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SHEAR STRENGTH PROPERTIES OF COARSE-GRAINED SOILS DETERMINED USING LARGE-SIZE DIRECT SHEAR TEST

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Abstract

The paper deals with an analysis of the shear strength behaviour of coarse-grained materials using large-sized direct shear apparatus. The shear strength curve of compacted coarse-grained material allows for determining the three typical shear strengths, i.e. the shear strength for a maximum density of the sample tested (shear strength at point A), the peak shear strength (shear strength at point P), and the critical shear strength achieved for a large horizontal movement (shear strength at point C). Professional laboratories usually apply for determining the critical shear strength two different methods. The first method considers the critical shear strength at point A and the second method considers the critical shear strength at point C. The aim of the article is not to critically evaluate both methods, but to analyze the results of shear tests of gravels. The results of a large series of tests showed that these stresses may differ from each other in some cases, especially for poorly graded materials. In the case of sandy soil, the shear strength at point A and at point C is equal. However, in the case of poorly graded gravel, the shear strength at point C is greater than the shear strength at point A. The greatest difference between these shear strengths was measured in the case of poorly graded medium gravels. The results showed that the ratio of shear strength at point C to the shear strength at point A increases with increasing the grain size of the material tested. The peak as well as the critical shear strength parameters of poorly graded gravels is better represented by a multi-linear or a nonlinear failure line.

Keywords:

Shear strength;
Coarse-grained soils;
Poorly graded gravels;
Shear stress apparatus;
Soil testing.

1 Introduction

The shear strength properties of coarse-grained materials can be determined using field or laboratory tests. Field tests can include in-situ testing such as dynamic penetration tests, see e.g. [1] or nonconventional methods such as, e.g., seismic-wave testing [2]. The laboratory tests can be executed using a large-size direct shear test apparatus, e.g. [3, 4] or triaxial tests, see e.g. [5-7]. In-situ tests such as dynamic penetration tests allow for determining the angle of shear strength as well as the relative density and deformation modulus based on the correlation formulas selected. An advantage of these tests is that they allow for determining changes in the properties over a depth. On the contrary, large-size direct shear tests allow for a more detailed determining of the behaviour of the shear strength of the soil tested. There are two main types of large-size direct shear test apparatus, i.e., stress controlled, e.g. [8], and displacement controlled, e.g. [3, 9]. The stress-controlled direct shear test apparatus is based on the principle that the shear stress is controlled and that the horizontal deformation (movement) is recorded, while the most used deformation controlled direct shear test apparatus has a controlled horizontal movement, and the shear stress is recorded. The most relevant results of the direct shear test are the parameters of the peak and critical shear strength. The peak and critical shear strength parameters can be determined in the case of dense coarse-grained soil and, in the case of the loose coarse-grained soil, only the critical shear strength can be determined.

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While the determination of the peak shear strength is clearly defined by a maximum shear stress, soil mechanics laboratories apply two different methods in determining the critical shear strength.

Based on the theory presented by Atkinson [10], the critical shear strength can be determined in two different states, i.e. when the soil sample reaches its maximum density, and when the peak stress state is exceeded and the critical void ratio is reached. In these cases, the dilatancy angle is equal to 0°. Atkinson [10] stated that the shear strength in both of these cases is exactly the same. The results of the series of large-scale direct shear tests showed that the shear strength measured at these points can vary from each other in some cases.

The current state of the problem was based on collecting and summarizing the results of the shear tests of some other researchers, e.g. [5, 11-14]. The results of other researchers, e.g. [3, 15, 16], which were focused on determining the shear strength properties of reinforced coarse-grained materials, were also used, because their studies included results of shear strength tests of unreinforced coarse-grained samples. The results included shear stress curves as well as the relationship between horizontal movements (displacements) and vertical deformations, which allowed for the confrontation of these results with the results of the study presented. Based on the analysis of the results presented by Vallejo, et al. [17], it can be seen that the difference between the shear strength at points C and A can be ignored in the case of sandy soil, which was also seen on the results presented by [18], and that the difference between these shear stresses increased with increases in the gravel content. The differences between these shear stresses can also be seen in the results presented by other authors, e.g. [19]. Kharanaghi and Briaud [4] published the results of a large-scale direct shear test on railroad ballast materials. They showed that the peak and critical shear strengths can be better interpreted by a nonlinear failure envelope instead of a linear Mohr-Coulomb failure line. Similar results were published by, e.g. Estaire and Santana [20]. This effect is mainly caused by the crushing of grains, which can be significant, especially in poorly graded materials, see, e.g. [21, 22]. In the case of sand, Feda [23] stated that the nonlinearity of the failure envelope is only measured for tests with the constant effective mean stress. He stated that for standard experiments with a constant effective radial stress, a constant value of the angle of internal friction was measured. Liu et al. [24] stated that increasing the vertical stress of the sample would increase the shear strength and decrease the dilating behaviour of the samples. They stated that the shear strength envelope of coarse-grained material is nonlinear and can be well represented by a power function.

The aim of the study presented in this paper was to analyse the results of large-scale direct shear tests focused on determining the shear strength at described points, to analyse when and how the shear strength of coarse-grained materials determined in the selected points, i.e. points A and C; may differ from each other, and to recommend the most appropriate method for the evaluation of shear strength tests of poorly graded coarse-grained materials.

2 Theoretical background

The theoretical background to the shear tests of soils was published in detail by, e.g. [25-27]. The idealized behaviour of soil in a shear test was presented by [10]. The shear strength curves of dense and loose samples are presented in Fig. 1a, where the relationship between shear stress τ´ and shear strain γ is shown. The dense (compacted) samples during a shear test increase their volume; they dilate, and loose samples during the shear test decrease their volume, they contract, see Fig. 1b, where ε_v represents a vertical strain. The void ratio *e* of a dense sample increases its value during the shear test, and the void ratio of a loose sample decreases its value, see Fig. 1c. A loose sample can only reach critical shear strength τ_c; however, a dense sample can reach peak shear strength τ_p as well as critical shear strength τ_c. Independently, upon the initial state of the sample, the critical shear strength of the sample has the same value in all the cases. The critical shear strength is obtained at the point when the critical void ratio of the soil is reached [28], see Fig. 1.

