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Keywords

coarse-grained soil, skeleton void ratio, strength and deformation characteristics, scale-down method

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Abstract: The strength and deformation characteristics of coarse-grained soil are closely related to the initial gradation and dry density during specimen preparation. To study the strength and deformation characteristics of coarse-grain materials with different gradations and densities, a simple and generalized method for calculating the skeleton void ratio of coarse-grained soil is proposed. The results of laboratory tests show that there is an obvious monotonic variation between the skeleton void ratio obtained by this calculation method and the mechanical properties of coarse-grained soil. When the specimen size is consistent, with the decrease of the skeleton void ratio, the failure shear stress increases, the failure friction angle increases, and the cohesion decreases. Also, as the skeleton void ratio decreases, the average initial tangent elastic modulus increases and the average initial tangent Poisson's ratio decreases. On the whole, the smaller the skeleton void ratio, the greater the strength and stiffness of the coarse-grained soil. Based on this, a scale-down method of coarse-grained soil with equivalent skeleton void ratio is proposed in this paper.

Keywords: coarse-grained soil; skeleton void ratio; strength and deformation characteristics; scale-down method

1 Introduction

Coarse-grained soil is widely distributed and has excellent mechanical properties, so it is largely adopted in civil engineering, water conservancy engineering, traffic engineering, etc. As an important engineering building material, its mechanical behavior greatly affects the safety of engineering structures. Therefore, it is necessary to have a comprehensive understanding of the mechanical behavior of coarse-grained soil and its influencing factors.

Initial particle grading and dry density, as important factors affecting the strength and deformation characteristics of coarse-grained soil, their effects on the mechanical properties of coarse-grained soil should not be neglected. Currently, a large number of experimental studies have been carried out on the mechanical behavior of coarse-grained soils with different gradations and dry densities. Table 1 summarizes some typical relevant experimental studies in China in recent years.

As can be seen from Table 1, there is no consistent conclusion on the effects of initial gradation and dry density on the mechanical behavior of coarse-grained soil due to the different density and gradation indexes used. It should be noted that there seems to be a clear relationship between the skeleton void ratio e_{sk}, which incorporates the influencing of gradation and density, and the strength deformation characteristic. However, the existing calculation methods of e_{sk} are not very clear and complicated, making them difficult to be popularized and applied in engineering practice.

2 Calculation method of skeleton void ratio of coarse-grained soil

In the 1970s, while exploring the liquefaction resistance of the sand-silt mixture, some researchers found that not all fine particles filled in the pores between coarse particles bore external loads together with coarse particles, and some fine particles could move freely in the inter-coarse particle pores without acting as a skeleton [19–20]. Therefore, it is necessary to regard the space occupied by this part of fine particles that do not act the role of skeleton as pores, and then calculate a void ratio that can truly reflect the soil density, denoted as e_{sk}, which is called the skeleton void ratio.

In engineering practice, the coarse-grained soil is usually divided into coarse and fine particles with the grain size of 5 mm as the boundary. The skeleton bearing the external load is composed of all coarse particles and part of fine particles, but the coarse particles are the main bearers.

The overall void ratio is e. The percentage of the mass of particles smaller than 5 mm in the total particle mass is f_c, and f_c can also be expressed as $P_{<5}$. The percentage of the mass of the fine particles that serve as skeletons to the mass of all fine particles is B, and then the skeleton void ratio e_{sk}, can be written as

$$e_{sk}=\frac{e+(1-B)f_c}{1-(1-B)f_c} \tag{1}$$

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Table 1 Summary of previous experimental studies on different gradations and densities

