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Reconstruction of Holocene marine sand natural hydrostatic pressure and its relation with shearing strength

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Abstract. This research paper represents interpretation of engineering geological and geotechnical conditions by using a direct shear device with a possibility to apply hydrostatic pressure during the experimental testing. In the authors' opinion, the evaluation of in-situ hydrostatic pressure and reconstruction of it in the laboratory tests is a very important step in the determination of the engineering geological and geotechnical properties of the sand strength and its behaviour. To understand the influence of hydrostatic pressure on shearing strength results, there were performed direct shear tests with and without hydrostatic pressure. Obtained results clearly show the differences in the shearing strength analysed during this study. When the investigated sand sample was loaded with maximum hydrostatic pressure (σ_H = 100 kPa), the peak value of the angle of internal friction (ϕ'_{mean}) was 21.24% higher compared to that of dry soil. No tendencies were found for cohesion.

Keywords: *sand; direct shear test; saturated sand; hydraulic head; hydrostatic pressure; shearing strength*

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INTRODUCTION

Thirty percent of Earth's surface in Lithuania is covered by sand (Žaržojus 2006). The most of these sand deposits were formed in the Pleistocene epoch during the ice-age and interglacial epoch. During the Holocene epoch sandy deposits occur locally. Their genesis is divers: alluvium (aIV) deposits were left by flowing floodwater in a river valley or delta, eolian deposits (eIV) are the continental and coastal dunes formed by the wind, marine deposits (mIV) lie on the shore of the Baltic sea. We can assign the anthropogenic-technogenic deposits (tIV) to the contemporary deposits, which are formed by a human activity and are considered the man-made deposits. In the urban areas these deposits are predominant and the depth of occurrence is up to 10 m and more. Therefore, the Holocene epoch sandy deposits are very important as the base of foundation of the buildings. In the civil engineering it is very important to estimate the physical and mechanical properties of these sandy deposits, their behaviour under a load, in order to predict the possible changes of the properties of the aforementioned soils in the future. The differences between the properties of sandy deposits of different origins depend on their genesis. Especially, the properties of alluvium, eolian and marine sandy deposits are highly dependent on their grain size distribution, solid particles morphometric parameters, and the initial porosity (Prušinskienė 2012).

Partially saturated soils bring about the fairly noticeable compressibility, and hence affect the mechanical behaviour of soil (Kamata *et al.* 2009). The effect of water exists, especially in the depths where the hydrostatic pressure is big enough. The tests result for the saturated sand samples without and with the restored initial hydrostatic pressure may be different. Despite the theoretical solution, the studies have been carried out to clarify these differences. During the studies the direct shear tests around the sample and inside it were carried out having a possibility to load the soil with hydrostatic pressure. This feature ensures the reconstruction of the initial conditions of the sand samples, i.e. the shear test starts only after the hydrostatic pressure reconstruction (it is a simulation of hydraulic head). It is a very important step. Due to the difficulty of taking out the samples of saturated sand and a relatively high investigation price, the results estimated during the direct shear test with capacity to reconstruct the hydrostatic pressure are very significant, because the values of initial hydrostatic pressure are reconstructed and the natural conditions of sand occurrence are simulated. Investigating the soil with direct shear tests is much faster than with the triaxle tests, but the differences between the results of these two tests are negligible (Medzvieckas *et al.* 2017).

When the direct shear test is performed in accordance with ASTM guidelines, the measured shear stresses at failure estimate drained strength parameters. Investigators (Bro *et al.* 2013) researched the possibility of estimating undrained strength using direct shear testing at variable shear displacement rates on specimens composed of various combinations of kaolinite and bentonite. These investigators recognize the inability to control drainage during the direct shear test, but hypothesize direct shear tests are run on soils of low hydraulic conductivity at sufficiently fast shear displacement rates, saturated specimens can be sheared to failure without a significant volume change. They determined that direct shear tests run at fast shear displacement rates can, at best, only provide an approximation of undrained strength (Bro *et al.* 2013).