Fig 2a shows how the ratio of shear stress τ´ and normal stress σ´ change with horizontal displacement δ_h. Fig 2b shows how the height change δ_v with horizontal displacement δ_h. Atkinson [10] stated that the critical shear strength can be determined at two points, which are marked A and C. At point A, the maximum density of the sample is reached; however, no dilation occurs, the angle of dilatancy Ψ' = 0°. After point A, the sample begins to dilate; the angle of dilatancy increases until the maximum value (Ψ'_{max}), which corresponds to the peak state (point P), is reached. After exceeding the peak state, the shear strength decreases and stabilizes at the critical stress state, when the critical void ratio e_c is reached (point C), and the angle of dilatancy Ψ' = 0°. However, as stated in the introduction, the results of large-scale shear tests showed that in the case of poorly graded gravel

soils, the shear strength at point C can be higher than the shear strength at point A. Atkinson [10] also stated that in the critical shear strength, there is no change in the normal stress, shear stress, and void ratio. Taking into account this statement, the critical shear strength is represented by the shear strength at point C, which is also in agreement with statement presented by, e.g. [28].

Taylor [25] was one of the first to introduce the term “interlocking effect of soil particles” in a description of the soil dilatancy. He stated that the loss of shear strength after a peak is attributed to a decrease in interlocking as the sample decreases in density. The interlocking effect can be negligible in the 0 - A path, because the sample is still decreasing its volume and the grains are being pushed into each other. After exceeding point A, the interlocking effect is fully mobilized and is reflected in the shear strength measured. In the case of poorly graded coarse-grained gravels, after exceeding the peak shear strength, the shear strength caused by the interlocking effect decreases but never completely disappears. Depending on the grain sizes, shapes, porosity, and stress state, the interlocking effect also affects the shear strength after exceeding the peak state.

In the case of sandy soil, a relationship between the peak angle of shear strength φ'_p, the critical angle of shear strength φ'_c, and the maximal angle of dilatancy $\psi'_{\max}$ was presented by [10]:

$$\varphi'_p = \varphi'_c + 0.8\,\Psi'_{\max}\,. \tag{1}$$

The expression of the equation in a similar form was presented by [29]:

$$\varphi'_p = \varphi'_c + b\,\Psi'_{\max}\,. \tag{2}$$

where *b* is a coefficient replacing a constant value of 0.8. They also presented a decreasing in coefficient *b* with an increasing of gravel fraction.

Especially in the practical geotechnical engineering, the simplification of determining the angle of dilatancy by following equation (3) is used, see e.g. [30, 31].

$$\Psi'_{\max} = \varphi'_p - 30°. \tag{3}$$

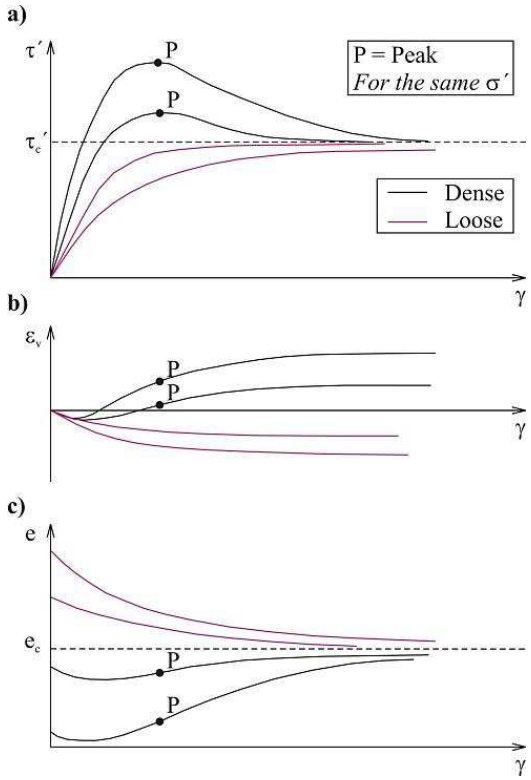


Fig. 1: Drained shear strength of a soil, according to [10].

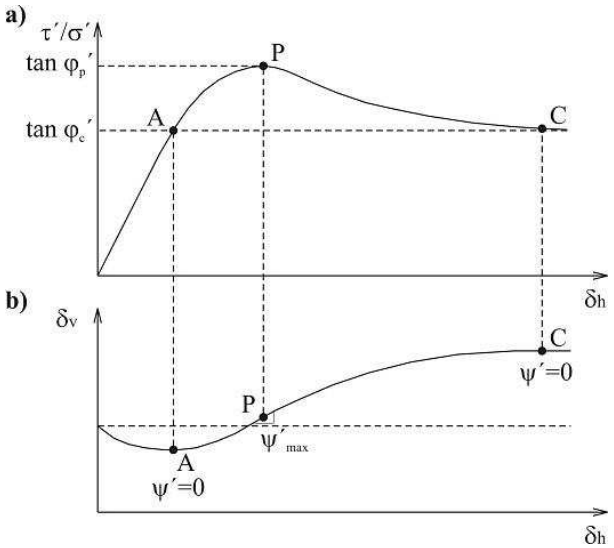


Fig. 2: Shear strength curve and dilation behaviour, according to [10].

3 Index properties of soil samples tested

The study presented is based on the results of about 30 large-scale direct shear tests. The article presents the results of 9 tests for the samples selected. The results that were quite similar to each other are omitted for the sake of clarity. The grain-size distribution curves of the tested samples selected are plotted in Fig. 3. The index properties of the selected soils are presented in Table 1. According to the classification system stated in STN 72 1001: 2010 (Classification of soil and rock), the samples 1, 2, 5, 6, 8, and 9 are classified as gravel poorly graded (GP), samples 3 and 4 as gravel well graded (GW), and Sample 7 as sand poorly graded (SP). Samples 1, 3, 4, 5, 6 consist of pebble gravel. Samples 2, 8 and 9 consist of crushed pebble gravel. Sample 7 consists of fluvial sand. The mean grain size parameter d_{50} for each sample is shown in Table 1. It was necessary to conduct laboratory tests for each sample to determine the minimum $\rho_{d,min}$ and maximum $\rho_{d,max}$ bulk density, Table 1, as well as the density of the solid particles ρ_s. These parameters allowed for determining the density index and void ratio and monitoring them during the tests. The bulk density ρ and the density index I_D of the sample before the large-scale shear strength test are stated in Table 1. The density index was determined using the following formula:

$$I_D = \frac{[\rho_{d,\max}(\rho_d - \rho_{d,\min})]}{[\rho_d(\rho_{d,\max} - \rho_{d,\min})]} . \tag{4}$$

The minimum and maximum bulk density was determined using standard laboratory tests. The unit weight of the solid particles ρ_s was determined using the following equation:

$$\rho_s = \frac{m_d}{V_m - V_w} , \tag{5}$$

where m_d is the dry weight of the sample; V_m is the volume of the testing container; V_w is the volume filled by water (volume of the pores). The void ratio e was determined using the formula:

$$e = \frac{V_v}{V_s} , \tag{6}$$

where V_v is the volume of void-space and V_s is the volume of solids.

