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# Estimation of the shear strength of coarse-grained soils with fine particles

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## **Abstract**

The determination of the shear strength of coarse-grained soils in standard shear devices is a challenging task due to the presence of large grains. Three approaches could be used to reduce the grain size of the tested specimen. This study aimed to determine whether the shear strength of a coarse-grained soil with fines (particles smaller than 0.08 mm) can be estimated by the parallel gradation approach. Direct shear tests were conducted on parallel graded specimens of two soil types with different low plastic fines content. Two criteria can be used to select the dry density of the reconstituted specimen: the same dry density or the same relative density as the initial soil. The results showed that when the specimens had a low fines content, both criteria resulted in a similar dry density of the reconstituted soil, and the shear strength of the initial soil was correctly estimated. However, for the specimens with a high fines content, the friction angle of the initial soil was correctly estimated, and its cohesion was overestimated when the reconstituted soil was tested at the same dry density as the initial soil. A cohesion-fines content relationship was proposed to predict the initial soil cohesion.

**Keywords:** coarse-grained soils, shear strength, parallel gradation method, fines content.

## 1. Introduction

Coarse-grained soils are materials with grain sizes ranging from micrometers to several tens of centimeters. These materials can be employed in various kind of geotechnical projects: the use of rockfill to improve the stability of dams and dikes, embankment projects with nearby natural coarse-grained soils, and mining waste materials containing large grains. The determination of the mechanical characteristics of these materials is therefore essential. However, the presence of large grains requires the use of large devices to test a complete sample of these soils. For example, Ovalle et al. [19] used a triaxial cell with a diameter of 1 m ( $d_{\max} = 160$  mm). Estaire and Ollala [9] used a direct shear box with a volume of 1 m<sup>3</sup> ( $d_{\max} = 100$  mm). Unfortunately, such devices are difficult to use and unsuitable for soils with grain sizes larger than 160 mm. Hence, authors have investigated ways of estimating the shear strength of these soils from tests with common laboratory devices. The approach is to produce a representative sample of the initial soil by selecting only the smaller grains, compatible with the size of the experimental device to be used. The initial soil characteristics are then estimated from the results obtained. Three approaches have been proposed in the literature: scalping, substitution and parallel gradation [4,28] (Figure 1).

The first method (scalping) consists of removing all particles larger than the acceptable size of the available device from the initial soil and performing the test on the rest of the soil [33]. In the second method (substitution), the grains larger than the acceptable size of the device are removed from the initial soil and replaced with an equal weight of grains that are close to the maximum acceptable size. The third method (parallel gradation) consists of scalping the oversized particles and producing a reconstituted sample with a grain size distribution parallel to that of the initial soil on a log scale. The scalping method can be used when the remaining soil controls the overall behavior of the initial soil, i.e. for less than 30 to 40% of removed grains [5,10]. As for the substitution method, it has been used in few studies and the conclusions were divergent. Tabibnejad et al. [21] concluded that the substitution method provides a satisfactory estimation of shear strength while Zhang et al. [34] found that the method led to an underestimation of the initial soil shear strength. The parallel gradation is the most used technique. This method requires that grain characteristics (shape, strength) change slightly with size. Several authors have concluded that the reconstituted

material obtained by this technique has a slightly higher friction angle than that of the initial material depending on the confining stress [15,17,19,32]. The difference in strength is due to the increased grain breakage when the grains are larger. Marachi et al. [17] showed that parallel graded specimens have the same friction angle for the same amount of grain breakage. However, when the coarse-grained soil contains fines (particles smaller than 0.08 mm in size), the three aspects below should be considered for a better application of the parallel gradation technique.

First, the parallel gradation technique causes an increase of the finer particles fraction in the reconstituted specimen comparatively to the complete soil. Thus, the initial soil could have a low fines content that has negligible influence on its shear strength, while the reconstituted soil would have a fines content that significantly contributes to the shear strength. As a solution, Thiers and Donovan [24] proposed that similar fines contents should be attained for the initial and reconstituted soils. However, Verdugo and De La Hoz [29] showed that this method underestimates the shear strength of the initial soil. Therefore, the effect of the increased fines content in the reconstituted soil should be investigated.

Second, the criterion for choosing the density of the reconstituted soil is not unique. Authors tested reconstituted soils at the same dry density as that of the initial soil [14,15,17], while other researchers tested reconstituted soils at the same relative density [27,28], which could result in a significantly different dry density than that of the initial soil. It is then necessary to determine the criterion that provides the best estimation of the initial soil shear strength.

Finally, it is important to consider the influence of the equipment used for testing parallel graded specimens. In the literature, authors carried out all their tests with the same size equipment [15,27], while others used equipment of different sizes [17,19,32]. However, scale parameters (length and height of the specimen/ $d_{max}$ ) influence the soil shear strength [6,30,31]. The results could differ depending on whether a single device is used instead of several devices of different dimensions. Thus, the appropriate device size should be used when studying soils with the parallel gradation technique.

Therefore, for a better estimation of the initial soil shear strength by the parallel gradation method, it is necessary to understand the effect of the increased fines content on the shear strength and to select the appropriate test parameters (the density of the reconstituted soil and the  $L/d_{max}$  ratio). The aim of this study was to determine

whether the shear strength of a coarse-grained soil with fine particles (size smaller than 0.08 mm) can be estimated by the parallel gradation method. For this purpose, the parallel gradation method was applied to two different soils with different initial fines contents. Test parameters of the reconstituted soil (the dry density and the  $L/d_{max}$  ratio) were investigated to elucidate the effects of these parameters on the evaluated soil characteristics.

## **2. Materials and methods**

### **2.1. *Material properties***

Two coarse-grained soils were chosen for this study: a 0-80 mm limestone aggregate (LA) from a quarry and a natural gravel (NG) with a grain size up to 120 mm.

LA has a very low fines content. The particles of the sample are angular. Since the dimensions of the available equipment (see the next section) require a maximum grain size of 30 mm, the 0-30 mm specimen (LA30) with a parallel gradation to the initial material was chosen as the prototype soil for the shear tests. A 0-5 mm specimen (LA5) with a grain size distribution parallel to that of LA30 was reconstituted. Their minimum and maximum dry densities were determined according to the French standard for cohesionless soils AFNOR NF P94-059 [1]. The minimum dry densities were measured by filling a mold without disturbing the samples. The maximum dry densities were measured by shaking the samples in a mold at 50 Hz of frequency under a load surcharge of 10 kPa for 8 minutes. The tests were carried out three times for each sample and the average value was used.

NG is a soil with fines (low plastic silt) composed of subrounded to rounded particles. As with the LA, 0-30 (NG30), 0-15 (NG15) and 0-5 mm (NG5) specimens with grain size distribution curves parallel to that of the initial sample were selected for the tests. The maximum dry bulk density of all specimens was determined via a normal Proctor test (AFNOR NF P94-093 [2]) with respect to their fines contents.

The particle size distribution curves of the specimens are shown in Figure 2, and their physical characteristics are summarized in Table 1.

## **2.2. Direct shear devices**

Three direct shear boxes were used in this study: a large box (30 cm x 30 cm x 16.5 cm), a medium box (15 cm x 15 cm x 8.8 cm) and a small box (6 cm x 6 cm x 3.5 cm).

The large direct shear box consisted of two half boxes: the lower half box was fixed while the upper half box was driven by a motor. A system consisting of a frame and a hydraulic piston with a maximum capacity of 100 kN applied a constant normal load during the test. Displacement transducers (Linear Variable Differential Transformer or LVDT transducers) were installed to monitor the horizontal and vertical movements of the sample. These transducers had a range of 50 mm with an accuracy of 0.2%. The shear strength was measured by a load sensor with a capacity of 100 kN. The normal load, applied to the center of the plate, was measured by a load sensor placed between the frame and the upper plate. All the sensors were connected to a central acquisition unit that transmitted the data for recording and visualization via software.

The medium direct shear box is obtained by reducing the cross-section of the large direct shear box using adapters.

The small direct shear box is a classic direct shear box for soil mechanics. The normal load was applied by a dead weight system. The shear strength and vertical and horizontal displacements were measured by sensors.

## **2.3. Sample preparation and shear testing**

To obtain the reconstituted specimens with the desired grain size distribution curve, the 0-30 mm fraction of the samples was separated into different grain size fractions by sieving (Table 2). The different fractions were mixed according to the proportions given by the grain size distribution curve of the specimens. The samples were wetted to the desired water content ( $w_i$  in Table 3) and kept in airtight bags for at least one day for moisture content equilibration. Then, the samples were statically compacted in three layers in the large and medium shear boxes and in one layer in the small shear box until the desired density was reached. Static compaction was used, on the one hand to achieve high dry densities for the NG samples, and on the other hand to preserve the homogeneity of the LA samples. The saturation of the specimens was achieved by filling the shear apparatus tank with demineralized water and allowing the whole to stand for at least one day (the duration was set from previous saturation

tests). Under these conditions, the degree of saturation of the samples, calculated at the end of each test (as the ratio of the water content after testing and the saturation water content), was always higher than 98%. After saturation, the specimens were consolidated under the desired normal stress and then sheared in drained condition under the same normal stress at a constant shear rate (0.02%/min for the NG specimens and 0.10%/min for the LA specimens). The shear rate was selected to be lower than the maximum shear rate of NG30 or LA30 calculated from the consolidation curve.

#### **2.4. Experimental program**

The tests performed in this study aim to determine whether the parallel gradation method can be used to estimate the shear strength of coarse-grained soils with low plastic fines. The effects of the test parameters of the reconstituted soil (the dry density and the  $L/d_{max}$  ratio) on the evaluated initial soil characteristics were investigated.

The first test series was carried out to select the appropriate specimen size. The shear strength of NG5 under 100 kPa normal stress and at a dry density of  $2 \text{ Mg/m}^3$  was determined in the small and large shear devices.

The second series of tests aimed to evaluate the use of the parallel gradation method to estimate the shear strength of coarse-grained soils with a low fines content. The shear strength of LA30 was determined for a dry density of  $1.73 \text{ Mg/m}^3$  (which corresponds to a relative density of 28%). Due to the low fines content of LA5, the two criteria for selecting the dry density of the reconstituted soil based on the initial soil parameters resulted in similar dry densities:  $1.73 \text{ Mg/m}^3$  (same dry density) or  $1.68 \text{ Mg/m}^3$  (same relative density). LA5 was tested at a dry density of  $1.73 \text{ Mg/m}^3$ .