Literature	Density index	Gradation index	Test type	Conclusions
Chen et al.[1]	Dry density ρ_d	Gravel content G_c	Medium-scale consolidated drained triaxial shear test	When ρ_d is the same, the peak value of deviational stress increases with the increase of G_c. When G_c is the same, the initial elastic modulus and bulk deformation modulus increase with the increase of ρ_d under each confining pressure.
Du et al.[2]	Dry density ρ_d	P_5 content	Large-scale confined compression test	When ρ_d is the same, the compression deformation first increases and then decreases with the increase of P_5 content, and when P_5 content is the same, the compression deformation decreases with the increase of ρ_d.
Cai et al.[3]	Dry density ρ_d	P_5 content	Large-scale direct shear test	When ρ_d is the same, the shear strength firstly increases and then decreases, with the increase of P_5 content, and the peak value of shear strength is at about 70% P_5 content. There is no obvious rule in the change of cohesion.
Ling et al.[4]	Dry density ρ_d	$P_{<5}$ content	Large-scale consolidated drained triaxial shear test	When ρ_d is the same, with the increase of fine particle content, the peak value at failure decreases under low confining pressure, while the peak value at failure increases under high confining pressure.
Fu et al.[5]	Porosity n	$P_{<5}$ content	Large-scale consolidated drained triaxial shear test	For the same gradation, the smaller n is, the larger the strength index c and φ are, and the larger the parameter k of Duncan-Zhang model is. The effect of gradation on the strength is not significant.
Xu et al.[6]	Porosity n	$P_{<5}$ content	Large-scale consolidated drained triaxial shear test	In the Duncan-Zhang and NHRI models, with the increase of porosity, K and n_d decrease while n, d_c and R_d increase. For the same porosity, the peak strength and the shear displacement corresponding to the shear to the peak strength point increase with the increase of $P_{<5}$ content.
Zuo et al.[7]	Degree of compaction K	P_5 content	Large-scale consolidated drained triaxial shear test	When K is the same, with the increase of P_5 content, the parameters of K and K_b of the Duncan-Zhang E-B model increase, i.e., the modulus increases.
Ma et al.[8]	Degree of compaction K	P_5 content	Large-scale confined compression test	When K is the same, with the increase of P_5 content, the compression modulus increases first and then decreases, and the peak value is located at about 75% P_5 content.
Wang et al.[9]	Degree of compaction K	P_5 content		When K is the same, with the increase of P_5 content, the compression modulus increases rapidly first and then increases slowly
Ma et al.[10]	Degree of compaction K	P_5 content	Unconsolidated undrained triaxial shear test	When K is the same, the internal friction angle φ increases first and then decreases with the increase of P_5 content, and the peak value is located at about 75% P_5 content. The cohesion decreases first and then increases, and the minimum value is located at about 55% P_5 content.
Wei et al.[11]	Maximum dry density ρ_{dmax}	Fractal dimension D, Gradation width	Large-scale confined compression test	Under the maximum dry density, the compression modulus increases first and then decreases with the increase of fractal dimension, and there is a maximum value near the fractal dimension of 2.2. The compression modulus increases with the increase of gradation width.
Li et al.[12]	Relative density D_r	$P_{<5}$ content	Large-scale consolidated drained triaxial shear test	The denser the sample, the larger the particle content less than 5 mm, the larger the peak value of deviatoric stress, the more obvious the phenomenon of strain softening and dilatancy.
Zhu et al.[13]	Relative density D_r	Fractal dimension D	Large-scale consolidated drained triaxial shear test	When the D_r is the same, with the increase of fractal dimension D, the shear strength increases first and then decreases, and the peak strength is the highest near the fractal dimension of 2.6.
Wu et al.[14]	Relative density D_r	Fractal dimension D	Medium-scale confined compression test	For a same D_r, the dry density under all levels of stress increases first and then decreases, and the peak value is located around the fractal dimension of 2.6 as the fractal dimension D increases
Li et al.[15]	Relative density D_r	Gravel content G_c	Large-scale consolidated drained triaxial shear test	In the case of same D_r, the internal friction angle φ and nonlinear strength index φ_0 gradually increase with increasing the gravel content, and the cohesion c first increases and then decreases with increasing the gravel content.
Wan et al.[16]	Relative density D_r	P_5 content		The shear strength increases with the decrease of P_5 content. With the increase of initial dry density, the internal friction angle increases, and the dilatancy becomes more obvious.
Wang et al.[17]	Skeleton void ratio e_{sk}		Medium-scale consolidated drained triaxial shear test	The strength and deformation characteristics are similar when the skeleton void ratio is the same.
Zhao et al.[18]	Skeleton void ratio e_{sk}			The drained shear strength decreases with the increase of e_{sk}, showing a good negative power function relationship.

The higher the fine particle content, the greater the proportion of fine grain content involved in undertaking the skeleton, that is, the larger the value of B. Value of 0 for B means that the fine particles are all filled in the pores between the coarse particles and do not act as a skeleton.

At present, there are various methods for calculating the value of B [17, 20–22], but they are not simple enough to be popularized in engineering practice.

In the particle analysis test, in accordance with '*Specification for coarse-grained soil test*' (T/CHES29－2019)[23], the apertures of standard fine-meshed sieves used include 0.075, 0.25, 0.5, 1, and 2 mm, and the apertures of standard coarse-meshed sieves are 5, 10, 20, 40, 60, 80, and 100 mm. For particles larger than 100 mm, the mechanical screen aperture diameter or sleeve ring diameter used includes 200, 400, 600, and 800 mm.

