Particle size effects on critical strength of granular soils through numerical and laboratory testing

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Efectos de tamaño de partícula en la resistencia al corte critica de suelos granulares mediante ensayos numéricos y de laboratorio.

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ABSTRACT: The critical shear strength characterization of waste rock (WR) is crucial for assessing the stability of waste rock piles. However, laboratory testing is challenging due to the broad grain size distribution (GSD) that ranges from silts to oversized rock clasts. The maximum grain size in a sample is restricted by the size of the equipment and the recommendations by international standards. This implies that WR materials require their GSD to be altered using grading scaling techniques to fit into standard testing cells. The common geotechnical practice for scaling is based on the aspect ratio $\alpha = D/d_{max}$, where D is the diameter of the sample and d_{max} the maximum grain diameter. However, different standards disagree on the minimum α to ensure a representative sample, ranging from α = 5 to 20. The main objective of this paper is to study the effects of α on the critical shear strength of granular materials through numerical triaxial tests in the frame of the discrete-element method (DEM) and laboratory triaxial tests on WR material. The samples used have α values varying from 5 to 20 and GSD of uniformity coefficients ranging from C_u = 1 to 2.2. The results show that the numerical and physical samples exhibit stable critical strength for $\alpha \geq 12$, regardless their initial grading. These results demonstrate the advantages of numerically testing materials that are challenging to fit into standard devices and suggest that international standards should be revisited to include the significant effects of α.

KEYWORDS: shear strength, DEM, mine waste rock, size effect.

1 INTRODUCTION.

Waste rocks (WR) are the valueless portion extracted to mine the ore, and their grain size distribution (GSD) includes a variety of particles, from silts up to clasts with more than 1 m in diameter (Bard et al., 2012; Ovalle et al., 2020). WR form waste rock piles which are recognized as one of the highest man-made geo-structures (Valenzuela et al., 2008). However, WR piles present high environmental risks and uncertain mechanical stability. To ensure the latter, extensive laboratory testing should be carried out to provide mechanical WR characterization (Linero et al., 2007, 2020; Ovalle et al., 2023). Due to the coarse rock clasts, the GSD must be altered according to the testing device available. Two small-scaling techniques are the most used: parallel grading (Marachi et al, 1972) and scalping (Zeller & Wulliman, 1957). In parallel grading, the shape of the GSD curve is preserved by scaling down both maximum (d_{max}) and minimum (d_{min}) particle sizes. In scalping, the material coarser than a size d_{max} is removed and a new GSD curve is generated by truncating the original one. Small-scaling methods are expected to provide representative results on the basis that critical shear strength does not depend on the GSD, as widely shown via experimental and numerical studies (Li et al., 2013; Yang and Luo, 2017; Azéma et al., 2017; Amirpour et al., 2019; Cantor et al., 2020). However, the scaling methods require that particle roughness and characteristic particle shape remain relatively constant in small-scaled samples (Ovalle and Dano, 2020; Carrasco et al., 2022, 2023).

Typically, triaxial or direct shear tests are used to measure the shear strength, applying d_{max} restrictions according to international testing standards (Cantor and Ovalle, 2023; Girumugisha et al., 2024). For triaxial tests, considering the sample aspect ratio $\alpha = D/d_{max}$, where D is the sample diameter, the values recommended are 5, 6 and 20, after British (BS 1377), American (ASTM D7181) and Japanese (JGS 0520) testing standards, respectively. Disagreement between the standards shows that the effects of α and GSD on the mechanical behavior are still poorly understood.

The main objective of this paper is to study the effects of α on the critical shear strength of granular materials. The recommended limits of α imposed by international standards are tested through triaxial numerical tests, in the frame of the discrete-element method (DEM), and triaxial laboratory tests on WR materials.

2 METHODOLOGIES

2.1 *Numerical DEM triaxial testing*

Drained triaxial 3D simulations on spheres are carried out using the simulation platform LMGC90, developed at the University of Montpellier, France. LMGC90 uses the non-smooth discrete-element approach known as contact dynamics (Dubois & Jean,

2006), which is capable of simulating interactions between rigid grains of varied shapes and sizes. The density of the solid material and the inter-particle friction coefficient are set to $\rho = 2000\ kg/m^3$ and $\mu_s = 0.4$, respectively.