The purpose of the third series of tests was to evaluate the use of the parallel gradation method to estimate the shear strength of coarse-grained soils with a high fines content. The shear strength of NG30 was determined for a dry density of  $2 \text{ Mg/m}^3$  (corresponding to 94% of compaction degree). Due to the difference in fines content between NG30 and NG5, the two criteria for selecting the dry density of NG5 did not result in similar values. Thus, to determine the density selection criterion of the reconstituted specimen that provides the best estimation of the initial soil shear strength, the shear strength of NG5 was determined for dry densities of  $2 \text{ Mg/m}^3$  (same dry density as NG30) and  $1.83 \text{ Mg/m}^3$  (same compaction degree as NG30, i.e., 94%

of the NG5 maximal dry density). Then, the shear strength of NG15 was determined at a dry density corresponding to the selected density criterion.

The experimental conditions are summarized in Table 3.

### **2.5. Test repeatability**

Figure 3 shows the results of the repeatability tests on NG30 under a normal stress of 100 kPa in the large shear box. The shear stress increased significantly for a relative horizontal displacement smaller than 2%. Then, the shear stress gradually increased and stabilized after 6% relative displacement. The difference between the maximum shear strength values ( $\Delta\tau = 2$  kPa) shows a low variability of the results. The contraction values of both specimens were close, while their maximum dilation differed by 0.12% at the end of the test. Therefore, the shear strength and volume variation curves can be considered to have a high degree of repeatability.

## **3. Experimental results**

### **3.1. Effect of the specimen dimensions on the shear strength**

The shear curves of NG5 obtained in the large and small shear boxes under a normal stress of 100 kPa are shown in Figure 4. The shear strength in both cases increased to a maximum value and then decreased to a constant residual value. However, the difference between the peak and residual values was more pronounced in the small shear box ( $\Delta\tau = 26$  kPa) than in the large shear box ( $\Delta\tau = 4$  kPa). Similar observations were made by authors [5,6,12,30]. The peak resistance was reached at a lower relative displacement in the small shear box (1.3% of the relative displacement) than in the large shear box (3% of the relative displacement).

The NG5 samples exhibited contractive-dilatative behavior (Figure 4). The critical state was reached in the large shear box after 7% relative displacement, while the vertical strain continued to increase until the end of the test in the small shear box. There was greater dilation in the small shear box (2% versus 0.6%). The increase in the specimen dimensions allowed the critical state to be reached after 7% relative displacement.

The peak shear strength of NG5 obtained in the small shear box was 29 kPa higher than that obtained in the large shear box. The difference in shear strength is due to the difference in the development of the shear band in the two boxes [31]. When the  $L/d_{50}$  ratio (length of the shear box / mean grain size of the soil) is low, only one

continuous shear band develops in the specimen while several shear bands develop when the  $L/d_{50}$  ratio is high. Thus, the ratio between the specimen dimensions and the average or maximum grain size influences the peak shear strength. To be comparable, parallel graded specimens should have similar  $L/d_{max}$  ratios. Thus, laboratory studies should focus on conducting tests in different apparatuses with similar specimen ratios rather than using the same device for all particle sizes. Therefore, in this study, the 0-30 mm specimens were tested in the large shear box ( $L/d_{max} = 10$ ), the 0-15 mm specimens were tested in the medium shear box ( $L/d_{max} = 10$ ) and the 0-5 mm specimens were tested in the small shear box ( $L/d_{max} = 12.5$ ).

### ***3.2. Estimation of the shear strength of the coarse-grained soils with a low fines content by the parallel gradation method: the LA case***

The shear curves of parallel graded specimens LA5 (4% fines) and LA30 (2.5% fines) tested at  $1.73 \text{ Mg/m}^3$  are shown in Figure 5. Under a normal stress of 50 kPa, the shear curves were coincident. However, under stresses of 100 and 200 kPa, the shear stress of LA5 increased significantly for a relative displacement smaller than 2% and then increased gradually until the shear stress reached a constant value at a relative displacement of 10 to 12%. The shear stress of LA30 increased and reached a constant value after 8% relative displacement under 100 and 200 kPa. The maximum shear stresses for both specimens were the same under all normal stresses.

The volume change curves of both specimens showed a contractive-dilative behavior for normal stresses of 50 and 100 kPa (Figure 5). However, LA5 had contraction values that were 0.2% higher than those of LA30 at 50 kPa of normal stress, while both curves were similar at 100 kPa of normal stress. In contrast, for 200 kPa of normal stress, LA30 exhibited contractive behavior up to 4% relative displacement, followed by a dilation behavior, whereas the behavior of LA5 was essentially contractive. This difference in behavior can be explained by the more progressive mobilization of LA5 shear stress under 200 kPa of normal stress.

The Mohr-Coulomb criterion, commonly used to analyze soil failure envelopes, was adopted in this study. The failure criteria of the LA specimens are shown in Figure 6. The constant shear stresses reached at the end of each test were used to plot the failure criteria. Both specimens were assumed to have zero cohesion because the specimens are essentially composed of granular particles. Their friction angles can be considered identical (Table 4). The LA30 shear strength was correctly estimated when

both specimens (LA30 and LA5) had a fines content lower than 4% under normal stresses between 50 and 200 kPa.

### **3.3. Estimation of the shear strength of the coarse-grained soils with a high fines content by the parallel gradation method: the NG case**

In this section, the shear strengths of NG30 and NG5 determined at the same compaction degree and at the same dry density are presented. Then, the shear strength of NG15 determined at a dry density corresponding to the selected density criterion is presented.

#### **3.3.1. Initial and reconstituted soils tested at the same compaction degree**

Figure 7 shows the stress-strain and volume variation curves of NG30 (15% fines) and NG5 (22.5% fines) tested at 94% of the maximum dry density of each specimen. The shear curves had a similar trend for all applied normal stresses: a significant increase for a relative horizontal displacement smaller than 2%, followed by the shear stress reaching a constant value. This value was reached after 2% relative displacement under 50 and 100 kPa for NG5. Under the same normal stresses, constant shear stresses were reached after 3% relative displacement for NG30. However, under 200 kPa, the shear stress stabilized at 10% relative displacement for both specimens. The maximum shear stress of NG30 was approximately 20% higher than that of NG5 under all applied normal stresses.

When the normal stress increased, the contraction phase ended at a higher relative displacement, and the maximum contraction value increased for both specimens. The increase in normal stress resulted in a decrease in the maximum dilation value. NG5 had a contraction that was 0.06 to 0.10% higher than that of NG30 during shearing. However, NG30 had higher dilation values at the end of the tests. The difference between the maximum dilation values of the two specimens decreased from 0.90% to 0.10% when the normal stress increased from 50 to 200 kPa.

The failure criteria for the NG specimens, plot with the constant shear stresses reached at the end of each test, are shown in Figure 9. The cohesion and friction angle values of NG5 tested at  $1.83 \text{ Mg/m}^3$  were lower than those of NG30 tested at  $2 \text{ Mg/m}^3$ . Indeed, the cohesion of NG30 was 8 kPa higher and the friction angle was  $4^\circ$  larger than the values of NG5 (Table 5).

### 3.3.2. *Initial and reconstituted soils tested at the same dry density*

The shear curves of NG5 exhibited peak and residual values when the specimen had a dry density of  $2 \text{ Mg/m}^3$  (Figure 8). The peak shear strength was obtained for a relative displacement between 1 and 2% depending on the applied normal stress. The residual shear strength was obtained for a relative displacement between 6 and 8%. In contrast to NG5, the shear strength curves of NG30 did not exhibit peak values. Additionally, the maximum shear stress values of NG30 were between the peak and residual values of NG5. As the normal stress increased, the residual shear stress of NG30 was closer to the peak shear stress of NG5. The difference between the two values, which was 24% under 50 kPa, was only 4% under 200 kPa.

The volume variation curves are shown in Figure 8. The dilation of NG5 was more pronounced than that of NG30 under all applied normal stresses. When the normal stress increased, the difference in the maximum dilation of the two specimens increased from 0.25% under 50 kPa to 1.18% under 200 kPa. The increase in dry density of the NG5 specimen led to an increase of 1 to 1.3% in the dilation values. Additionally, this increase in density led to the end of the contraction phase of the specimen at a smaller relative displacement under a normal stress of 200 kPa. The failure criteria (Figure 9) showed that NG5 tested at  $2 \text{ Mg/m}^3$  had a peak friction angle that was  $1.6^\circ$  smaller than that of NG30 and a cohesion that was 17 kPa higher than that of NG30 (Table 5).

### 3.3.3. *Density selection criterion of the reconstituted soil*

When NG5 was tested at the same compaction degree as NG30 (94%), the shear curves of both specimens had the same trend (Figure 7). Nevertheless, the difference between the dry densities led to an underestimation of the maximum shear strength, the cohesion, and the friction angle of NG30. However, when NG5 was tested at the same dry density as NG30 ( $2 \text{ Mg/m}^3$ ), its higher compaction degree (103%) induced a peak shear strength development (Figure 8). The peak shear strength of NG5, compared to the residual strength of NG5 obtained at a compaction degree of 94%, reflected a gain in strength ( $\Delta\tau = 30 \text{ kPa}$ ) due to the increase in NG5 dry density. The NG5 peak shear strengths were higher than the residual shear strengths of NG30 under normal stresses of 50, 100 and 200 kPa. The maximum shear strengths of NG5 obtained with both criteria (same compaction degree and same dry density) could not predict the NG30 maximum shear strength. However, when NG5 was tested at the

same dry density as NG30, both specimens had similar friction angles, and the cohesion of NG30 was overestimated. For the same compaction degree, both the cohesion and friction angle of NG30 were underestimated (Figure 9). Therefore, when the fines content increases in the reconstituted soil, it should be tested at the same dry density as the initial soil, rather than at the same compaction degree. To support that observation, the shear strength of the NG15 specimen was also determined at a dry density of  $2 \text{ Mg/m}^3$  (same dry density as NG30).

The results of the shear tests of NG15 at a dry density of  $2 \text{ Mg/m}^3$  are shown in Figure 10. The shear curves had the same trend as that of NG30. Under the normal stresses of 50 and 100 kPa, the shear stress increased significantly for a relative displacement smaller than 4% and reached a constant value between 4 and 6% of relative displacement. Then, the shear stress decreased slightly and was reduced by 7 kPa at the end of the test (10% relative displacement). Under 200 kPa of normal stress, the shear stress increased and reached a constant value at 7% relative displacement. The maximum shear stresses of NG15 are 11% to 20% higher than those of NG30 under all applied normal stresses.