The characteristic particle size $d_0 = 5$ mm, and the maximum particle size $d_{\max}$ is d_n. The aperture diameter of the standard sieve between the two is denoted as d_i, and $d_i > d_{i-1}$, where the subscript i is an integer between 1 and $n-1$. According to the specified sieve aperture diameter, $1 \leqslant n \leqslant 11$. The percentage of the mass of particles with particle diameter in the range of $d_{i-1} - d_i$ to the total mass of the soil sample is denoted as $\Delta P_{d_{i-1} \sim d_i}$, and then the average particle size of coarse particles d_{cu}, can be easily calculated by Eq. (2).

$$d_{\mathrm{cu}}=\sum_{i=2}^{n}\left(\frac{d_{i-1}+d_i}{2}\right)\Delta P_{d_{i-1}\sim d_i} \tag{2}$$

The common gradation of the soil used in indoor large triaxial test $d_{\max}=60$ mm is as follows: $n=4$, $d_0=5$ mm, $d_1=10$ mm, $d_2=20$ mm, $d_3=40$ mm, and $d_4=60$ mm. The percentage of each particle group is ΔP_{5-10}, ΔP_{10-20}, ΔP_{20-40}, and ΔP_{40-60}, so $d_{\mathrm{cu}}=7.5\,\Delta P_{5-10}+15\ \ \Delta P_{10-20}+30\ \Delta P_{10-20}+50\ \Delta P_{40-60}$. Similarly, the average particle size of fine particles less than 5 mm d_{xi}, can be calculated.

In practical engineering, Eq.(3) can be further used to calculate the average particle size of coarse particles d_{cu}, and the average particle size of fine particles d_{xi}.

$$\left.\begin{aligned}d_{\mathrm{xi}}&=2.5P_{<5}\\ d_{\mathrm{cu}}&=\left(\frac{5+d_{\max}}{2}\right)P_{>5}\end{aligned}\right\} \tag{3}$$

Based on the values of d_{cu} and d_{xi}, an approximate calculation equation of B in Eq.(1) is proposed in this study, as shown below:

$$B=\sqrt[3]{\frac{d_{\mathrm{cu}}}{d_{\mathrm{xi}}}} \tag{4}$$

It should be noted that this proposed calculation method of the skeleton void ratio is not suitable for excessively sanded coarse-grained soil, such as coarse-grained soil containing more than 50% of particles less than 5 mm in size. In addition, the calculation method of B proposed in this study still belongs to the empirical equation.

3 Laboratory tests

3.1 Test materials and testing program

The coarse-grained soil used in the test is the natural sand and gravel material extracted from a borrow area for core rockfill dam located in a secondary terrace of in Tibet, China, as shown in Fig.1. The specific gravity of the coarse-grained soil is 2.73.

Fig. 1 Test coarse-grained soil

A large-scale triaxial consolidated drained shear test was carried out in this study. The test apparatus is YLSZ30-3 high-pressure triaxial apparatus from Changjiang River Scientific Research Institute. The size of the specimen used for triaxial shear tests is Φ300 mm×600 mm. During the test, the shear rate was controlled at 0.4mm /min, and the test was terminated when the axial strain reached 15%.

The gradation curves of the soil samples are shown in Fig. 2. The values of f_{c}, d_{cu}, d_{xi}, and ρ_{d} (g/cm^3) for each soil sample are listed in Table 2, where ρ_{d} is the dry density of the soil sample.

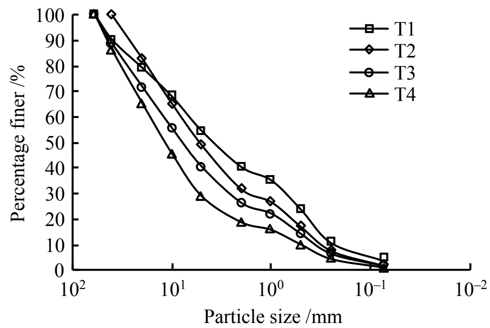


Fig. 2 Gradation curves

Table 2 Testing program

No.	ρ_{d}	e	f_{c}	d_{cu}	d_{xi}	B	e_{sk}
T1	2.085	0.309	0.545	10.88	0.716	0.404	0.940
T2	2.075	0.316	0.491	8.95	0.796	0.446	0.807
T3	2.094	0.304	0.403	14.33	0.651	0.357	0.760
T4	2.146	0.272	0.288	17.44	0.465	0.299	0.594

