delta \bar{f})$$  \hspace{1cm} (9)

$$s_5^k(x) = s(E_c, \bar{k}_n/\bar{E}_c, \bar{f} - \Delta \bar{f})$$  \hspace{1cm} (10)

$$s_6^k(x) = s(E_c, \bar{k}_n/\bar{E}_c, \bar{f} + \Delta \bar{f})$$  \hspace{1cm} (11)

$$s_7^k(x) = s(E_c, \bar{k}_n/\bar{E}_c, \bar{f} - \Delta \bar{f})$$  \hspace{1cm} (12)

Where $\Delta E_c = 0.1$, $\Delta \bar{k}_n/\bar{E}_c = 0.1$, $\Delta \bar{f} = 0.1$, these are used to indicate the increment of dimensionless contact modulus, the increment of dimensionless stiffness ratio as well as the increment of dimensionless friction coefficient, respectively. $s$ means the deviator stress under the 7 parameter combinations in the first load step, and it is calculated by using PFC$^{3D}$. It is seen from equation (6) to equation (12) that there are seven equations and seven undetermined coefficients, So Matlab is used to address the linear system of equations, and analogous processing is also conducted in other load step cases. After the response surface function is determined, the objective function is defined as the mean square error between the calculated deviator stress of the specimen and the experimental value, the objective function is expressed as

$$\min f = \sqrt{\frac{1}{N_k} \sum_{k=1}^{N_k} \left| s_k^*(\mathbf{x}) - s_k^{\text{obs}} \right|^2}$$  \hspace{1cm} (13)

Where $f$ stands for the objective function, $s_k^{\text{obs}}$ denotes the observed values of deviator stress in load step $k$, $N$ is the total load steps, in this case $N = 10$.

Quasi-Newtonian method is one of the most effective method to address the problem of unconstrained optimization, both DFP (Davidon-Fletcher-Powell) algorithm and BFGS (Broyden-Fletcher-Goldfarb-Shanno) algorithm possesses higher efficiency. It is generally believed that the latter shows better stability in numerical simulation than the former. So BFGS algorithm is employed to set up an optimization program and calculation is performed. Accordingly, the microscopic parameters of sandy pebble soil are derived through back analysis and as shown in table 4.

It is seen from table 3 and table 4 that the values of microscopic parameters obtained from the experimental method and the response method are basically the same, in which the correlation coefficient $R^2$ of $E_c$ is 0.999 and that of $f$, is 0.986. However, the microscopic parameters calculated by using the response surface methods possess higher accuracy and reliability. In addition, the back analysis to obtain microscopic parameters of sandy pebble soil based on response surface method is used as a sample analysis by using 7 sets of numerical tests, its reliability is also improved in comparison with the single trial method. This is because the empirical method is based on the laboratory test, through constant adjustment of data to get the mesoscopic parameters, numerical ratio is not unique, and the data accuracy also needs to be improved. Through the effective combination with response surface method, this shortcoming can be overcome perfectly, and the obtained mesoscopic parameters are more consistent with the reality.
3.2. Influence of microscopic parameters on macroscopic mechanical properties of sandy pebble soil

According to Table 3 and Table 4, the contact modulus $E_c$ and friction coefficient $f_r$ changed greatly when the moisture content was kept different, while the bond stiffness ratio $n$ remained basically unchanged. Therefore, the influence of different contact modulus and friction coefficient on the macroscopic mechanical properties of sandy pebble soil is mainly discussed. PFC$^{3D}$ is used to simulate the triaxial compression of sandy pebble soil with different contact modulus and friction coefficient at confining pressure of 100 kPa. The stress-strain curve of sandy pebble soil with different contact modulus or friction coefficient and the relationship between principal stress difference and contact modulus as well as friction coefficient is also obtained.

### 3.2.1. Influence of particle contact modulus on macroscopic mechanical properties of sandy pebble soil

The stress-strain curves of sandy pebble soil with different contact modulus $E_c$ obtained through PFC$^{3D}$ simulation are shown in Figure 8, and the peak values of stress-strain curves under different contact modulus are also shown in Table 5.

As is seen from Figure 10 that with the increase of contact modulus $E_c$, the initial tangential modulus of sandy pebble soil tends to increase, the peak stress also increases, and the axial strain at the time of failure decreases correspondingly. As is also seen in Table 5, when the contact stiffness increases from 5 MPa to 40 MPa, the peak stress increases from 0.216 MPa to 1.989 MPa. The following equation (14) is used to fit the functional relationship between $\sigma_1 - \sigma_3$ and $E_c$, as shown in the figure 11.

\[
\sigma_1 - \sigma_3 = a \ln (E_c) + b
\]  

(14)

Where $\sigma_1 - \sigma_3$ represents the principal stress difference, in MPa; $E_c$ is the contact modulus, MPa; $a$ and $b$ are dimensionless test parameters to be determined. As is also seen from Figure 11, the correlation coefficient $R^2$ of the fitting curve reaches 0.9922 where $a = 0.8284$ and $b = -1.0831$.  

### Table 4. Microscopic parameters of sandy pebble soil.

| Moisture content | Contact modulus $E_c$ | Friction coefficient $f_r$ | Bond stiffness ratio $n$ |
|------------------|-----------------------|---------------------------|------------------------|
| 1.5              | 80.394                | 0.284                     | 1.982                  |
| 4                | 20.314                | 0.352                     | 2.084                  |
| 6                | 39.568                | 0.389                     | 1.995                  |

### Table 5. Principal stress difference in specimen with different contact modulus.

| Contact modulus $E_c$/MPa | Principal stress difference $\ (\sigma_1 - \sigma_3)/MPa$ |
|---------------------------|----------------------------------------------------------|
| 5            | 0.216                                                    |
| 10           | 0.909                                                    |
| 20           | 1.332                                                    |
| 40           | 1.989                                                    |

Figure 10. Stress-strain curves of soil specimen at different particle contact modulus.
3.2.2. Influence of friction coefficient on macroscopic mechanical properties of sandy pebble soil

The stress-strain curves of sandy pebble soil with different friction coefficients obtained through PFC3D simulation are shown in the figure 12, and the peaks of the stress-strain curves under different friction coefficients are also shown in table 6.