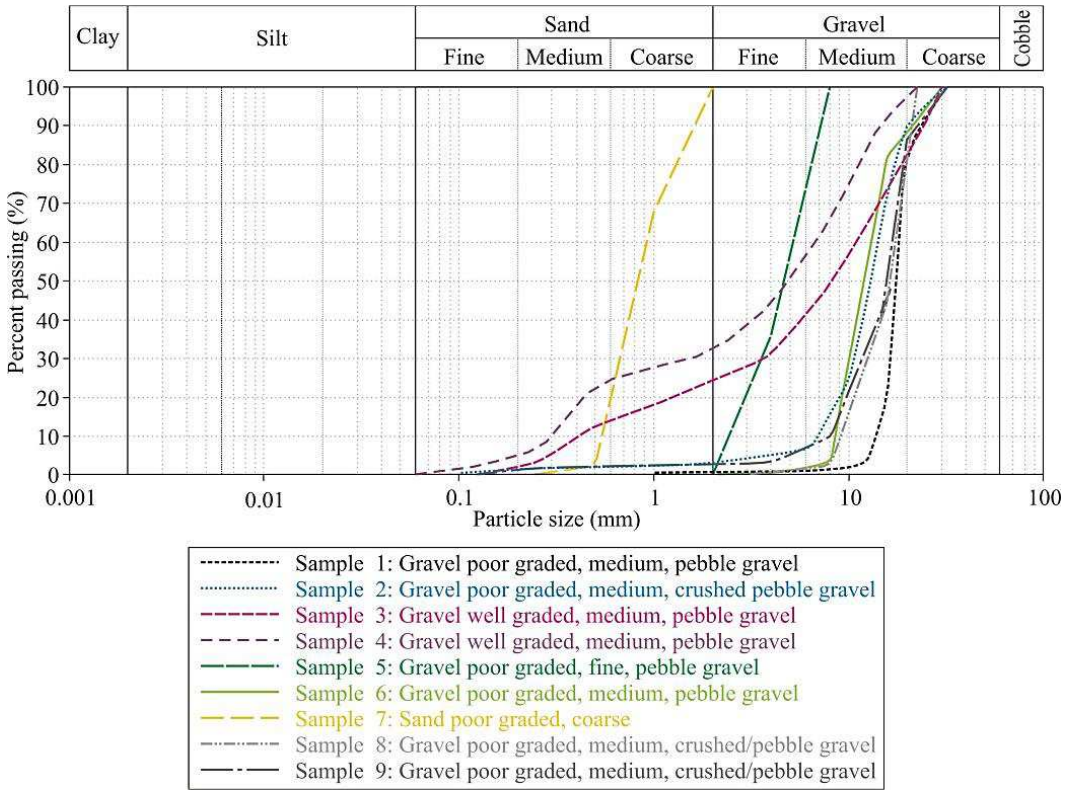


Fig. 3: Grain-size distribution curves of the soils tested.

Table 1: Selected index properties of tested soils.

Sample	Soil symbol	Diameter of grain d_{50} [mm]	Density index I_D	Density of the solid particles ρ_s [kgm^{-3}]	Bulk density		
					$\rho_{d,min}$ [kgm^{-3}]	$\rho_{d,max}$ [kgm^{-3}]	ρ_d [kgm^{-3}]
Sample 1	GP	17.80	0.493	2629.1	1512.8	1884.8	1675.9
Sample 2	GP	13.00	0.533	2575.7	1435.5	1966.9	1676.8
Sample 3	GW	8.45	0.821	2600.6	1774.9	2232.3	2134.0
Sample 4	GW	5.13	0.876	2569.2	1817.6	2248.2	2184.2
Sample 5	GP	4.90	0.975	2509.0	1620.0	1823.3	1817.7
Sample 6	GP	12.72	0.716	2540.4	1527.4	1799.1	1712.6
Sample 7	SP	0.86	0.984	2511.8	1448.1	1751.1	1745.4
Sample 8	GP	16.37	0.921	2644.7	1493.8	1740.1	1717.8
Sample 9	GP	17.05	0.869	2623.9	1481.7	1764.2	1721.3

4 Preparation and execution of large-scale direct shear tests

The shear tests were executed using the large-size SHEARMATIC 27-WF 2304 direct shear test apparatus. This is a horizontal movement (deformation) controlled direct shear test apparatus. The dimensions of the shear box are 300 x 300 x 200 mm (length x width x height). The shearing area is in the middle of the height of the shear box. The movement of the lower part of the shear box is controlled by a servo unit. The upper part of the shear box is fixed to a load cell to determine the shear force. The tests are automated and the recording includes: normal stress, shear force (stress), vertical deformation, and horizontal movement. The speed of the horizontal movement was equal to 0.5 mm·min^{-1} in the case of the gravel samples and a 0.25 mm·min^{-1} in the case of the sand sample. The tests were performed for a horizontal movement slightly exceeding 60 mm. The tests were done for normal stresses, i.e. σ_n = 50, 100 and 150 kPa when there was no significant crushing of the grains. Some of the tests were also executed for the normal stress of σ_n = 400 kPa, but in those cases there was a significant crushing of the grains during the shear testing. The testing procedure consisted of two main phases, i.e. consolidation and the shearing test. Because only dry coarse-grained materials were tested, the first phase is only represented by a compaction (quick deformation) of the sample for the given normal stress without any further consolidation (deformation over time). The times of this compaction phase were about 30 min for the gravel samples and about 120 min for the sand sample. The shear strength tests were executed using the following procedure:

- A sample of a known weight (about 20 kg in the case of the gravel and about 30 kg in the case of the sand) was compacted in 5 layers into the shearing box. The samples were compacted using wood plates and a rubber hammer to the maximum possible state.
- After the compaction of the sample, the height of the sample was measured, which allowed for determining the bulk density and the density index of the sample tested.
- The first phase of the testing, i.e., compaction of sample, was performed.
- After the compaction of sample and based on the vertical deformation measured, the bulk density ρ and the density index I_D are determined again as an initial state for the shearing test.
- The shear strength test is performed.