The failure criterion of NG15 is shown in Figure 11. The friction angle of NG15 was  $1.2^\circ$  higher than that of NG30 and its cohesion was 15 kPa higher than that of NG30.

Thus, the three NG specimens tested at the same dry density ( $2 \text{ Mg/m}^3$ ) had similar friction angles while the cohesion increased with the fines content in the specimens.

#### **4. Discussion on the estimation of the shear strength of coarse-grained soils with fines by the parallel gradation method**

The parallel gradation method was applied to two coarse-grained soils: a limestone aggregate (LA) containing very few fines (2.5%) and a natural gravel (NG) with a high fines content (15%). The objective was to highlight the impact of the fines content (particles smaller than 0.08 mm in size) on the estimation of the shear strength of the coarse-grained soils using the parallel gradation method. In the following, the results are analyzed to extend them to other coarse-grained soils.

The parallel gradation method results in a higher fines content in the reconstituted soil than in the initial coarse-grained soil (Figure 2). The results of this study on LA specimens showed that parallel graded specimens with less than 4% fines have similar friction angles (Figure 6). Previous studies on materials with a low fines content also

showed a good estimation of the friction angle by the parallel gradation technique under low applied confining stresses [17,19,32]. For NG specimens, the increase in fines content due to the use of the parallel gradation method could influence the soil fabric (the interparticle contacts network in the soil). In fact, up to a certain percentage of fines, the soil fabric is dominated by coarse particles that are in contact with each other in the specimen. Beyond this percentage, the fines dominate the soil fabric, the coarse particles are no longer in contact [7,23]. The results on NG specimens indicated that the friction angle was less impacted by the increase in low plastic fines content in the reconstituted specimen (Figure 11). This could be due to a similarity in the interparticle contacts of the studied specimens. In fact, the application of the parallel gradation method requires that grain properties such as shape and strength should be similar with grain size [3,16,11]. When these conditions are met, interparticle contacts could be considered similar in initial and reconstituted soils [11]. As the friction angle depends on the interparticle contacts, it can be assumed that the friction angle of the reconstituted soil will be similar to that of the initial soil if the soil fabric is still dominated by the coarser particles. However, the higher fines content in the reconstituted soil compared to the initial soil led to an increase in cohesion, which is in line with the results of Hsiao et al. [13]. Thus, the parallel gradation method gives a good estimation of the friction angle if the reconstituted soil has coarser particles in contact as the initial soil and the fines have a low plasticity. However, the cohesion increased with the increase in fines content. A change in the dominant grains network occurs when the void ratio of the coarse particles in the whole soil is higher than the maximum void ratio of the coarse particles alone [22]. The percentage of fines that correspond to the change in dominant grains network depends on factors such as the nature of the soil, relative density, and normal stress [18]. Previous studies on sand-silt or sand-clay mixtures showed that the change in dominant grains network occurs for a fines content higher than 20 to 30% [18,20,25,26]. Thus, it can be assumed that for less than 20 to 30% fines in coarse-grained soils, the soil fabric is dominated by coarse particles. Therefore, the parallel gradation method makes it possible to estimate the friction angle of coarse-grained soils with low plastic fines when the fines content in the reconstituted soil is lower than 20 to 30%. However, the difference in the fines content between the initial and reconstituted soils could lead to an overestimation of the cohesion. The results of Elkady et al. [8] on a sand-clay mixture showed that the cohesion-clay content curve can be divided into three parts (Figure 12):

- The cohesion was close to zero for a clay content smaller than 10%;
- Between 10 and 40% clay, the cohesion increased significantly with clay content;
- The cohesion reached a constant value after 40% clay content.

The same trend was also observed in the results of Hsiao et al. [13] on a sand-silt mixture with different silt content thresholds for each part of the curve (Figure 12). Thus, the variation in cohesion with the fines content could be represented by a sigmoid function with cohesion values ranging between 0 and  $c_{100}$  (cohesion for 100% fines) when the fines content ( $f$ ) increases from 0 to 100%:

$$(1) \quad \frac{c}{c_{100}} = \frac{1 - e^{-\lambda f}}{1 + e^{-\lambda f}}$$

$\lambda$  is a model parameter;  $c$  and  $c_{100}$  are expressed in kPa.

This correlation was used to fit the data from Elkady et al., Hsiao et al. [8,13] and this study (Figure 12). The theoretical curves were in good agreement with the experimental data. The parameters  $c_{100}$  and  $\lambda$  of each mixture, assigned so that the theoretical curve was as close as possible to the experimental points, are given in Table 6. Then, the relationship between the cohesion and the fines content (Eq. 1) can be used to estimate the cohesion of the initial coarse-grained soil.

In this study, the very initial LA sample (LA80) had a fines content (1.9%) close to that of the reconstituted specimens LA30 and LA5 (2.5 and 4% respectively). If tested under the same conditions, the LA80 specimen would have similar failure parameters. Thus, a zero cohesion and a friction angle of  $48^\circ$  are assumed for LA80. Furthermore, the very initial NG sample (NG120) had a lower fines content (4.3%) than the reconstituted specimens NG30, NG15 and NG5 (15, 19.8 and 22.5% respectively). If tested under the same conditions (a dry density of  $2 \text{ Mg/m}^3$ ), the cohesion of NG120 would be lower and its friction angle would be similar to those of the reconstituted specimens. The relationship between the cohesion and the fines content for NG specimens was used to estimate NG120 cohesion. Thus, a cohesion of 1.8 kPa and a friction angle of  $41^\circ$  can be predicted for NG120.

The observations of this study lead to a simple methodology for estimating the shear strength of coarse-grained soils with grain characteristics (shape, strength) vary slightly with size and having less than 30% low plastic fines under normal stresses lower than 200 kPa:

- Produce a reconstituted soil sample with a grain size distribution curve that is parallel to that of the initial soil and have a fines content lower than 30%;
- Perform shear tests on the reconstituted soil at the same dry density as the initial soil.

The friction angle of the reconstituted soil will be similar to that of the initial soil. If the initial soil has cohesion, it will be overestimated due to the difference in the percentage of fines. To better estimate the initial soil cohesion, the cohesion of at least two other reconstituted specimens with different fines contents (lower than 30%) can be determined. Then, the cohesion-fines content relationship proposed in this study can be used to predict the cohesion of the initial soil.

## 5. Conclusions

The aim of this study was to establish an estimation method for the shear strength of coarse-grained soils with fines by performing tests on their finer fraction. Two types of soils with low plastic fines were studied: a natural gravel (NG) with 15% fines and a limestone aggregate (LA) with 2.5% fines. Reconstituted specimens were produced using the parallel gradation approach, and the shear strengths of the initial and reconstituted specimens were determined. The following observations can be drawn:

- The peak shear strength is influenced by the ratio between the specimen dimensions and the maximum grain size. To study different parallel graded specimens in the laboratory, devices with similar  $L/d_{max}$  ratios should be used rather than the same device to obtain comparable shear strengths.
- When the reconstituted soil specimen is tested at the same compaction degree as that of the initial soil, the dry density could be lower due to the higher fines content. In that case, the shear strength curve would have the same trend as that of the initial soil. However, the lower dry density of the reconstituted soil would lead to an underestimation of the initial soil shear strength.
- In contrast, when the reconstituted soil is tested at the same dry density as that of the initial soil, the trends of both shear curves could be different due to their different compaction degrees. However, the friction angle of the initial soil is better estimated.
- The friction angle of coarse-grained soils with low plastic fines can be correctly estimated by the parallel gradation method when the reconstituted soil has a

similar fabric (formed by the interparticle contacts network in the specimen) as the initial soil, i.e., for a fines content lower than 30%. However, the difference in fines content between the initial and reconstituted soils could lead to an overestimation of the cohesion. Further tests on other reconstituted specimens allow a correlation to be established between the cohesion and the fines content. This relationship can be used to estimate the cohesion of the initial sample.

The results of this study made it possible to clearly specify the conditions of use of the parallel gradation approach to estimate the shear strength of coarse-grained soils with fines content lower than 30% under saturated conditions. Additional tests are required to extrapolate the findings under unsaturated conditions.

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Table 1: Physical characteristics of soils.

| Samples | Fines content<br>(%) | $\rho_{dmin}$<br>(Mg/m <sup>3</sup> ) | $\rho_{dmax}$<br>(Mg/m <sup>3</sup> ) | $W_{opt}$<br>(%) | $d_{50}$<br>(mm) |
|---------|----------------------|---------------------------------------|---------------------------------------|------------------|------------------|
| NG30    | 15.0                 | -                                     | 2.13                                  | 8.3              | 6.6              |
| NG15    | 19.8                 | -                                     | 2.06                                  | 9.7              | 3.3              |
| NG5     | 22.5                 | -                                     | 1.95                                  | 10.4             | 1.1              |
| LA30    | 2.5                  | 1.65                                  | 1.92                                  | -                | 9                |
| LA5     | 4.0                  | 1.61                                  | 1.88                                  | -                | 1.5              |

*Table 2: Separation of initial samples into grain size fractions.*

| Samples | Grain size fractions (mm)  |
|---------|--|
| NG      | < 0.08; 0.08-0.4; 0.4-0.63; 0.63-1.25; 1.25-2.5; 2.5-5; 5-10; 10-15; 15-30 |
| LA      | < 0.4; 0.4-1; 1-2; 2-5; 5-10; 10-15; 15-25; 25-30                          |

Table 3: Tests parameters.

| Series | Specimens | L x l x H (cm <sup>3</sup> ) | $\rho_d$<br>(Mg/m <sup>3</sup> ) | $\rho_d/\rho_{dmax}$<br>(%) | Dr*<br>(%) | W <sub>i</sub><br>(%) | $\sigma_n$ (kPa) |
|--------|-----------|------------------------------|----------------------------------|-----------------------------|------------|-----------------------|------------------|
| 1      | NG5       | 30 x 30 x 16.5               | 2.00                             | 103                         | -          | 10.4                  | 100              |
|        | NG5       | 6 x 6 x 3.5                  | 2.00                             | 103                         | -          | 10.4                  | 100              |
| 2      | LA30      | 30 x 30 x 16.5               | 1.73                             | -                           | 28         | 2.0                   | 50, 100, 200     |
|        | LA5       | 6 x 6 x 3.5                  | 1.73                             | -                           | 48         | 4.0                   | 50, 100, 200     |
| 3      | NG30      | 30 x 30 x 16.5               | 2.00                             | 94                          | -          | 8.3                   | 50, 100, 200     |
|        | NG5       | 6 x 6 x 3.5                  | 1.83                             | 94                          | -          | 10.4                  | 50, 100, 200     |
|        | NG5       | 6 x 6 x 3.5                  | 2.00                             | 103                         | -          | 10.4                  | 50, 100, 200     |
|        | NG15      | 15 x 15 x 8.8                | 2.00                             | 97                          | -          | 9.7                   | 50, 100, 200     |