As is seen from figure 12, when other microscopic parameters are kept unchanged, the peak value of the stress-strain curve also increases with the increase of the frictional coefficient of sandy pebble soil, and the initial elastic modulus of sandy pebble soil also increases with the increase of the friction coefficient, indicating that the frictional coefficient of particles has a great influence on the strength of sandy pebble soil. As is also seen in table 6, when the frictional coefficient increases from 0.2 to 1.0, the peak stress increases from 0.828 MPa to 1.899 MPa. As a result, equation (15) is presented to fit the functional relationship between $\sigma_1 - \sigma_3$ and $f_r$, as shown in the figure 13.

$$ (\sigma_1 - \sigma_3) = c (f_r)^2 + d (f_r) + e $$

Where $\sigma_1 - \sigma_3$ is the principal stress difference, in MPa; $f_r$ is friction coefficient; $c$, $d$, and $e$ are dimensionless test parameters to be determined.

It is seen from figure 11 that the correlation coefficient $R^2$ of the fitting curve reaches 0.995 where $c = 0.739$, $d = 2.1831$ and $e = 0.441$. Therefore, the assumption for the relationship between $\sigma_1 - \sigma_3$ and $f_r$ is reasonable.
When other microscopic parameters are determined and kept unchanged, the peak value of the stress-strain curve increases with the increase of the contact modulus of sandy pebble soil. Concurrently, the peak value of the stress-strain curve has a logarithmic relationship with the contact modulus, and the relationship is denoted with
\[ E_{c}^{0.8284 \ln 1.0831}. \]

While other microscopic parameters are kept unchanged, the peak value of the stress-strain curve also increases with the increase of the friction coefficient of sandy pebble soil.

Meanwhile, the peak value of the stress-strain curve shows a logarithmic relationship with the friction coefficient, and its relationship is expressed as:
\[ (\sigma_1 - \sigma_3) = 0.739(f_r)^2 + 2.1831(f_r) + 0.441 \quad (16) \]

4. Conclusion

Mechanical properties of sandy pebble soil are obtained from the indoor triaxial compressive test, meanwhile, the sandy pebble soil specimen with the same particle size and the loading mode as that generally used in the indoor compressive test has also been set up using the discrete element method. The empirical method is combined with the response surface method to obtain the accurate microscopic parameters of sandy pebble soil, including contact modulus \( E_c \), friction coefficient \( f_r \) and bond stiffness ratio \( n \). PFC\textsuperscript{3D} is employed to quantitatively analyze the influence of microscopic parameters on the macroscopic properties of sandy pebble soil. In this paper, the empirical method and the response surface method are used to determine the mesoscopic parameters of sand and gravel soil for the first time, so that the numerical simulation of the obtained mesoscopic parameters is more in line with the actual situation of the soil layer. Through a large number of numerical simulation experiments, the relationship between the mesoscopic parameters and the macroscopic mechanical parameters is obtained, which can more intuitively show the influence of the mesoscopic parameters on the macroscopic mechanical parameters. From the indoor triaxial test and numerical simulation of the mechanical behavior of sandy pebble soil using the discrete element method, a series of useful conclusions are drawn as follows:

(1) At the initial stage of the triaxial compressive stress-strain curve, its slope tends to decrease with the increase of moisture content in sandy pebble soil specimen, that is, the elastic modulus of sandy pebble soil decreases with the increase of its moisture content. In addition, the compressive stress-strain curves of sandy cobble soil show obvious local fluctuation. This is mainly because in the triaxial shear test, the samples are mainly in point contact, and stress concentration phenomenon appears at the contact point. In some areas, the stress is greater than the shear strength of the pebble, resulting in the fragmentation of the pebble particles.

(2) When the axial strain reaches 3.5%, sandy cobble soil begins to fail, while the strain of other fine grained soil generally reaches 15%. This is mainly because the density of sandy cobble soil is higher than that of other fine grained soils, so there is no obvious phenomenon of strain softening. That is, sandy cobble soil can still bear a certain load after reaching the peak strength.

(3) When friction coefficient \( f_r \) and bond stiffness ratio \( n \) are kept unchanged, the peak value of the stress-strain curve increases with the increase of the contact modulus of sandy pebble soil. Meanwhile, the peak value of the stress-strain curve shows a clear logarithmic relationship with the contact modules.

(4) When contact modulus \( E_c \) and bond stiffness ratio \( n \) are kept constant, the peak value of the stress-strain curve also increases with the increase of the frictional coefficient of sandy pebble soil. Concurrently, the peak value of the stress-strain curve shows a logarithmic relationship with the friction coefficient.
(5) In combination with the relationship among $\sigma_1-\sigma_3$, $E_c$ and $f_c$ presented in this paper, the mesoscopic parameters of sand and pebble soil can be quickly obtained through the peak stress. With the help of discrete element software, the simulation compression process of sand and pebble soil is more refined and visualized, thus making up for the defect that the laboratory test cannot be analyzed qualitatively, and at the same time providing basic parameters for engineering simulation analysis of sand and pebble strata.

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Research and Application of Intelligent Operation and Maintenance Technology for Shuohuang Heavy-Load Railway Infrastructure.

Data availability statement

No new data were created or analysed in this study.

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