A rigid wall box of width $W = 0.1\ m$ and ratio $H/W = 2$ is modeled, where H is the box's height. It is worth noting that this configuration is similar to a true triaxial test (Hambly, 1969), therefore different from the stress field generated by flexible membranes in standard triaxial testing. This configuration was simply chosen due to lower computational cost. A series of α are chosen to cover the values recommended by testing standards: $\alpha = [5, 6, 8, 10, 12, 15, 17.5, 20]$. The GSD is characterized by the parameter $R_d = d_{max}/d_{min}$, and the following values are modeled: $R_d = [1.0, 1.1, 1.8, 4]$, which correspond to $C_u = d_{60}/d_{10} = [1.0, 1.1, 1.4, 2.2]$, respectively. Thus, d_{max} is given by the α value chosen and d_{min} by a given R_d. The GSDs are shown in Fig. 1, where (a) presents different C_u values for a given α, and (b) shows eight parallel curves for a given $C_u = 1.4$, corresponding to the eight values of α proposed. 32 different samples are simulated in total. As an example, Fig. 2 presents screenshots of two samples, for combinations of small and large values for α and C_u.

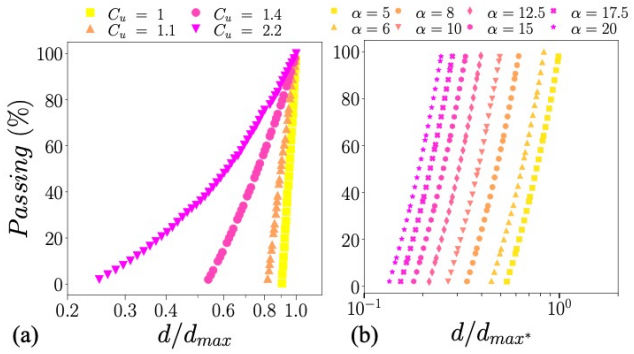

Figure 1. GSD of the numerical samples: (a) four C_u simulated for each α; (b) parallel GSDs with $C_u = 1.4$ covering a wide range of α.

The numerical triaxial tests are performed in two stages. First, the samples are compressed with a constant stress $\sigma_0 = 10\ kPa$, applied through rigid walls. To avoid stress gradients and boundary effects, the friction coefficient between the walls and the particles is null. This stage is carried out until the variation of the void ratio is less than 0.1%. The second stage is the triaxial shearing, which is undertaken by applying a constant velocity v_z on the top and bottom walls of the box, keeping constant the confining pressure through lateral rigid walls. The value of the velocity is calculated based on the inertial number $I = d_{max}(v_z/H_0)/\sqrt{\rho/\sigma_0} \ll 1$ (GDR MiDi, 2004), where H_0 is the height of the sample after isotropic compression. The inertial number is fixed for all the samples at $I = 10^{-4}$, to ensure a quasi-static shearing regime. Finally, the tests were finished when the deformation of the sample reached 60% of H_0.

2.2 Physical triaxial testing

A material called WR1 was collected from a hard rock mine. The material has a specific gravity of G_s= 2.75 and a uniaxial compression strength of UCS = 128 MPa. Mineralogy of WR1 is mainly composed of silicates (quartz and albite). Fig. 3a presents the GSD of the field material ($d_{max} = 75\ mm$), as well as scalped samples of WR1 down to d_{max} from 19 to 5 mm. The material classifies as a well-graded gravel with silt and sand (GW-GM). Additionally, a second set of samples (WR2, shown in Fig. 3b) is prepared from WR1, after removing all the fines (<0.08 mm) and particles coarser than 19 mm. The scalping technique is also applied to prepare small-scaled samples of WR2, as shown in Fig. 3b. Each sample is tested on triaxial specimens of diameter D = 150 mm, which allowed to accommodate different aspect ratios α, from 6 to 30.