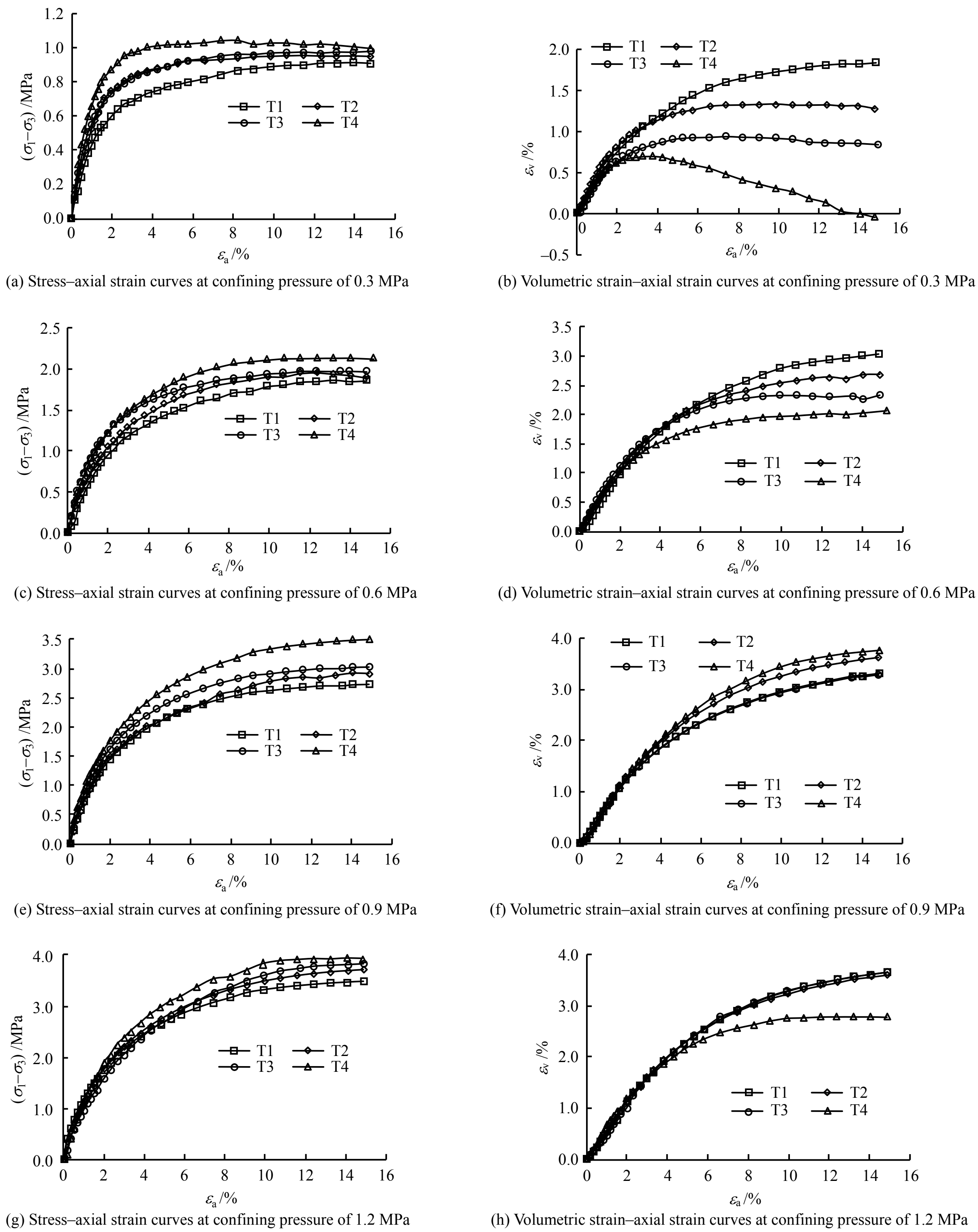
3.2 Test results

Figure 3 shows the variation of shear stress and volumetric strain ε_{v} with axial strain ε_{a} of samples with four different combinations of dry densities and gradations under different confining pressures. Note that the volumetric strain in Fig. 3 is the average value of the internal and external volume changes. When the volume decreases (compression), the volumetric strain is positive, while when the volume increases (swelling), the volumetric strain is negative.

As can be seen from Fig.3, the strength and deformation characteristics of coarse-grained soil with different e_{sk} values are relatively consistent under higher confining pressures, that is to say, the effects of gradation and density on the mechanical behavior of coarse-grained soil gradually weaken as the confining pressure increases.

The peak strength of coarse-grained soil gradually increases with the decrease of skeleton void ratio e_{sk}, but the soil samples have similar residual strength under lower confining pressures. As e_{sk} decreases, the volumetric deformation of coarse-grained soil basically presents a decreasing trend. It can be therefore considered that the smaller the skeleton void ratio, the greater the strength and stiffness of coarse-grained soil.

To further analyze the relationship between skeleton void ratio and strength deformation characteristics, it is necessary to conduct quantitative analysis.



(a) Stress–axial strain curves at confining pressure of 0.3 MPa

(b) Volumetric strain–axial strain curves at confining pressure of 0.3 MPa

(c) Stress–axial strain curves at confining pressure of 0.6 MPa

(d) Volumetric strain–axial strain curves at confining pressure of 0.6 MPa

(e) Stress–axial strain curves at confining pressure of 0.9 MPa

(f) Volumetric strain–axial strain curves at confining pressure of 0.9 MPa

(g) Stress–axial strain curves at confining pressure of 1.2 MPa

(h) Volumetric strain–axial strain curves at confining pressure of 1.2 MPa

Fig. 3 Large-scale triaxial test (CD) results

3.3 Skeleton void ratio and strength deformation characteristics

To further analyze the relationship between the skeleton void ratio and strength deformation characteristics, quantitative analysis was carried out on them respectively in this study. Under four different confining pressures, the shear stress at failure, that is, the peak shear stress within the range of 15% axial strain, and the strength parameters are shown in Table 3.