The shear strength tests consisted of 3 or 4 measurements, based on the normal stresses selected. Each shear strength test was executed with a new soil sample. The bulk density and the density index were controlled after the compaction of the soil in the shearing box to obtain the same values for all the tests with different normal stresses.

In the cases of the 6 samples, the shear strength tests were performed for the "loose" and "dense" samples. The tests for the dense samples were executed exactly according to the above description. In the case of the loose samples, the only difference was the sample was put into the shear box. The sample was not compacted, but was only loosely loaded.

5 Results of testing

5.1 Confrontation of tests with theoretical background

The first part of the study was focused on verification if the horizontal deformation of about 60 mm is great enough to measure the critical shear strength. Six gravel samples were tested twice, i.e.

the first test series was done for a dense state, and the second test series was done for a loose state. The results of tests for samples tested showed similar results. For the sake of clarity, only the results of Sample 1 are presented (Figs. 4 and 5). The tests were executed for normal stresses, i.e. 50, 100, and 150 kPa. The density index of the loose sample was about 0.19, and the density index of the dense sample was about 0.50 (the highest value reached for the compaction by a rubber hammer). The shear strength curves of the loose and dense Sample 1 are shown in Fig. 4. The dense sample reached a peak state for the horizontal movement within a range of 20 - 30 mm (depending on the normal stress). The critical shear strength that was reached for horizontal movement exceeded about 45 mm. The loose sample only achieved the critical shear strength. The critical shear strength is approximately the same for the loose as well as the dense samples, which is in agreement with the theoretical background.

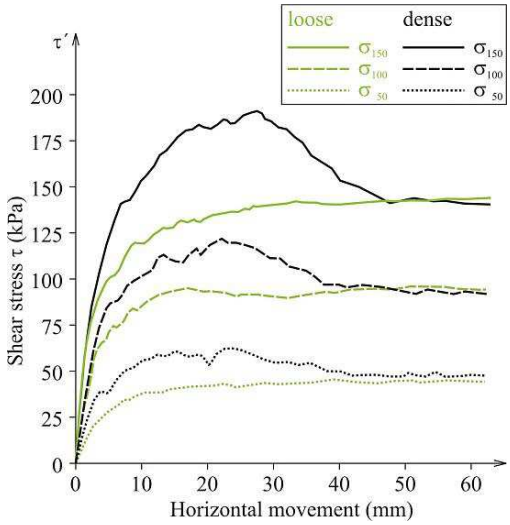


Fig. 4: Shear strength curves of the loose and dense Sample 1.

The relationship between the horizontal movement and the vertical deformation is shown in Fig. 5a. The dense sample had a typical dilative behaviour. The loose sample initially slightly decreased its volume; however, when exceeding a horizontal movement of about 10 mm, the sample had dilative behaviour, and the value of the vertical deformation was about half of the vertical deformation of the dense sample. This behaviour is typical of poorly graded gravels as stated by, e.g. [28]. The relationship between the void ratio and horizontal movement is shown in Fig. 5b.

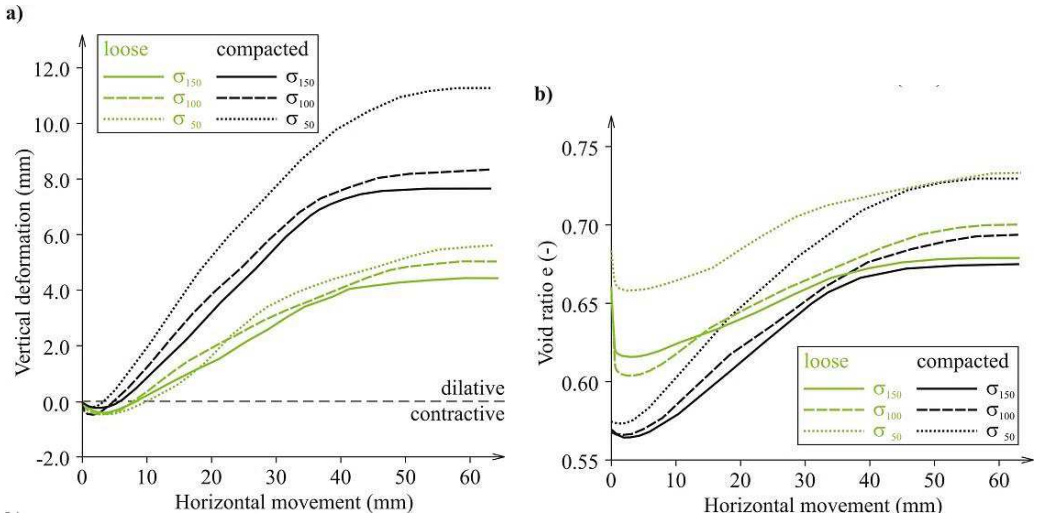


Fig. 5: Horizontal movement vs. vertical deformation (above) and porosity number (below) for Sample 1.

At the initial stage, the void ratio is different for the loose and dense samples, which depends on the initial density of the sample. When the critical shear strength is reached for a horizontal movement

greater than about 45 mm, the void ratio of the loose and dense samples, depending on the normal stress, is approximately the same. This is also in good agreement with the theoretical background, see e.g. [28]. Based on these results it can also be stated that the horizontal movement of about 60 mm is sufficient to achieve the critical shear strength.