\*Dr =  $(e_{max} - e)/(e_{max} - e_{min})$  : relative density

Table 4: Shear strength of limestone aggregate specimens.

| Specimens | $\rho_d$ (Mg/m <sup>3</sup> ) | $\sigma_n$ (kPa) | $\tau_{max}$ (kPa) | $c'$ (kPa) | $\phi'$ (°) |
|-----------|-------------------------------|------------------|--------------------|------------|-------------|
| LA5       | 1.73                          | 50               | 61.7               | 0.0        | 48.4        |
|           |                               | 100              | 112.7              |            |             |
|           |                               | 200              | 223.8              |            |             |
| LA30      | 1.73                          | 50               | 65.5               | 0.0        | 48.9        |
|           |                               | 100              | 112.3              |            |             |
|           |                               | 200              | 228.6              |            |             |

Table 5: Shear strength of natural gravel specimens.

| Specimens | $\rho_d$ (Mg/m <sup>3</sup> ) | $\sigma_n$ (kPa) | $\tau_{max}$ (kPa) | $c'$ (kPa) | $\phi'$ (°) |
|-----------|-------------------------------|------------------|--------------------|------------|-------------|
| NG5       | 1.83                          | 50               | 46.8               | 9.3        | 37.0        |
|           |                               | 100              | 85.0               |            |             |
|           |                               | 200              | 160.0              |            |             |
| NG5       | 2.00                          | 50               | 75.6               | 35.1       | 39.6        |
|           |                               | 100              | 119.5              |            |             |
|           |                               | 200              | 200.3              |            |             |
| NG15      | 2.00                          | 50               | 79.2               | 32.8       | 42.4        |
|           |                               | 100              | 125.0              |            |             |
|           |                               | 200              | 215.3              |            |             |
| NG30      | 2.00                          | 50               | 61.1               | 17.5       | 41.2        |
|           |                               | 100              | 105.7              |            |             |
|           |                               | 200              | 192.8              |            |             |

Table 6: Parameters of the relationship between the cohesion and the fines content

| Samples          | $c_{100}$ (kPa) | $\lambda$ | References           |
|------------------|-----------------|-----------|----------------------|
| Sand-clay (OMC)  | 22              | 0.12      | Elkady et al. (2015) |
| Sand-clay (WOMC) | 15              | 0.14      | Elkady et al. (2015) |
| Sand-silt        | 18              | 0.10      | Hsiao et al. (2015)  |
| NG               | 40              | 0.25      | This study           |

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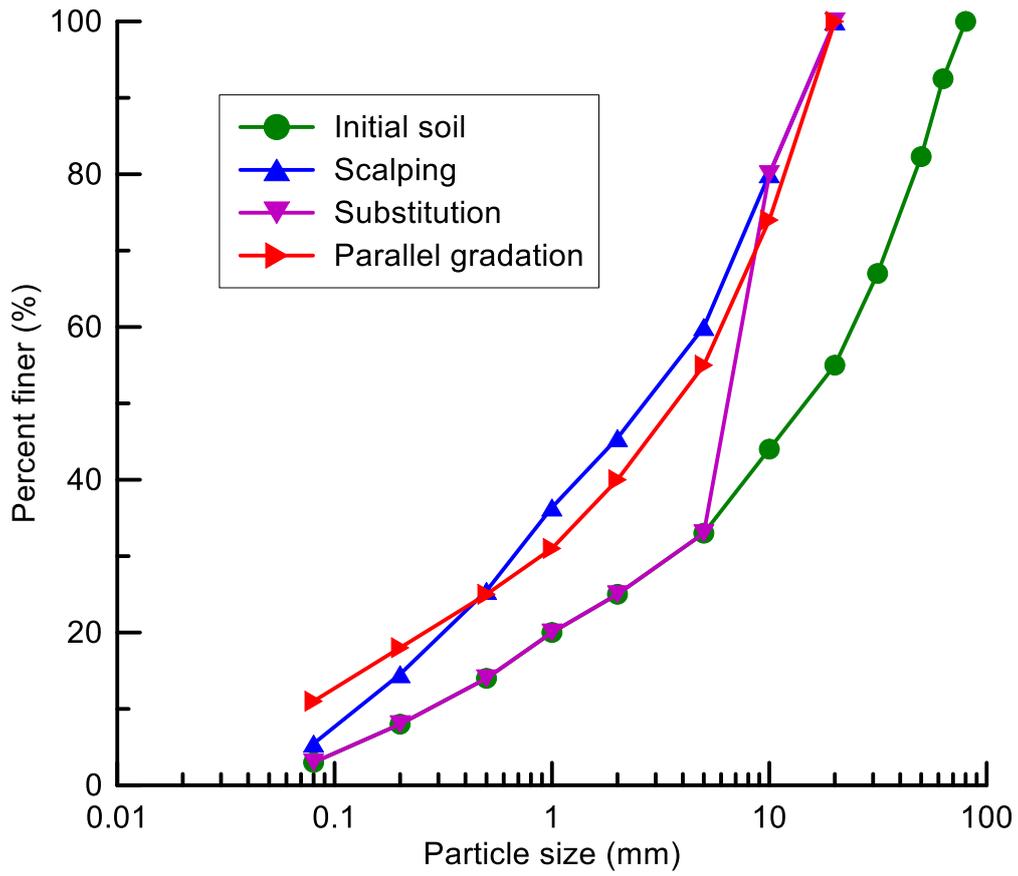


Figure 1: Modification of the grain size distribution curve for the three different approaches (initial soil:  $d_{max} = 80$  mm and reconstituted soils:  $d_{max} = 20$  mm)

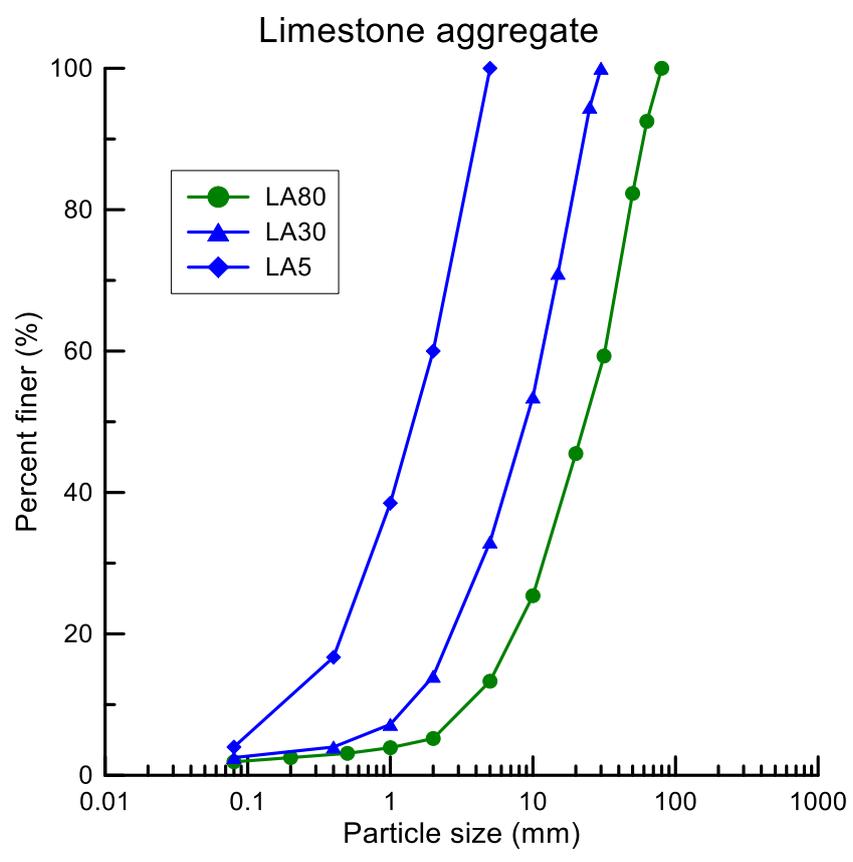
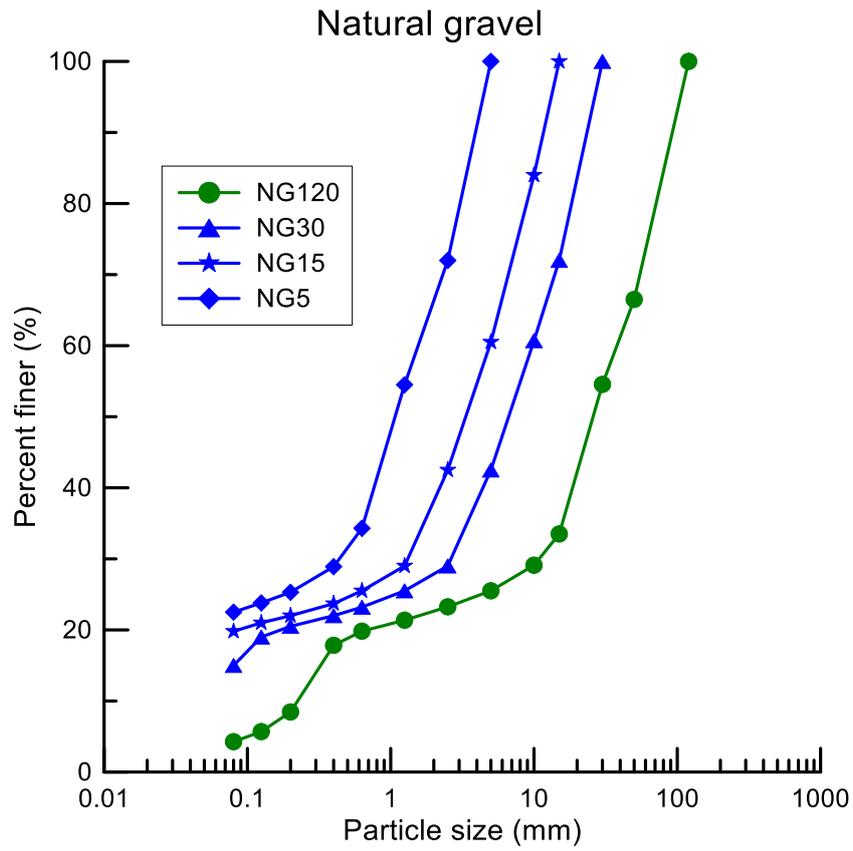