Table 3 Strength parameters

No.	$(\sigma_1-\sigma_3)_{max}$ at different confining pressure(MPa) /MPa				c /kPa	φ /(°)	φ_0 /(°)	$\Delta\varphi$ /(°)
	0.3	0.6	0.9	1.2				
T1	0.913	1.868	2.730	3.479	26.4	36.0	38.0	1.2
T2	0.955	1.962	2.917	3.701	20.1	37.3	38.4	0.6
T3	0.979	1.974	3.026	3.818	12.7	38.0	38.6	0.3
T4	1.047	2.142	3.498	3.926	23.5	39.1	40.0	0.3

In this study, the hyperbolic model was used to fit the stress–axial strain relationship and the volumetric strain–axial strain curve:

$$(\sigma_1-\sigma_3)=\frac{\varepsilon_a}{\dfrac{1}{E_i}+\dfrac{R_f\varepsilon_a}{(\sigma_1-\sigma_3)_{max}}} \quad (5)$$

$$\varepsilon_a=\frac{\varepsilon_a-\varepsilon_v}{2\mu_i+D(\varepsilon_a-\varepsilon_v)} \quad (6)$$

where E_i is the initial tangential deformation modulus of the soil; R_f is the ratio of the shear stress at failure to the lower limit stress of hyperbolic model, where the greater the value of R_f, the more obvious the softening phenomenon; μ_i is the initial tangential Poisson's ratio of the soil; and D reflects the magnitude of lateral expansion strain increment caused by small deviatoric stress increment.

Strese level is $S=(\sigma_1-\sigma_3)/(\sigma_1-\sigma_3)_{max}$. By taking the derivative of Eq.(5), the expression of tangent modulus E_t can be obtained as follows:

$$E_t=E_i(1-R_fS)^2 \quad (7)$$

By taking the derivative of Eq.(6), the expression of the tangent Poisson's ratio μ_t can be obtained as follows:

$$\mu_t=\frac{\mu_i}{\left\{1-\dfrac{DS(\sigma_1-\sigma_3)_{max}}{E_i(1-R_fS)}\right\}^2} \quad (8)$$

where $(\sigma_1-\sigma_3)_{max}$ satisfies the relationship with nonlinear strength parameters φ_0 and $\Delta\varphi$:

$$\sin\left(\varphi_0-\Delta\varphi\lg\left(\frac{\sigma_3}{p_a}\right)\right)=\frac{(\sigma_1-\sigma_3)_{max}}{(\sigma_1-\sigma_3)_{max}+2\sigma_3} \quad (9)$$

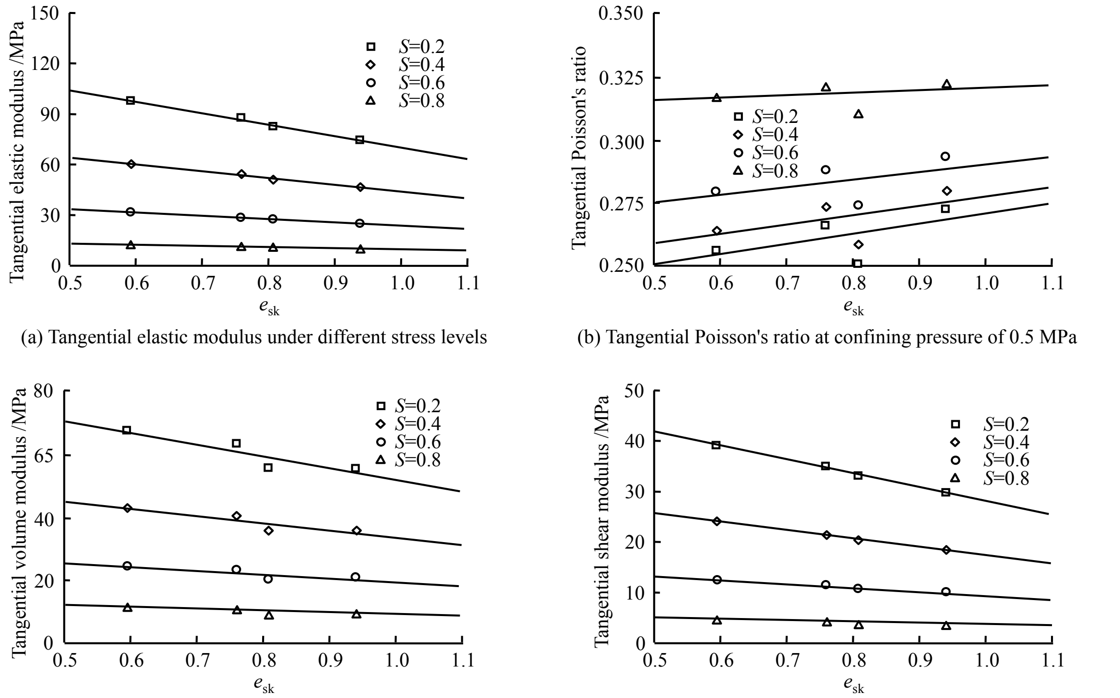
Under four distinct confining pressures, the mean values of the four deformation parameters corresponding to the test results in Fig. 3 are summarized in Table 4.