5.2 Shear strength at selected points

The second part of the study was based on the execution of large-size shear strength tests. The results were focused on plotting the shear strength curves and vertical deformations. The shear strength was determined at points A, P, and C. The article includes some of the most relevant results. The results for sample 6, medium gravel poor grade, are shown in Fig. 6. The upper graph includes the shear strength curves for the selected normal stresses, and the lower graph includes the distribution of the vertical deformation. The shear strength curves as well as the curves of the vertical deformation are plotted for two independent scales. The black curves correspond to the black scale (0 - 60 mm), and the blue curves correspond to the blue scale (0 - 12 mm). The blue scale was added due to the more relevant determination of the shear strength at point A when the maximum density of the sample was reached. The shear strength at point A was determined for the maximum density of the samples when the horizontal movement was within a range of 0.9 - 1.8 mm. The peak shear strength was reached for a horizontal movement within a range of 14 - 18 mm. The critical shear strength was reached when the horizontal movement exceeded about 40 mm, which can be seen from the vertical deformation of the sample. Average critical shear strength is marked by green dot-line and represents the value of the critical shear strength, which was used in the evaluation of the results. The vertical deformation of the sample was about 9 mm (depending on the normal stress). For used normal stresses, i.e. 50, 100, and 150 kPa, the ratios of the shear strength at point C to the shear strength at point A were about 1.69, 1.47, and 1.53. The determination of the shear strength properties is summarized in section 5.3 for all the samples tested.

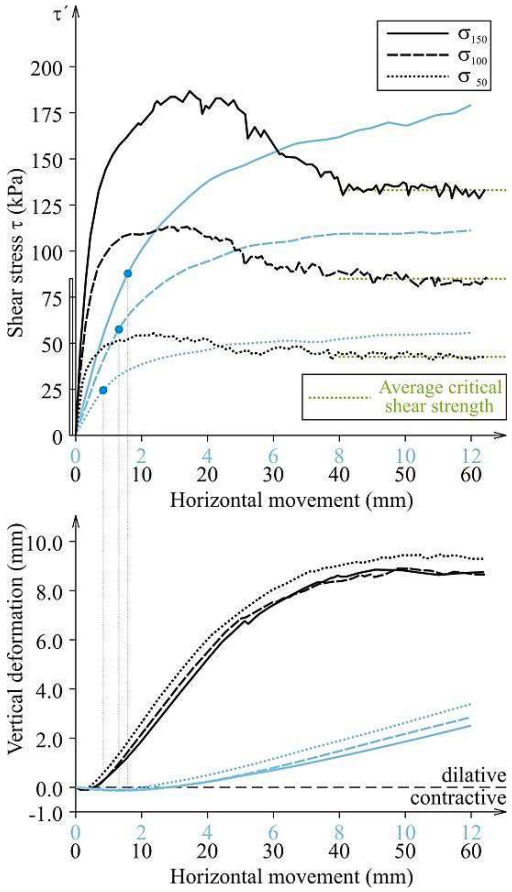


Fig. 6: The results of the shear strength tests for Sample 6.

The results of the shear strength test for sample 5, fine gravel poorly graded with a fraction of 2 - 8 mm, are presented in Fig. 7. The results of the measurements were a part of a previous study

presented by the authors [32]. The shear strength at point A was reached for a horizontal movement of about 0.8 - 1.0 mm and the peak shear strength for a horizontal movement of about 4 - 6 mm. The e_{crit} was reached for a horizontal movement exceeding about 25 mm, and the vertical deformation was within a range of 3 - 4 mm, depending on the normal stress. The critical shear strength used for the evaluation is marked by green dot-line. The ratios of the shear strength at point C to the shear strength at point A, for normal stresses used, were about 1.33, 1.22, and 1.20.

In the same way the results of sample 4, the gravel well graded are presented; the gravel consists of a fraction of 0.1 - 22.0 mm, Fig. 8. The results showed that the shear strength at point A is reached for a horizontal movement of about 1 - 1.2 mm, and the peak shear strength is reached for a horizontal movement of about 5 - 6 mm. The value of e_{crit} is reached for a horizontal movement exceeding about 40 mm, and the values of the vertical deformation was within a range of 3 - 3.2 mm. The shear strength reached for e_{crit} was in average about 1.48 times greater than the shear strength at point A. The shear strength properties of these soils are summarized in section 5.3.

The large-scale direct shear tests of Samples 7 - 9 were executed for 4 normal stresses, i.e., 50, 100, 150, and 400 kPa. While in the range of normal stresses of 0 - 150 kPa, there was only a small crushing of the grains, at a normal stress of 400 kPa, the crushing of the grains was very significant. This effect was noticeable, especially in the cases of the gravel samples (Samples 8 and 9).

The results of the shear strength test of Sample 7, sand poorly graded, are shown in Fig. 9. The shear strength at point A was determined for horizontal movement in the range of 1.1 mm for the σ_n = 50 kPa up to 3.5 mm for the σ_n = 400 kPa. The dilatancy of the sample was small and adequate for the grain sizes. The critical shear strength was determined for a horizontal movement of 40 – 60 mm, when a stable distribution of shear strength was reached (marked by green dot-line in Fig. 9). In the case of the sandy samples, the results of the shear strength test were in very good agreement with the results of the idealized shear strength of the soil presented by [10]. For normal stresses 50, 100, and 150 kPa, the ratios of the shear strength at point C to the shear strength at point A were about 1.05, 0.99, and 1.02. The values were close to 1.0, so it can be stated that the shear strengths at point C and point A are similar to each other in the case of sandy sample.

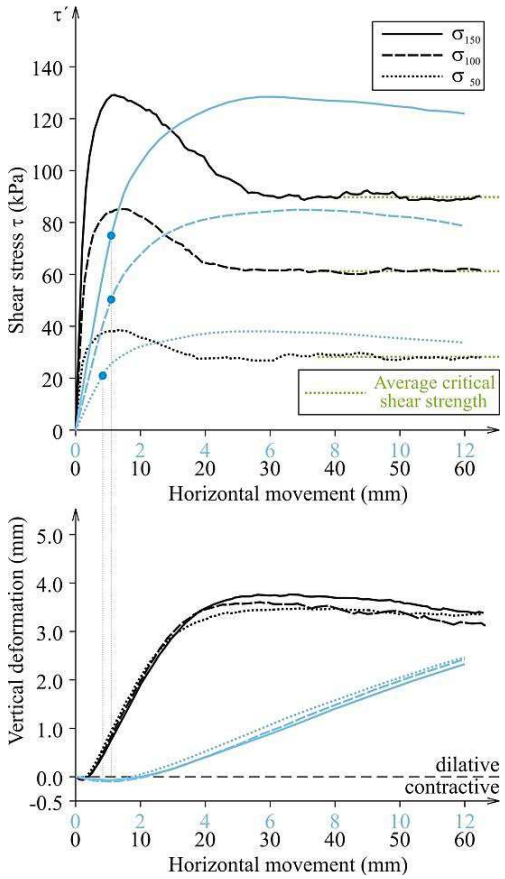


Fig. 7: The results of the shear strength tests for Sample 5.