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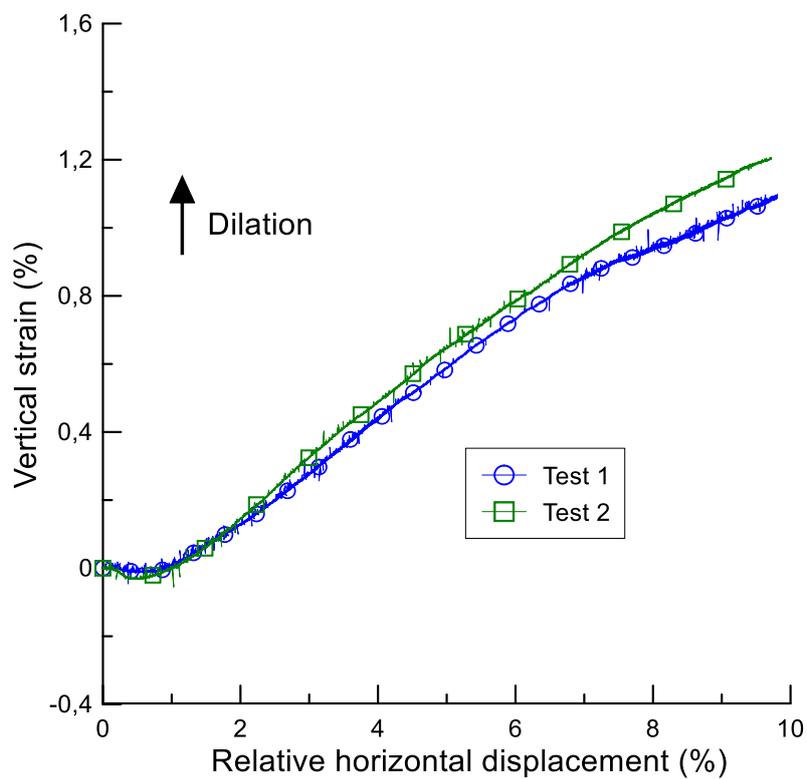
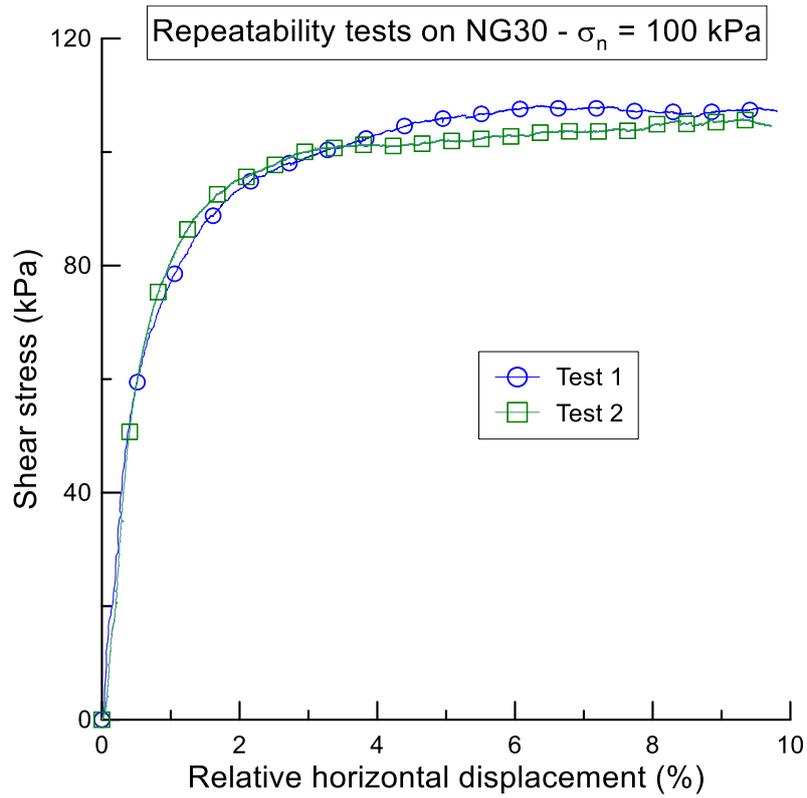


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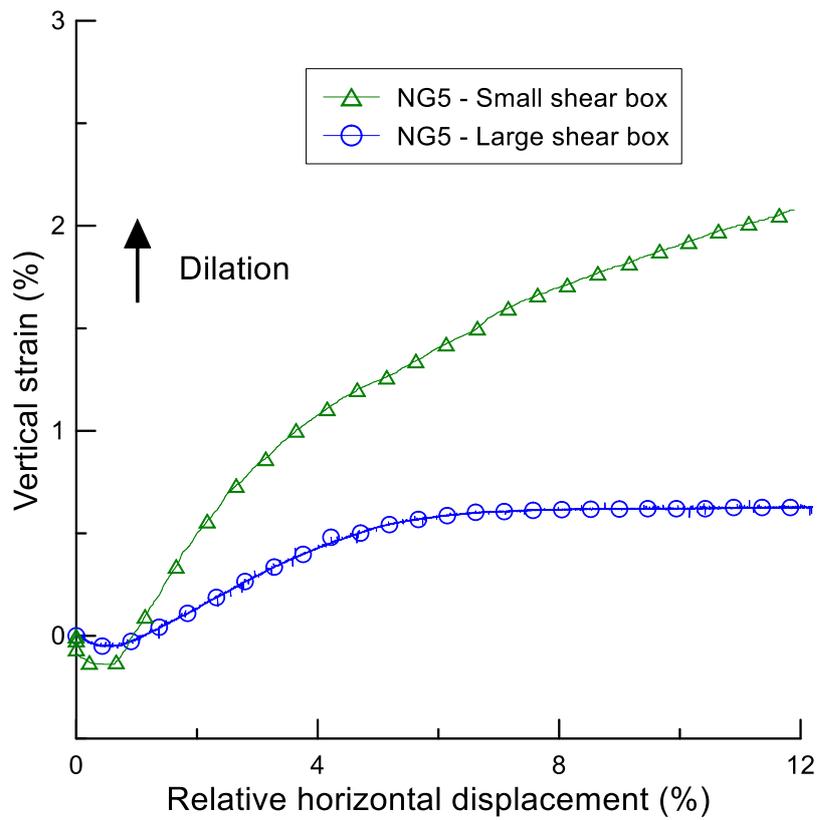
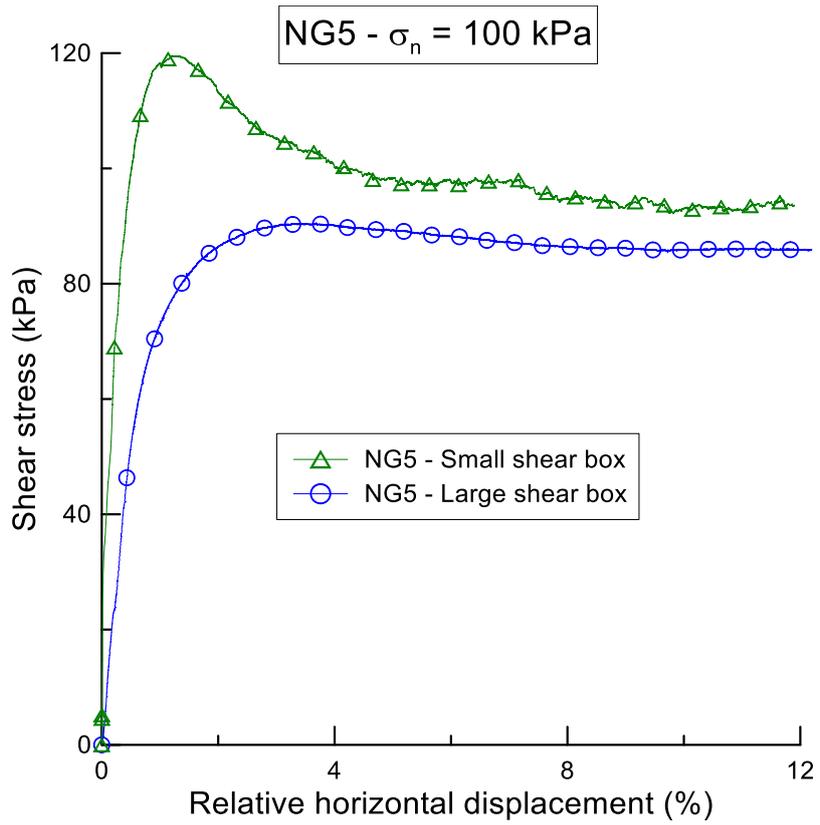