Table 4 Deformation parameters

No.	E_i /MPa	R_f	μ_i	D
T1	110.4	0.867	0.268	2.469
T2	122.3	0.882	0.246	3.116
T3	130.2	0.877	0.262	2.911
T4	145.3	0.883	0.251	3.362

One can see from Tables 3 and 4 that as the skeleton void ratio decreases, the strength parameters of φ_0 and φ increase, while the parameters of $\Delta\varphi$ and c decrease; the average initial tangent modulus increases, the failure ratio R_f basically remains unchanged, the average initial tangent Poisson's ratio decreases, and the parameter D increases.

Based on the above parameters, by using the hyperbolic model, tangential deformation modulus E_t, tangential Poisson's ratio μ_t, volumetric modulus $B=E/[(1-2\mu)\cdot 3]$, and shear modulus, $G=E/[(1+\mu)\cdot 2]$ are calculated when the confining pressure $\sigma_3=0.5$ MPa and stress level $S=0.2$, 0.4, 0.6, and 0.8. The relationship between the calculated results and the skeleton void ratio e_{sk}, is shown in Fig. 4.



(a) Tangential elastic modulus under different stress levels

(b) Tangential Poisson's ratio at confining pressure of 0.5 MPa

(c) Tangential volume modulus at confining pressure of 0.5 MPa

(d) Tangential shear modulus at confining pressure of 0.5 MPa

Fig. 4 Skeleton void ratio e_{sk} and deformation index

Figure 4 shows that as the skeleton void ratio increases, the tangent deformation modulus decreases while the Poisson's ratio, the volumetric modulus and shear modulus increases all show a decreasing trend.

In short, with the decrease of skeleton void ratio e_{sk}, the modulus indexes reflecting the stiffness of coarse-grained soil all increase.

4 Skeleton void ratio and scale-down effect of coarse-grained soil

Over the past few decades, a large number of experimental and theoretical studies on the scale effect of coarse-grained materials have been carried out, and a series of remarkable results have been obtained, but there are still considerable differences between these results.

Some researchers have divided the scale effect of coarse-grained soil into two types: One is the scale effect caused by the change of maximum particle size (i.e., the change of gradation), referred to as the particle size effect. The other is the boundary effect caused by the change of the dimension of the sample, referred to as sample size scale effect[24]. In addition, the difference between the test apparatus itself is also an important reason for the difference of test results before and after the scale-down of coarse-grained soil.

Xiao et al.[25] in Changjiang River Research Institute adapted two types of instruments to carry out a series of scale-down tests on the same type of coarse-grained soil samples with three sample sizes and three maximum particle sizes. The gradation curves of the tested soil samples are plotted in Fig. 5.

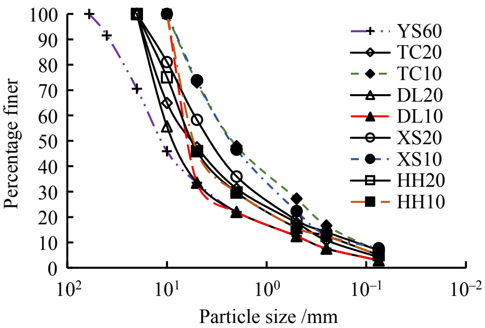


Fig. 5 Graduation curves in the literature [25]

The values of f_c, d_{cu}, and d_{xi}, corresponding to the gradation curves shown in Fig.5, are listed in Table 5. The corresponding skeleton void ratio can be obtained from the dry density of the sample ρ_d (g/cm^3) and the calculation method proposed above, also given in Table 5. Four confining pressures of 0.3, 0.6, 0.9 and 1.2 MPa were designed in the test, with a total of 100 soil samples.