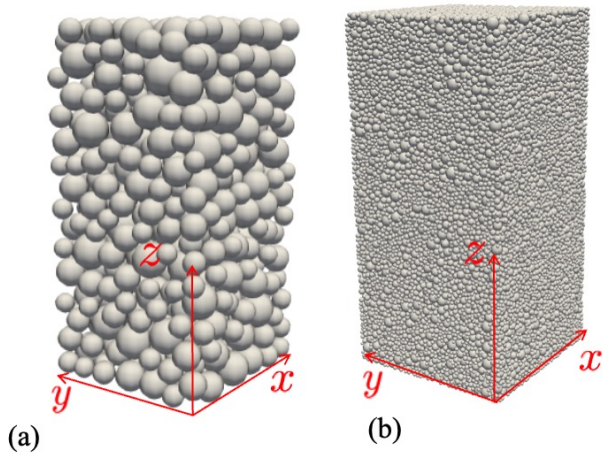

Figure 2. Examples of two samples tested: (a) $(\alpha, C_u) = [5, 1.0]$; (b) $(\alpha, C_u) = [15,\ 2.2]$.

Characteristic particle shape of the WR material is characterized through width to length sphericity S_{WL} = particle width/particle length, and roundness $R = \sum(r_i/N)/r_{in}$, where r_i is the radius of circles fitting the particle's concave corners, N the number of the fitted circles, and r_{in} the radius of the largest inscribed circle (Zheng and Hryciw, 2015). The results show that characteristic particle shapes are relatively constant over the whole range of particle sizes, with R = 0.3 to 0.4 and S_{WL} = 0.7, which classify as subangular particles with low sphericity.

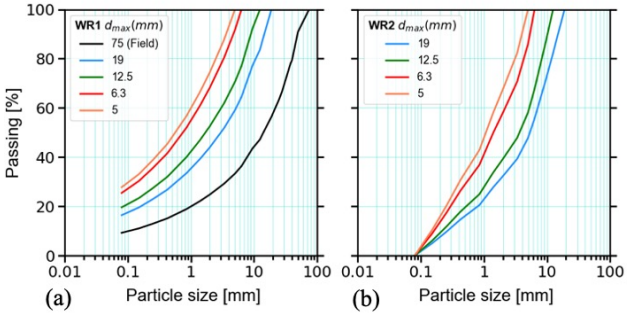

Figure 3. GSD of WR materials scalped for testing: (a) WR1 and (b) WR2.

Triaxial tests under consolidated drained conditions were performed on cylindrical dry specimens of D = 150 mm and H = 300 mm. A total of 18 tests are carried out on WR1 samples, and 15 on WR2. Loose samples are prepared without compaction by simply pouring the material into 10 distinct layers of homogeneous materials. Approximately 1.0 kg per layer is used. Dry densities obtained are different since the GSD varied between scalped samples. Dry densities of WR1 and WR2 varied between 17.83 – 18.95 kN/m^3, and 18.57 – 18.95 kN/m^3, respectively. An isotropic consolidation stage was performed by maintaining a desired confinement stress, varying between 45 to 210 kPa.

3 RESULTS

3.1 *Numerical results*

The discrete-element approach for simulations allows us to determine the inter-granular forces and stresses, which are useful to estimate the deviatoric stress $q = \sigma_1 - \sigma_3$ and the mean pressure $p = (\sigma_1 + 2\sigma_3)/3$ during triaxial shearing. This is based on the construction of a granular stress tensor computed through contact forces and particle positions (Ouadfadel and Rothenburg, 2001; Radjaï and Dubois, 2011).

In Fig. 4, the evolution of q/p and volumetric deformation (ε_v) are shown for all cases with $C_u = 2.2$ as a function of vertical deformation ε_1. A clear peak of q/p is reached around 10 % of deformation, and the specimen shows dilatant behavior followed by a critical state. Fig. 5 illustrates that, regardless of the GSD, the values of critical state friction angle ϕ_{cr} stabilize only if $\alpha \geq 12$. Moreover, better graded samples ($C_u = 1.4 - 2.2$) result in steady values of ϕ_{cr} at $\alpha \geq 8$. Laboratory triaxial tests also indicate that the critical shear strength is stable only if $\alpha \geq 12$.