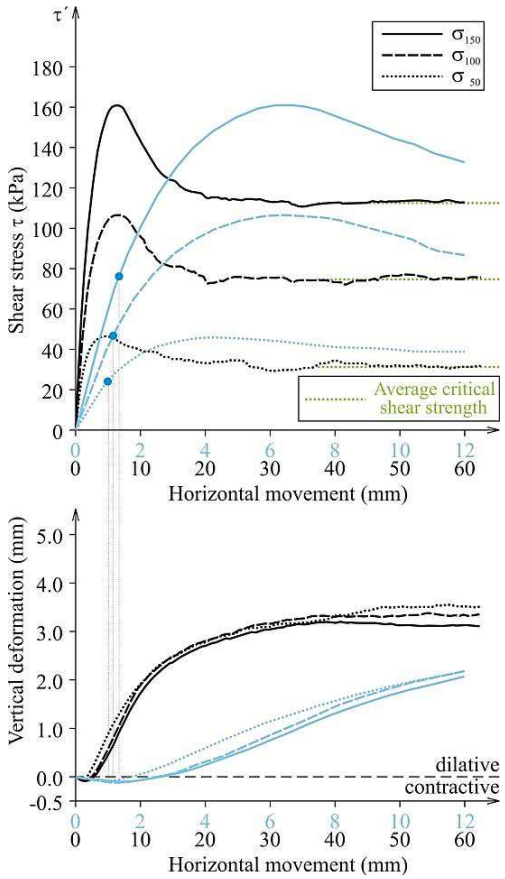


Fig. 8: The results of the shear strength tests for Sample 4.

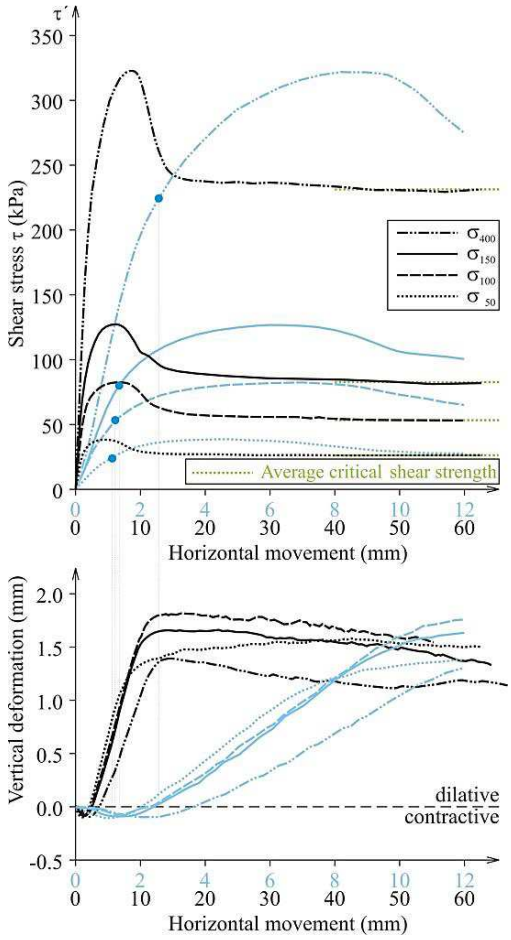


Fig. 9: The results of the shear strength tests for Sample 7.

In the case of gravel with sharp edged grains, the interlocking effect is very significant, and the shear strength is defined not only by the angle of the shear strength but also by the initial shear strength. Samples 8 and 9 were made of mechanically crushed grains, which had sharp-edges. The results of the shear strength test of Samples 8 and 9 are presented in Figs. 10 and 11, respectively. In both cases the tests were executed for normal stresses up to 400 kPa. The results showed a significant decrease in dilatancy with increases in the normal stress.

In the case of Sample 8, Fig. 10, the e_{crit} value was reached for horizontal movement exceeding about 50 mm for normal stresses 50, 100, and 150 kPa. For a normal stress of 400 kPa, the e_{crit} was not reached; however, it can be estimated from the results of the vertical deformation that the value can be reached immediately after 60 mm. For this reason, the stable shear strength was not exactly reached, and the value for the maximum horizontal movement was assumed to be the shear strength at point C. In the case of Sample 9, Fig. 11, the value of e_{crit} was reached close before the horizontal movement of 60 mm for a normal stress of 400 kPa. The results of the shear strength test Sample 9 are similar to the results of Sample 8.

The results of the study showed that the ratio between the shear strength at points C to A (τ_c / τ_a) increases with increases in the grain size. The relationship between the ratio τ_c / τ_a and the mean grain size d_{50} is shown in Fig. 12. The most precise interpretation of the results is achieved when the d_{50} is plotted on a logarithmic scale and the exponential trend-line is used. The relationships between τ_c / τ_a and d_{50} are separately interpreted for each normal stress, i.e. 50, 100, and 150 kPa. The lowest value of the coefficient of the determination was equal to 0.959. The graph allows for simply estimating the difference between the shear strength determined at points A and C for compacted coarse-grained materials of similar grain size distributions tested in a large-size direct shear test apparatus.