Table 5 Testing program in the literature [25]

No.	ρ_d	e	f_c	d_{cu}	d_{xi}	B	e_{sk}
YS60	2.12	0.274	0.335	15.14	0.274	0.331	0.642
TC20	2.11	0.280	0.475	6.56	0.777	0.491	0.688
TC10	2.04	0.324	0.730	2.03	1.196	—	—
DL20	1.97	0.371	0.335	8.31	0.548	0.404	0.712
DL10	1.92	0.406	0.335	4.99	0.548	0.479	0.704
XS20	2.13	0.268	0.583	4.55	1.027	0.609	0.642
XS10	2.08	0.298	0.739	1.96	1.306	—	—
HH20	2.07	0.304	0.459	5.93	0.768	0.506	0.687
HH10	1.99	0.357	0.459	4.06	0.574	0.574	0.686

Due to the combination of gradation and density of the TC10 and XS10 groups, the percentage of fines contents is well over 50%, i.e., it has been over-sanded, resulting in the fact that it is not the coarse particles that serve as the main body of the skeleton, and therefore it cannot be called a coarse-grained soil. Accordingly, the aforementioned method of calculating the skeleton void ratio of coarse-grained soil is no longer applicable.

4.1 Scale effect of strength characteristics of coarse-grained soil

Large-scale saturated triaxial consolidation drained shear tests were carried out on the soil samples with a diameter of 300 mm in the YS60, TC20, DL20 and HH20 test groups by YLSZ30-3 high-pressure triaxial apparatus from Changjiang River Research Institute. The curves of peak shear stress within the range of 15% axial strain and skeleton void ratio, obtained from the triaxial shear tests, are shown in Fig. 6.

As can be observed from Fig.6, the shear stress at failure decreases with the increase of the skeleton void ratio. At low confining pressure, the decrease is obvious, while the decrease is slow at high confining pressure.

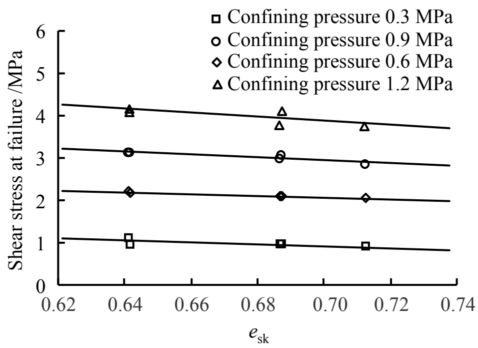


Fig. 6 Peak shear stress and skeleton void ratio in the literature [25]

For the XS20 test group, the saturated triaxial consolidation drained shear tests were carried out on the soil samples with three different sizes: (i) Diameter 300 mm, height 600 mm. (ii) Diameter 100 mm, height 200 mm. (iii) Diameter 150 mm, height 300 mm. The peak shear stress within the range of 15% axial strain is shown in Fig. 7.

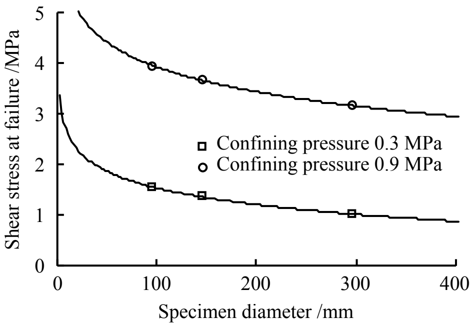


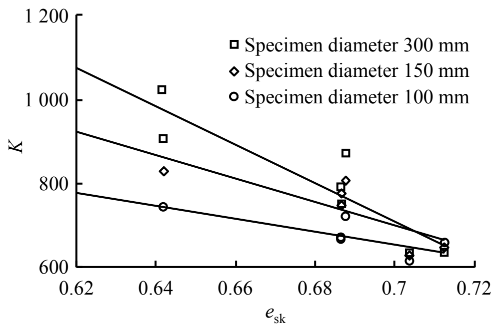
Fig. 7 Shear stress at failure of specimens with different sizes

Figure 7 shows that for coarse-grained soil samples with the same skeleton void ratio, the shear stress decreases with the increase of sample diameter, which is consistent with the test results presented by Zhu et al. [24]. This suggests that extra caution should be taken when using the strength index obtained by the soil sample of small size to analyze the stability of the dam slope, which may be dangerous.

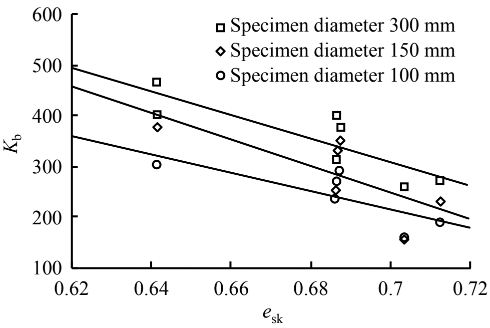
4.2 Scale effect of deformation characteristics of coarse-grained soil

The specific values of the deformation parameters of the Duncan-Zhang *E-B* model obtained by the tests are detailed in the literature [25]. In addition to the two combinations of gradation and density that shows over-sanded, the curves of the skeleton void ratio of the other test groups and the parameters of K and K_b of the Duncan-Zhang *E-B* model in the case of three different sample sizes are shown in Fig. 8.