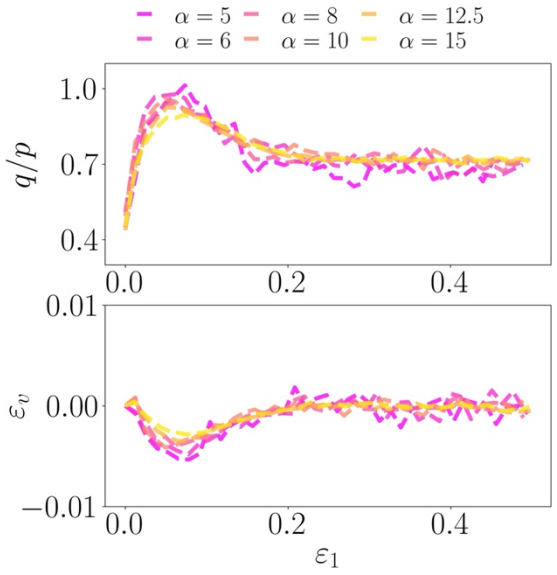

Figure 4. Normalized shear strength q/p and volumetric deformation ε_v as a function of the vertical deformation ε_1 for samples with $C_u = 2.2$

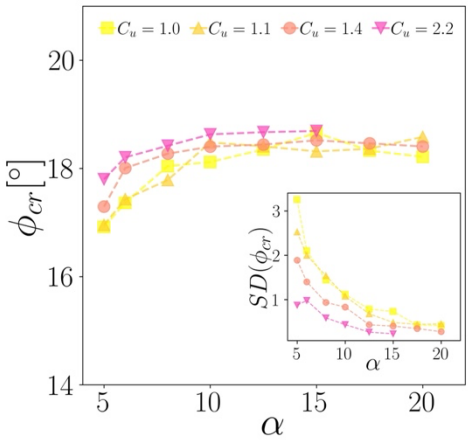

Figure 5. Critical friction angle ϕ_{cr} as a function of α

3.2 *Physical experimental results*

Fig. 6 shows the stress ratios q/p against ε_1 for all the triaxial tests. For each material, tests are grouped by confining pressure σ_3. As expected on loose samples, all tests exhibit hardening and reached their maximum strength around ε_1=10%, which remains almost constant up to ε_1=15 %. Loose samples of WR1 present slight dilation at low σ_3 of 45 and 80 kPa, which vanishes to fully contractive behavior for σ_3≥ 150 kPa.

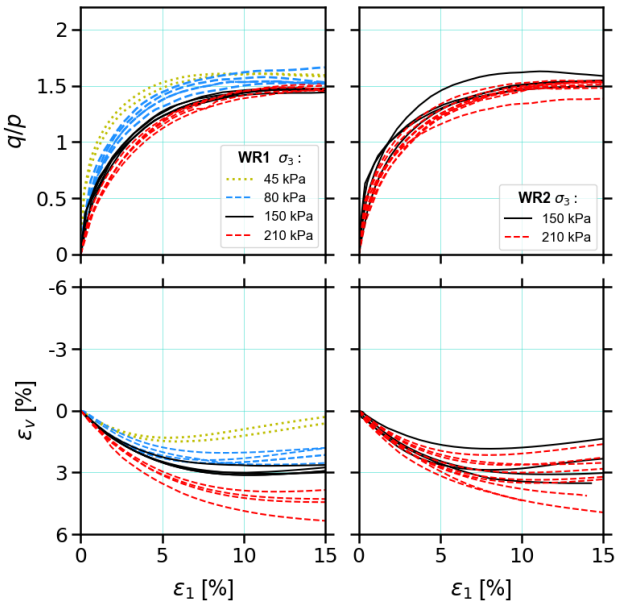

Figure 6. Normalized shear strength q/p results and volumetric deformation of the tested WRs, as a function of vertical deformation ε_1