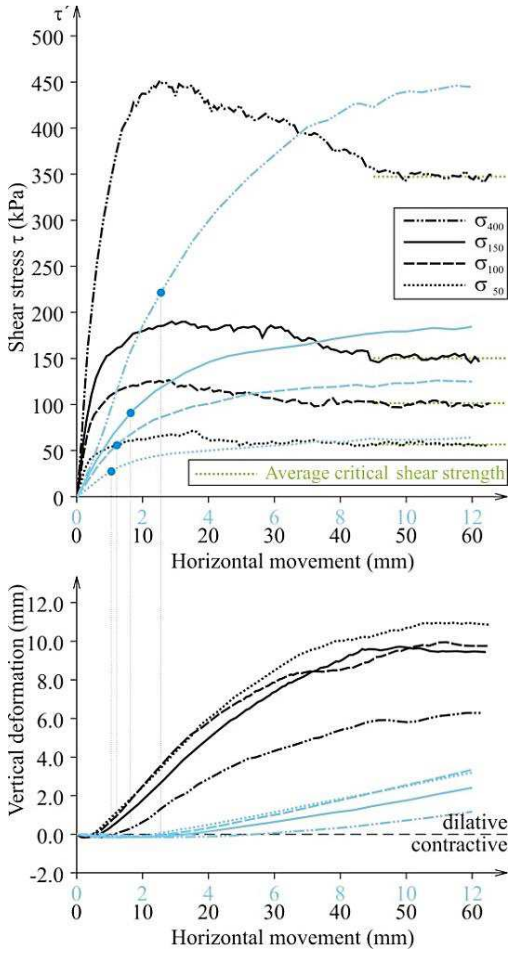


Fig. 10: The results of the shear strength tests for Sample 8.

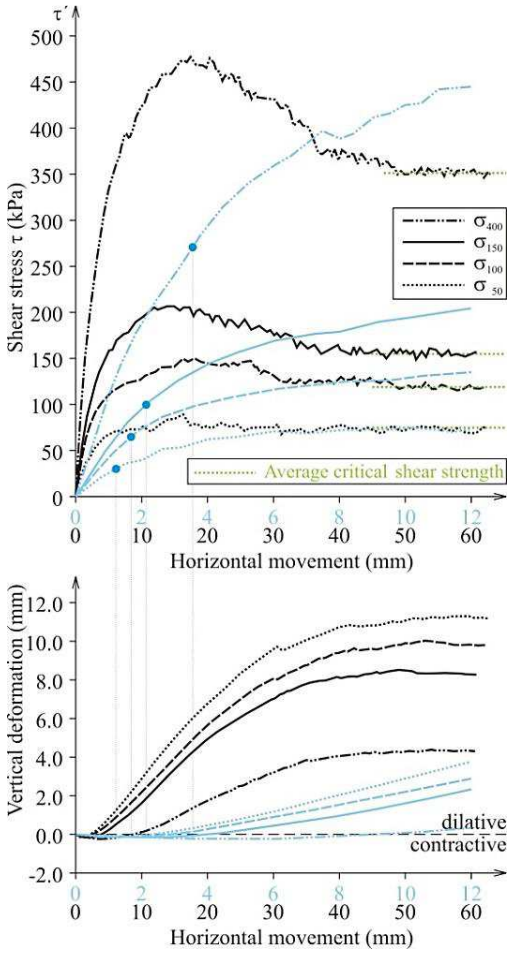


Fig. 11: The results of the shear strength tests for Sample 9.

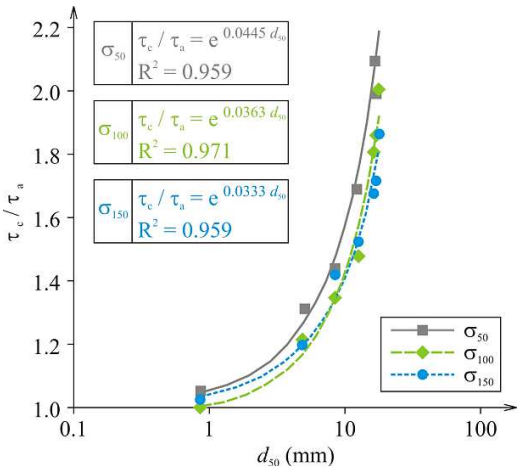


Fig. 12: The relationship between τ_c/τ_a ratio and d_{50}.

5.3 Evaluation of shear strength parameters

The evaluation of the results of the large-size direct shear tests is summarized in Table 2. The shear strength properties, i.e. the angle of shear strength and the initial shear strength, were determined at points A, P (peak shear strength), and C. The mean diameter of the grains as well as the density index at the initial state are given.

Table 2: Measured shear strength parameters of the soils tested.

Sample	Soil symbol	Diameter of grain d_{50} [mm]	Density Index I_D	Parameters of shear strength						
				Point A		Point P			Point C	
				φ' [°]	τ_0' [kPa]	φ' [°]	τ_0' [kPa]	Ψ' [°]	φ' [°]	τ_0' [kPa]
Sample 1	GP	17.80	0.493	26.2	0	49.9	0	15.75	43.8	0
Sample 2	GP	13.00	0.533	28.6	0	51.8	0	15.9	44.0	0
Sample 3	GW	8.45	0.821	31.9	0.5	49.0	4.7	14.7	42.2	0.8
Sample 4	GW	5.13	0.876	28.1	0	48.5	0.4	14.2	38.4	0.6
Sample 5	GP	4.90	0.975	28.1	0	42.2	1.0	13.8	31.8	2.9
Sample 6	GP	12.72	0.716	31.7	0	51.9	0	16.96	43.1	1.1
Sample 7	SP	0.86	0.984	29.0	0	41.1	2.9	14.8	28.8	2.1
Sample 8	GP	16.37	0.921	32.2	0.8	50.1	19.0	16.1	43.2	18.9
Sample 9	GP	17.05	0.869	34.7	1.7	50.4	37.1	15.6	41.2	38.6

Because all the samples tested were dilatante, the maximum value of the angle of shear strength was reached at the peak state. The results also showed that in the cases of gravel samples, the angle of shear strength at point C was higher than at point A. The shear strengths at points C and A were equal to each other only in the case of the sand sample. Samples 8 and 9 consisted of sharp-edged grains, which led to determining of their initial shear strength as significant.

As stated by, e.g. [23, 24], coarse-grained materials, especially poorly graded gravels, have a nonlinear failure line. The authors have already presented an evaluation of the results of the shear test of Sample 8 [9] using 4 different methods:

- a single linear failure line translated over all the measuring points,
- multilinear failure line - the slope of the failure line changes, i.e. it connects the measured points,
- secant lines - connecting 0 and the relevant measuring point,
- nonlinear failure line defined by a power function.