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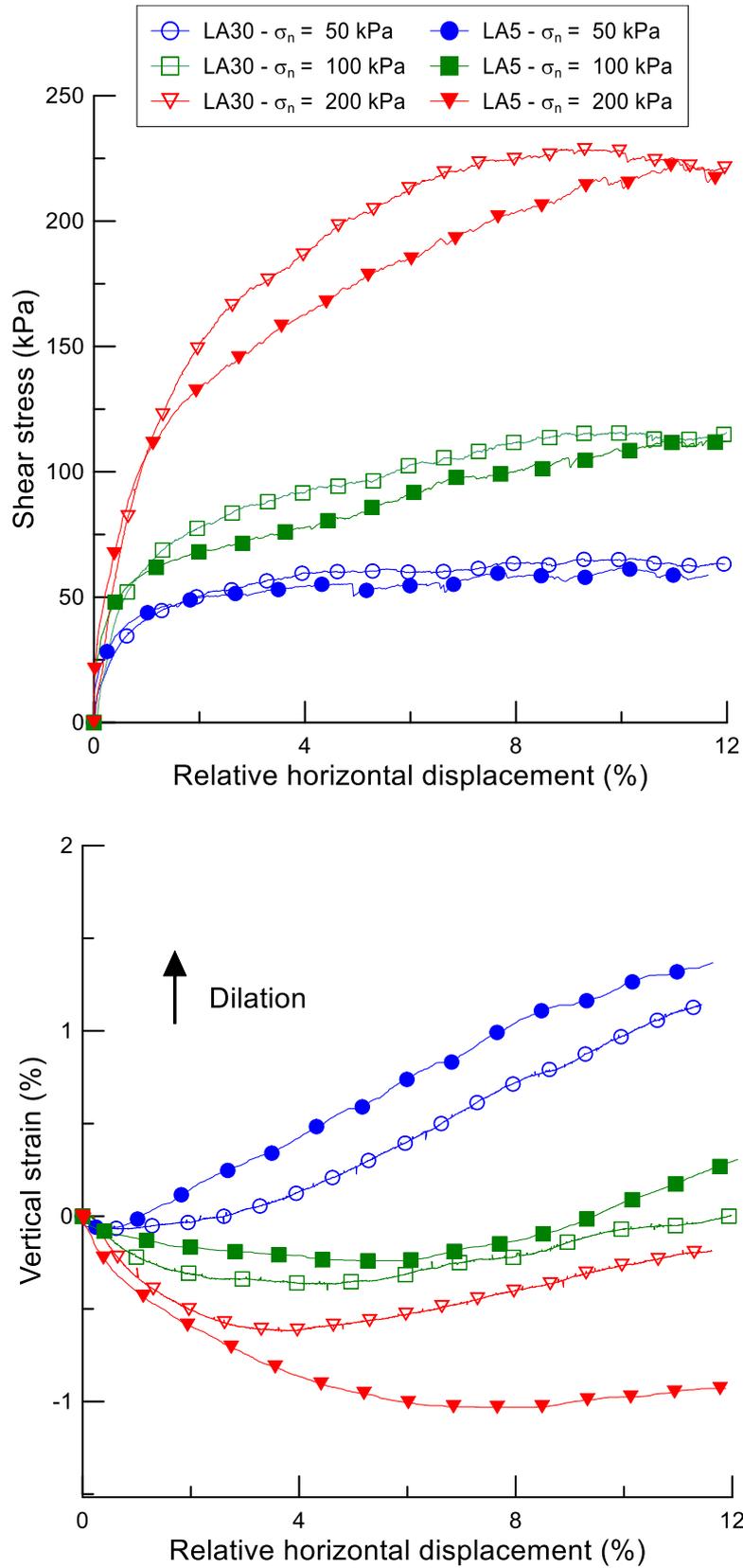


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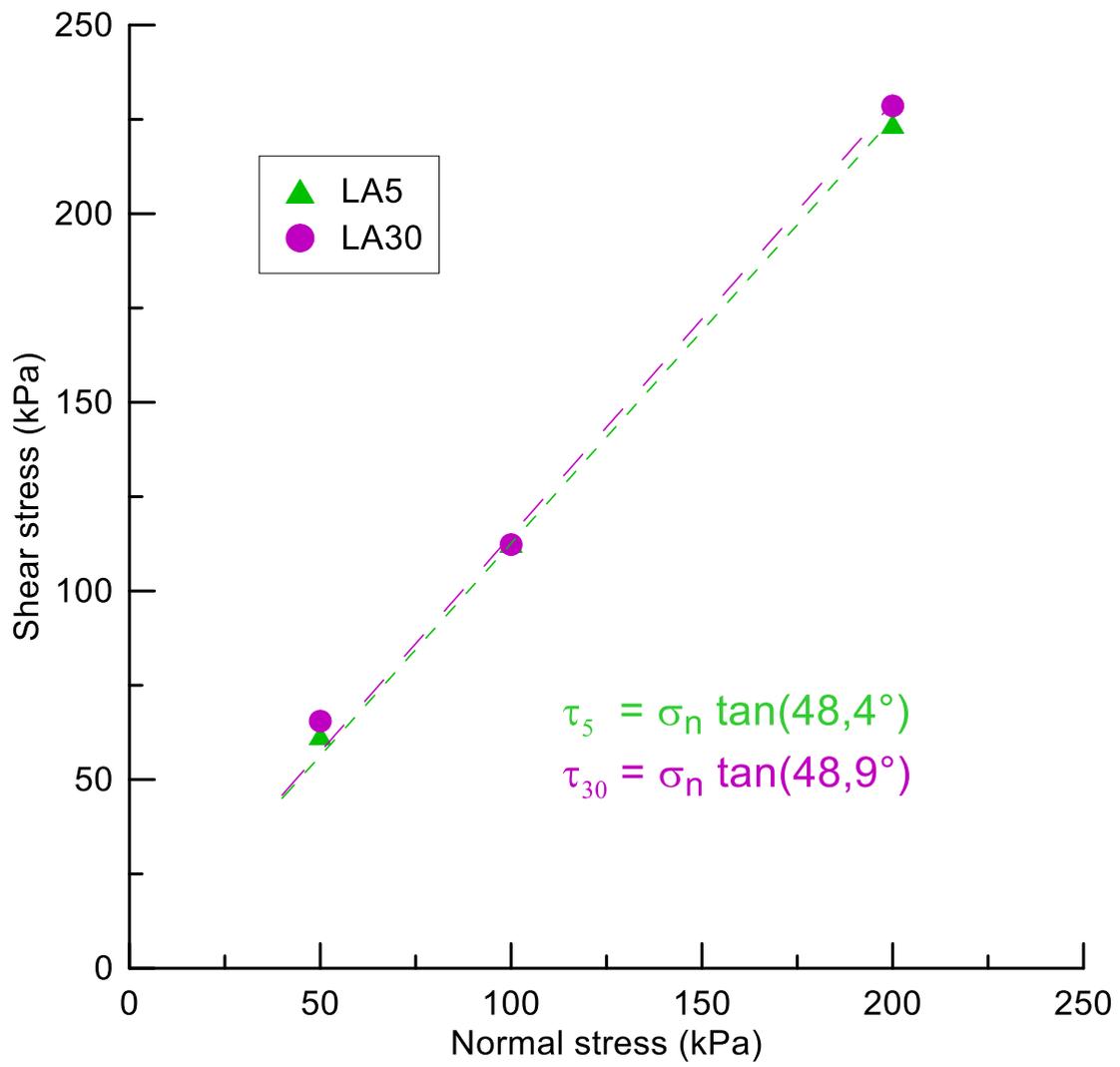


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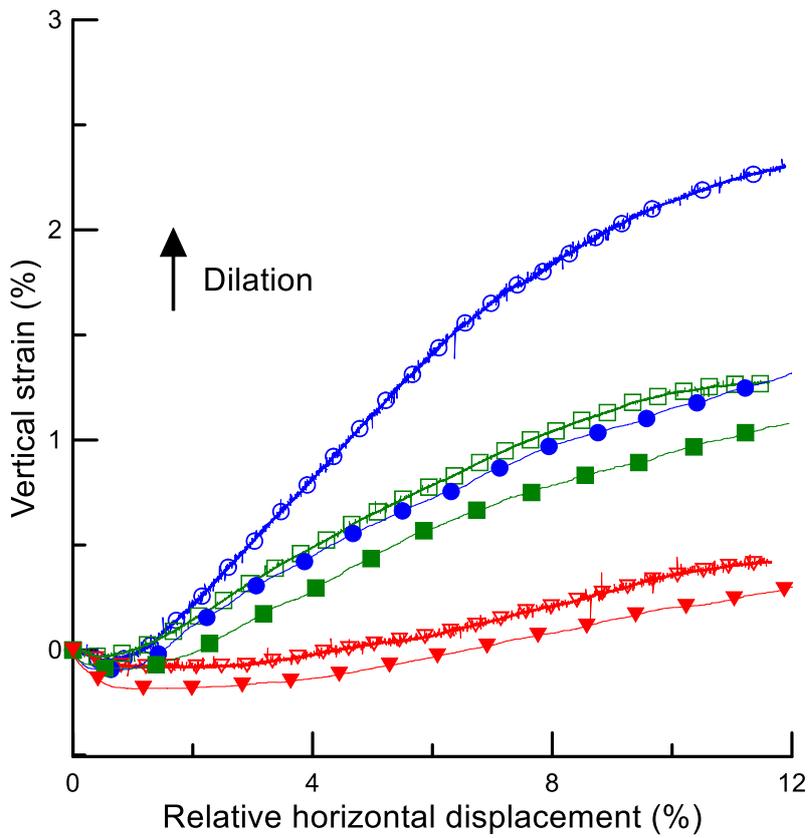
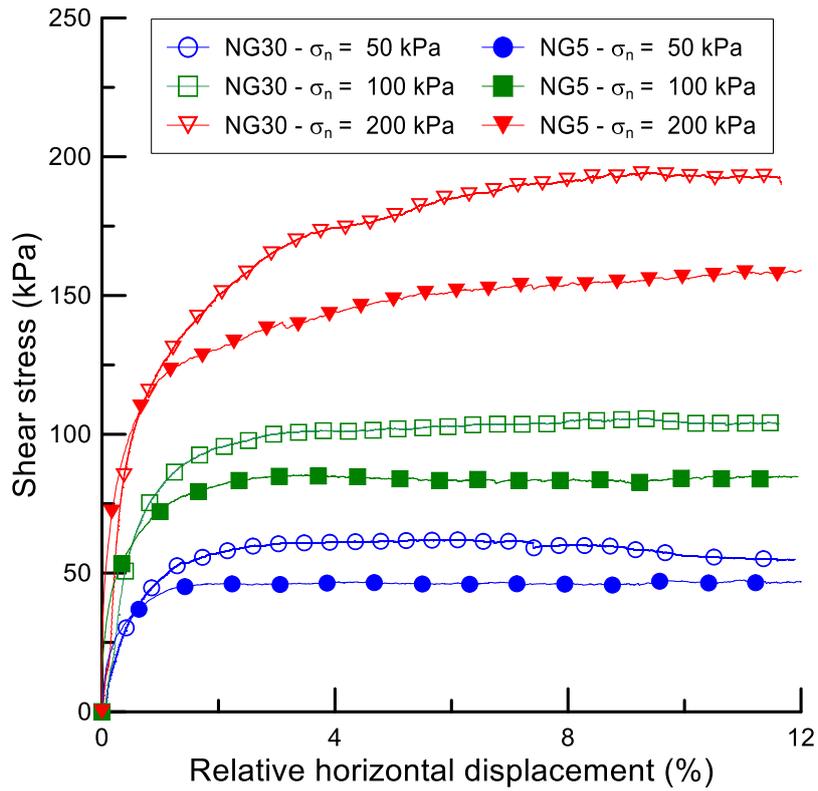