As can be seen from Fig. 8, when the size of the samples is the same, the samples with different maximum particle sizes (different gradations) are obtained by using different scale-down methods. As the skeleton void ratio increases, the parameters of K and K_b of the Duncan-Zhang *E-B* model decrease. Considering that the other deformation parameters of n, m, and R_f of the Duncan-Zhang *E-B* model change inconspicuous, the conclusion that the tangential deformation modulus and tangential volume modulus decrease with the increase of the skeleton void ratio is still basically valid.



(a) Initial tangent deformation modulus K



(b) Initial tangential volume modulus K_b

Fig. 8 Skeleton void ratio and deformation index of in the literature [25]

4.3 Scale-down method of equivalent skeleton void ratio

Based on the principle of 'scale-down method = gradation scale-down method+density control standard', this study develops an equivalent density scale-down method based on the same soil skeleton void ratio, as shown in Fig. 9. That is, the skeleton void ratio is calculated by the soil gradation and dry density before field scaling down, and the dry density of the soil after scaling down is determined by the gradation and the skeleton void ratio of the soil after scaling down.

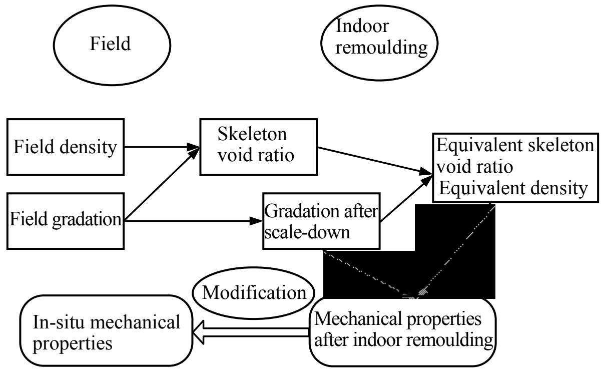


Fig. 9 Scaling method of equivalent skeleton void ratio

The premise of the scale-down method based on the same soil skeleton void ratio is that when the dimension of the samples is the same, the samples with different maximum grain size gradations are obtained by different scale-down methods. If the skeleton void ratio of the samples is equal, their mechanical behavior is basically the same.

The greatest advantage of this proposed scale-down method is that the dry density of soil in laboratory scaled tests can be determined without testing. Compared with the equivalent degree of compaction, the equivalent relative density and the equivalent density method based on pressuremeter modulus, this proposed method requires less testing work.

The disadvantage of the scale-down method based on the same soil skeleton void ratio is that it can only eliminate the scale effect of particle size gradation and cannot eliminate the scale effect of boundary dimensions. Therefore, it is necessary to modify the indoor substitute material test results based on the scale effect of boundary dimensions (as shown in Fig. 7) before this method can be applied to the mechanical properties of the field prototype material.

It should be emphasized that this paper only puts forward the preliminary idea of scale-down method based on the same soil skeleton void ratio, and the detailed demonstration of this proposed method needs to be further studied.

5 Conclusions

In this study, a new calculation method of skeleton void ratio e_{sk} is developed, which is very suitable for coarse-grained soil, simple to calculate and easy to be popularized in engineering field. The test results show that there is an obvious monotonic variation between the skeleton void ratio obtained by the proposed calculation method and the mechanical behavior of coarse-grained soil.

(1) The results of the large-scale triaxial consolidation drained shear tests show that with the decrease of skeleton void ratio, the peak strength of coarse-grained soil gradually increases and the volumetric strain gradually decreases. With the decrease of the skeleton void ratio, the strength parameters φ_0 and φ increase, while $\Delta\varphi$ and c decreases, the

average initial tangential elastic modulus increases, the failure ratio R_f basically remains unchanged, the average initial tangent Poisson's ratio decreases, and the parameter D slightly increases. With the increase of skeleton void ratio e_{sk}, the tangential elastic modulus, volumetric modulus and shear modulus decrease while the tangential Poisson's ratio shows an increasing trend

(2) The results of a series of triaxial consolidation drained shear tests indicate that when the dimension of the samples is the same, the soil samples with different maximum particle sizes (different gradations) are obtained by using different scale-down methods. As the skeleton void ratio e_{sk} decreases, the shear stress at failure increases. For coarse-grained soil samples with the same skeleton void ratio, the shear stress at failure decreases with the increase of sample diameter. As the skeleton void ratio increases, the parameters of K and K_b of the Duncan-Zhang E-B model decrease.

Overall, the smaller the skeleton void ratio, the greater the strength and stiffness of coarse-grained soil. This finding derived from the present study can be used to qualitatively predict the mechanical behaviors of coarse-grained soil with any combination of gradation and density and provide a reference for the optimal design of fillers in practical engineering.

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