They recommended using a multilinear failure line or a nonlinear failure line defined by a power function to obtain the most reliable shear strength properties of poorly graded gravels. The evaluation of the shear strength of Samples 8 and 9 is shown in Figs. 13 and 14, respectively. Two methods of evaluation were applied, i.e. a multilinear (bilinear) failure line (plotted by black lines), and a nonlinear failure line (plotted by a coloured curve). The multilinear failure line was defined by two lines within the range of normal stresses of 0 - 150 kPa (represented by parameters φ' and τ_0') and 150 - 400 kPa (represented by parameter φ'). The nonlinear failure line was defined by a power function; the expression of the function is given for $\mathrm{tg}\varphi'$.

The results showed that for both samples, the shear strength at point C is higher than at point A. The shear strength at the point C and the peak shear strength have approximately the same value of the τ_0', which is caused by the interlocking effect that persists after its mobilization in the case of poorly graded gravel.

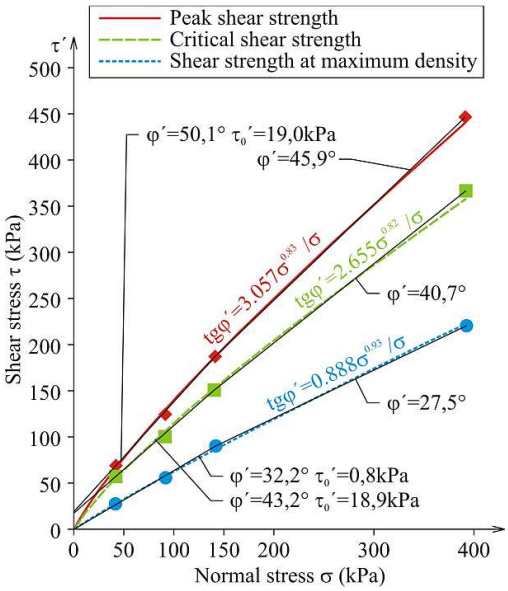


Fig. 13: The shear strength envelopes for Sample 8.

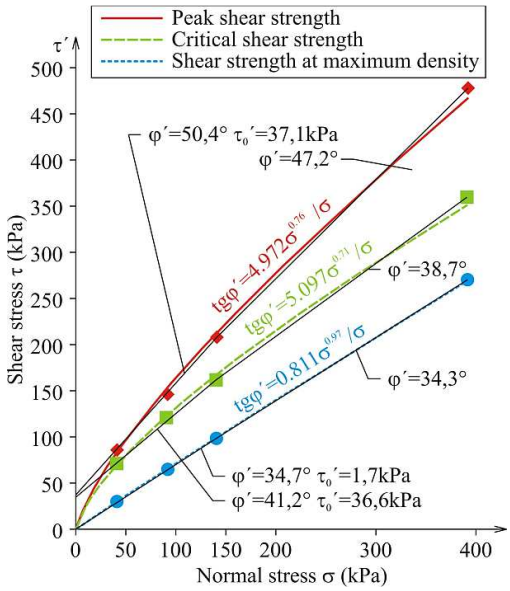


Fig. 14: The shear strength envelopes for Sample 9.

The evaluation of the shear strength of Sample 7 is shown in Fig. 15. In the case of the sand sample, the results showed that the shear strengths at points C and A are practically equal to each other. The nonlinear shear strength was determined only in the case of the peak shear strength. The results confirmed that in the case of the sandy soil, the critical shear strength can be determined at point A as well as at point C.

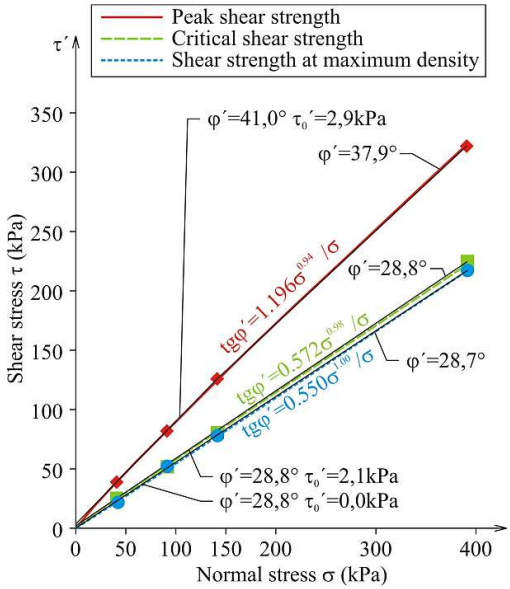


Fig. 15: The shear strength envelopes for Sample 7.

6 Conclusion

The shear strength of coarse-grained materials can be successfully determined using a large-size direct shear test apparatus. The first part of the study presented is focused on determining the peak and critical shear strength. The peak shear strength is determined for the maximum shear stress measured (shear strength at point P). The critical shear strength can be determined in two points, i.e., the shear strength at point A, when the maximum density of soil tested is reached, and the shear strength at point C, when the critical void ratio is reached at a large displacement in the shear area. The results of the study presented show that there are no differences between the shear strength at points A and C in the case of sandy soils. The differences between these shear strengths were measured in the case of the gravels, especially in the poorly graded gravels, and, as the results

showed, it increased with increases in the grain size. This difference was mostly given by an interlocking effect that particularly occurs in the case of poorly graded gravels. The relationship between the ratio of the shear stress at points C to A (τ_c / τ_a) and the mean grain size d_{50} was given by the exponential function in the case of the logarithmic scale. The value of the ratio was within a range of 1.0 for the sand up to 2.0 for the medium poorly graded gravel. Especially in evaluation of gravel samples, the critical shear strength should be determined only at point C, i.e. for a relatively large horizontal movement, when the peak shear strength is exceeded, and applies that the critical void ratio and the shear stress reached a stable value. In the case of the sandy soil, the failure line of the critical shear strength is linear; however, the failure line of the peak shear strength is nonlinear. In the case of gravels, especially poorly graded gravel, the failure line is nonlinear for the peak as well as the critical shear strength. The greatest nonlinearity can be visible in the case of the poorly graded gravels that had sharp-edged grains. The evaluation of the shear strength can be successfully done using a multilinear failure line, depending on the size and range of the normal stresses or using a nonlinear failure line defined by a power function.

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