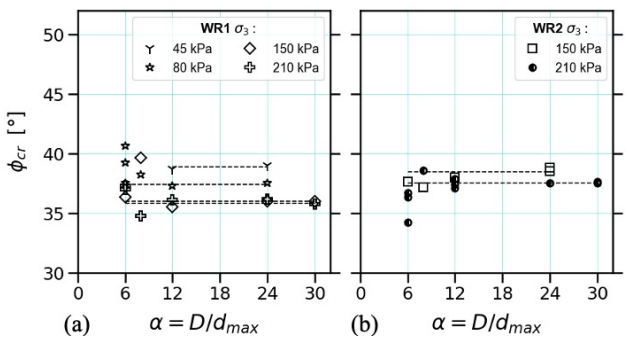

Figure 7. Critical friction angle comparison at different confinement pressure. (a) and (b) correspond with the two tested materials.

All q/p values reached at large vertical strain in Fig. 5 tend to similar values, which is consistent with several reported experimental results indicating that critical strength does not depend on PSD. However, the effect of α cannot be clearly identified in the figure. Therefore, Fig. 7 presents the critical friction angle (ϕ_{cr}) for all tests as a function of α, where dashed lines represent the mean ϕ_{cr} for each confining pressure σ_3. For comparison, ϕ_{cr} is assumed to be the mobilized shear strength at ε_1=15 % and calculated according to the Mohr-Coulomb failure criterion for non-cohesive materials. Samples tested at σ_3= 150 kPa exhibit mean ϕ_{cr} of 36° and 38° for WR1 and WR2, respectively. It can be observed that, for a given σ_3, ϕ_{cr} is stable for α between 12 and 30 but presents dispersion (of about 3°) at α < 12. To highlight the scatter of ϕ_{cr} on all samples, Fig. 8 illustrates the ratio $\phi_{cr}/\phi_{cr(\alpha=12)}$, where $\phi_{cr(\alpha=12)}$ is the ϕ_{cr} at α = 12, with both values obtained at the same σ_3. It can easily be noticed that the dispersion of ϕ_{cr} attains 10% at α < 12, and gradually vanishes to < 3% beyond this threshold value.

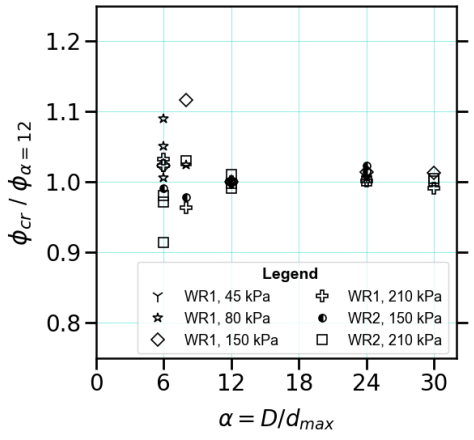

Figure 8. Normalized critical friction angle $\phi_{cr}/\phi_{cr(\alpha=12)}$ as a function of α.

4 CONCLUSIONS

A systematic study of numerical and physical experiments was presented to explore the effects of sample size on the critical shear strength of granular materials. Through triaxial test DEM simulations, it can be observed that the values of ϕ_{cr} stabilize when the sample scale $\alpha \geq 12$, regardless of the GSD. However, for better-graded samples, ϕ_{cr} reaches steady values as early as $\alpha = 8$. Laboratory triaxial tests also indicate that the critical shear strength is stable only if $\alpha \geq 12$, with significant dispersions for smaller α. These sample scales are bigger than what is indicated in widely used testing standards. Therefore, aspect ratio recommendations may not always ensure representative material characterization. This behavior may come from the formation of column-like structures mainly formed by the coarse fraction of the material (Cantor & Ovalle, 2023). Nonetheless, more extensive numerical and experimental testing is still needed to evaluate introducing new considerations in the testing standards.

It is important to note that while numerical work is not necessarily capable of replicating the complex soil behavior, it allows for an efficient study of specific mechanisms and effects, such as sample size assessments, under well-controlled conditions. Moreover, the parameters studied with DEM simulations in this work can be further analyzed to reveal the micro-mechanical sources of sample size effects, allowing for a better understanding of the mechanical behavior of granular soils.

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