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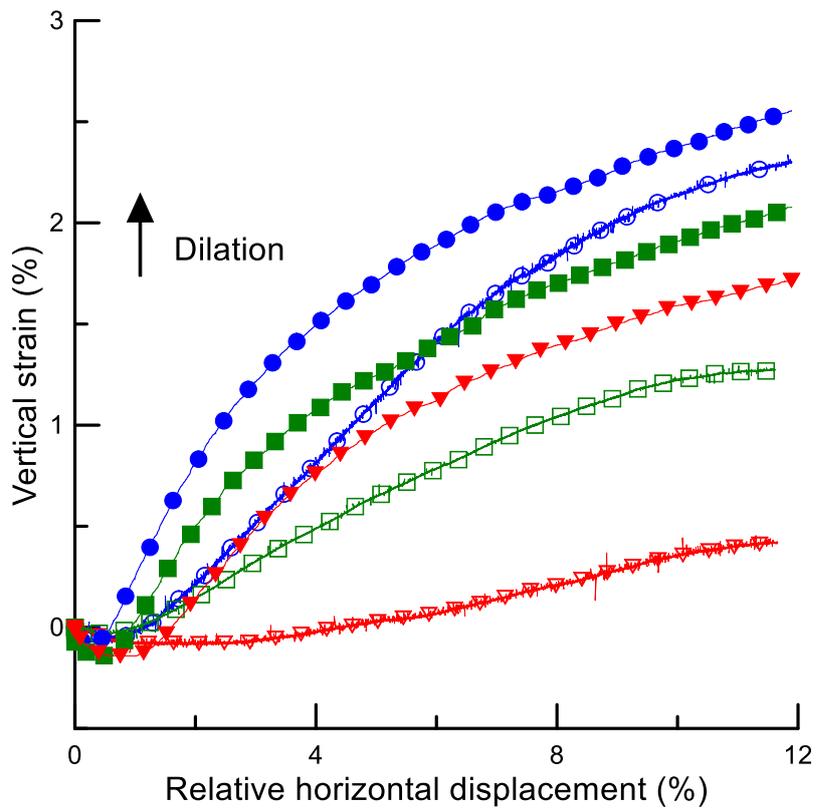
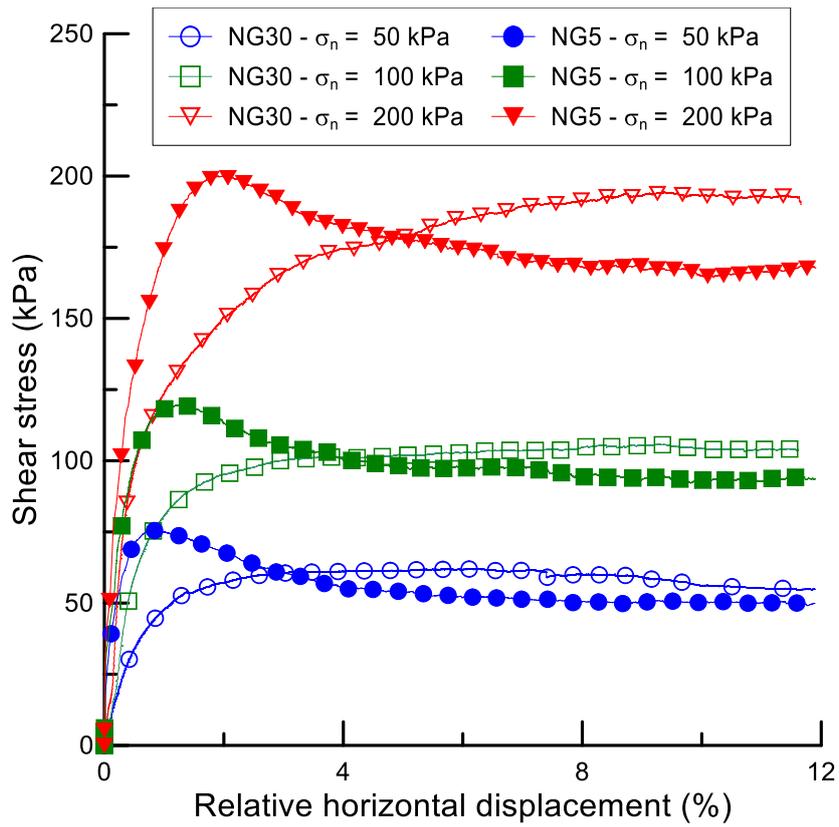


Figure 8: Shear stress-relative horizontal displacement and volume variation curves of NG5 and NG30 at  $\rho_d = 2 \text{ Mg/m}^3$ .

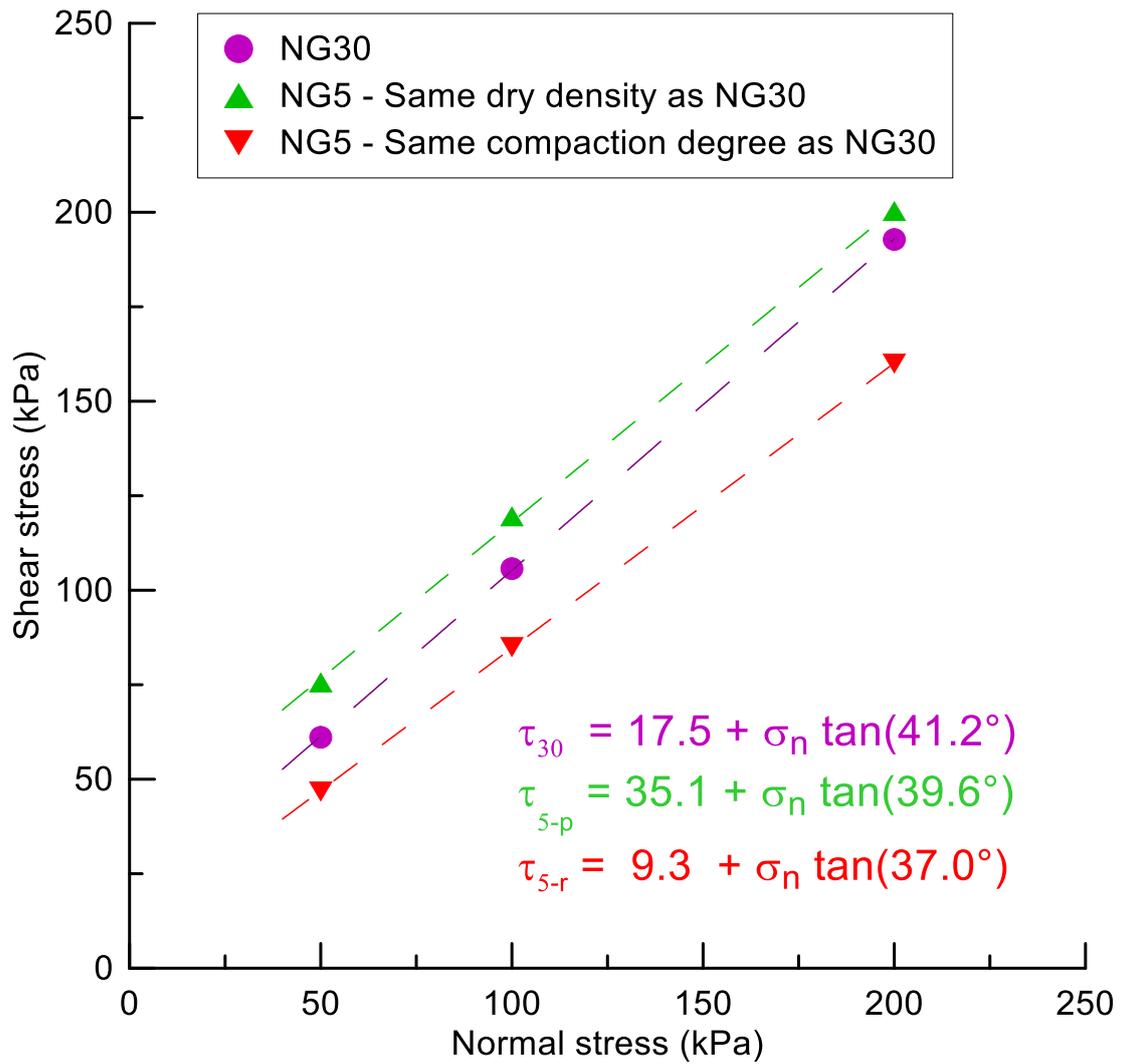


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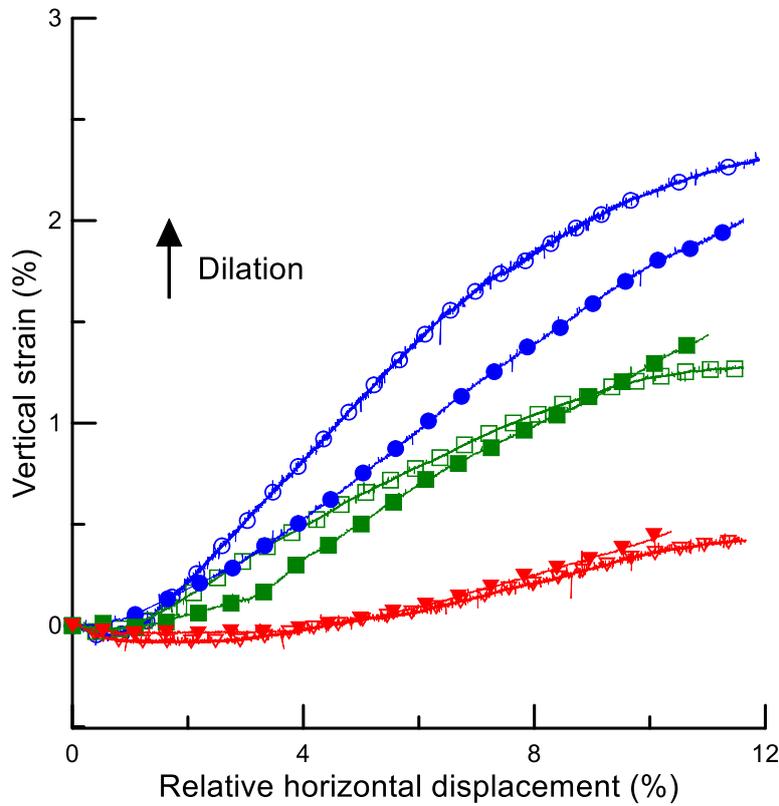
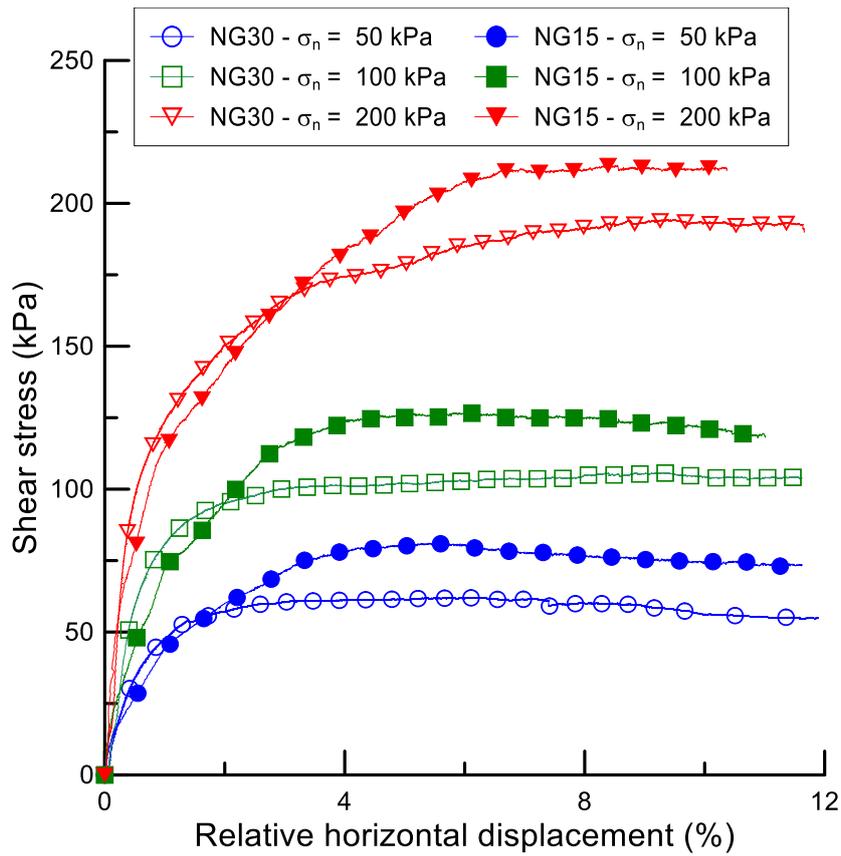


Figure 10: Shear stress-relative horizontal displacement and volume variation curves of NG15 and NG30 at  $\rho_d = 2 \text{ Mg/m}^3$ .

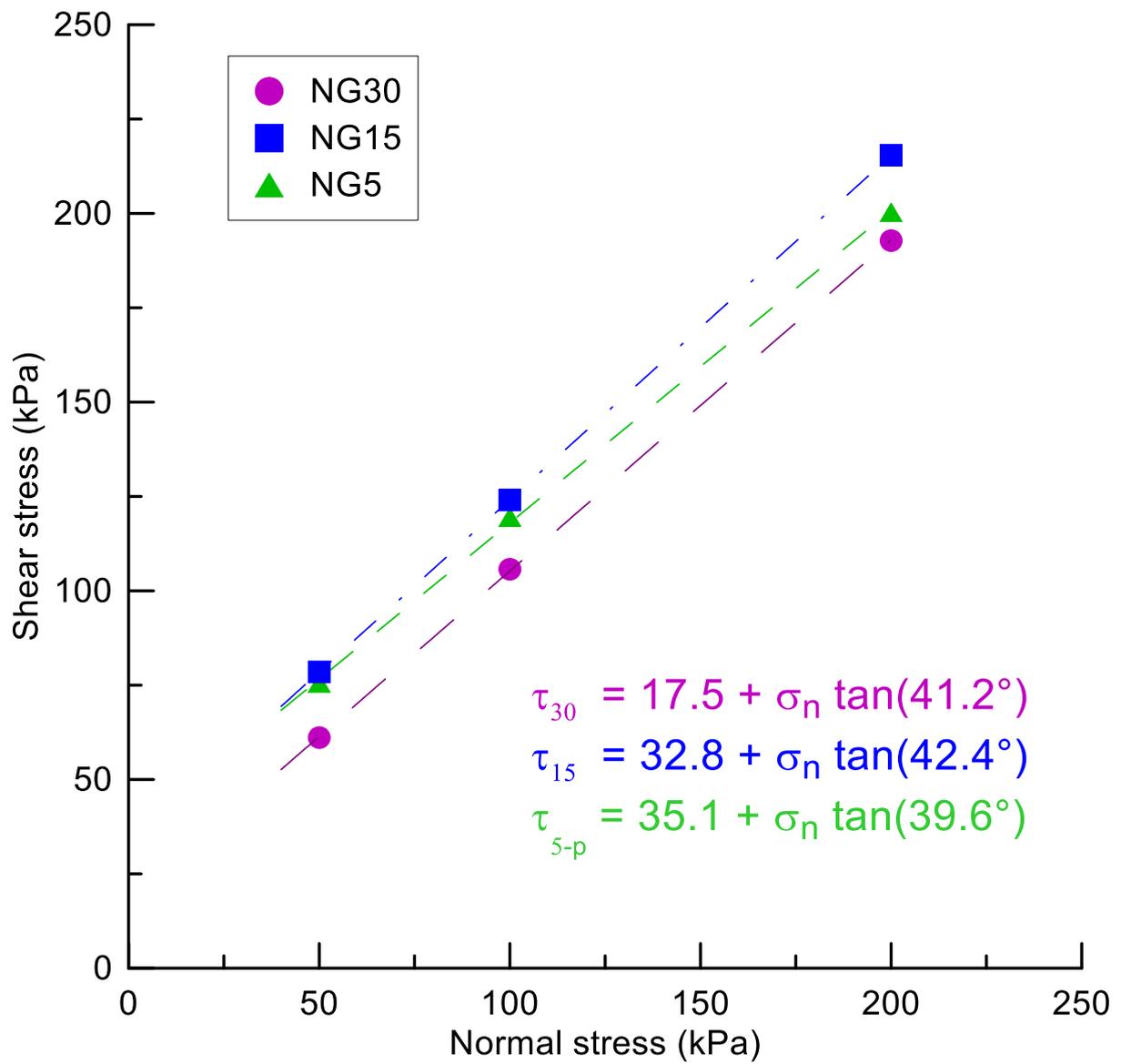


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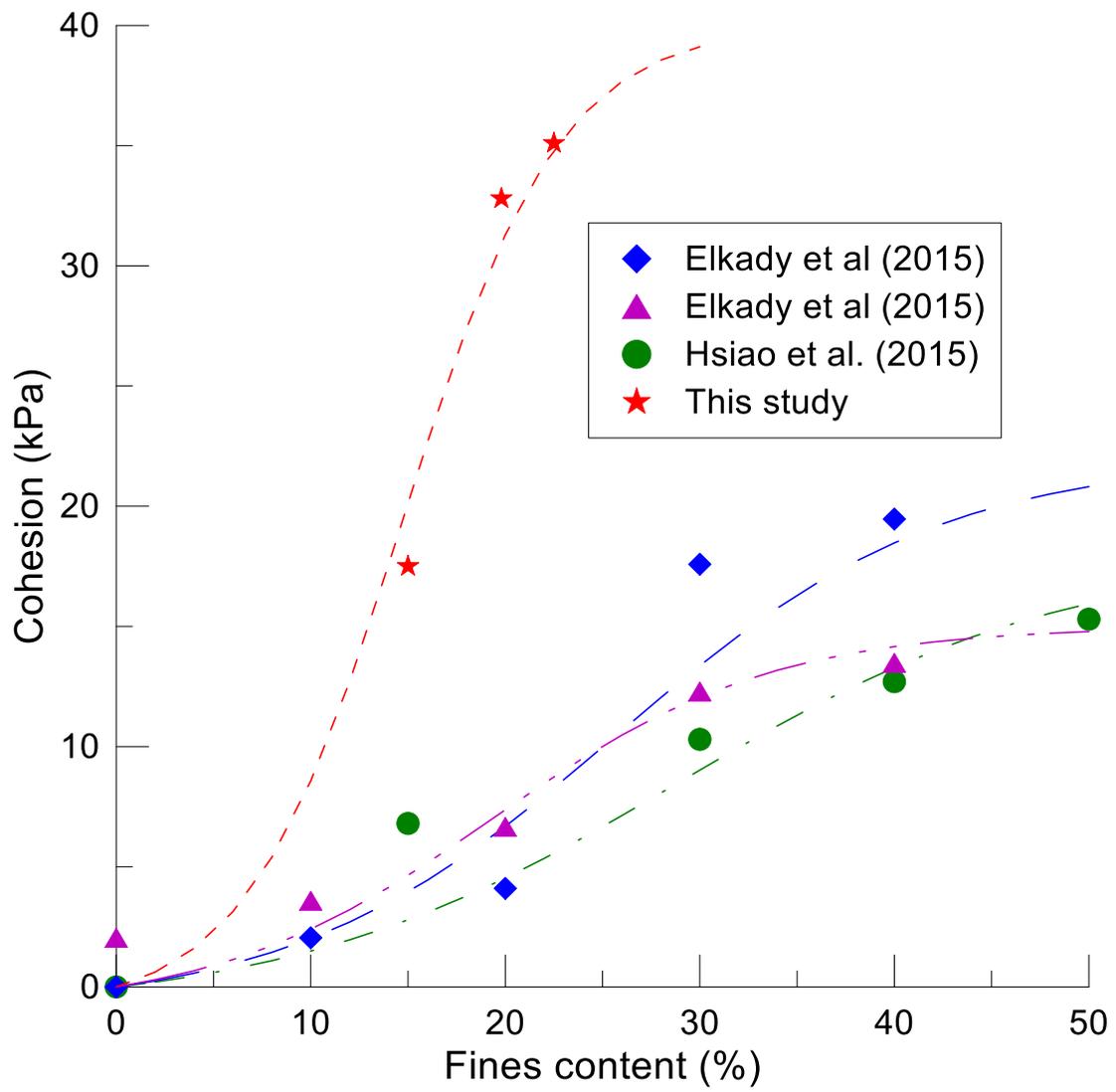


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