Some investigators (Thian, Lee 2018) researched the shear parameter differences for an offshore clay tested under simplified constant volume direct simple shear (CDSS) and conventional non-constant volume direct simple shear (DSS) conditions. The change in vertical stress which occurs during shearing is assumed to be the equivalent pore water pressure in a correct undrained test. The shear strength of clay specimen is overestimated when it is tested under the conventional DSS device as compared to that under the CDSS device (Thian, Lee 2018).

Other authors (Tika *et al.* 1996) designed a ring shear box which was surrounded by a water bath and they investigated the influence of the water in the bath. They determined that residual strength at a fast shear rate without water in the bath was higher than with water.

Other researchers (Antonio, Carraro 2017) developed a more convenient and rational method to interpret results from simple shear tests with cell pressure confinement, which can reduce costs and improve reliability of offshore infrastructure. The paper describes typical boundary conditions imposed in such tests and outlines an alternative procedure to properly account for the effect of evolving cylindrical specimen geometry and intermediate principal stress on the analysis of test results (Antonio, Carraro 2017).

The aim of this article is to show the differences between strength properties of the Holocene epoch sand deposits which were estimated having a reconstructed from natural conditions hydrostatic pressure and without it. To ensure the accuracy of the tests results, the clean sand with particles of uniform shape was used.

Description of investigated sand

The Holocene sands, due to their specific formation, are sufficiently weak and very compressible deposits; therefore, particular attention should be paid to an estimation of the strength of these soils. Generally, the sand strength is described by an angle of internal friction (ϕ), while cohesion (c, kPa) is negligible, especially for the Holocene sands. Due to specific origins of the Holocene sands, these occur under a groundwater table or are submerged by surface water (rivers, lakes, or sea). It is difficult to take to a laboratory the undisturbed samples of sand for the tests. It is not possible at all to take the saturated sand. Therefore, it is necessary to find a way to restore the natural properties of the saturated sand in the laboratory. However, it is not enough to have only the samples of saturated sand, it is necessary to restore the initial values of the natural hydrostatic pressure.

In this research the same soil as in a previous research of mechanical properties was used (Skuodis *et al.* 2014). The investigated area is located in the southern part of the Lithuanian mainland area of the Baltic Sea (sampling depth 0.4–0.5 meters, coordinates of sampling location 55°46'4,07", 21°4'39,06" (WGS)), where the immediate nearshore area contains a sandy strip of Holocene marine sediments (m IV) (Skuodis *et al.* 2014). Northwards from the Klaipėda city, only the immediate nearshore area contains a sandy strip of Holocene marine sediments (m IV), which occur as deep as to 4–5 meters in the sea (Bitinas *et al.* 2005; Viška, Soomere 2013). The material composing the nearshore sediments (m IV) mainly consists of different sand (Dundulis *et al.* 2006). Mineralogical composition of investigated sand consists basically of dominating ingredients, namely, of ~ 85% silica and ~ 6% sunstone with remaining contribution of carbonate, mica and some other minerals (Amšiejus *et al.* 2010). Solid density of determined investigated sand particles ρ_s = 2,65 g/cm³. Also, it was found that the sand grains are round-shaped, the grain surface is smooth and abrasive. The mean shape parameters (Table 1) of investigated sand grains were determined using Krumbein and Slos (1951) and Cho *et al.* (2006)

solutions for sand shape characterization according to the particle sphericity and roundness.

According to granulometric composition of investigated sand (Fig. 1), we determined soil name – sand (Sa), where uniformity coefficient C_U = 1.47, coefficient of curvature C_C = 0.93 (even-graded sand). Determination of uniformity coefficient and curvature coefficient is explained by Mahinroosta and Poorjafar (2017).

According to the above-mentioned investigated sand parameters (granulometry composition and particles shape), the Baltic Sea Klaipėda sand can be called the Lithuanian standard sand for scientific testing (Ojuri, Agbolade 2015; Skuodis *et al.* 2017). This is the reason why Klaipėda Holocene marine sand was chosen for direct shear tests with and without hydrostatic pressure.

Table 1 Mean morphological parameters (2D case)

Morphological parameter	Mean value
Area (mm^2)	0.1122
Equivalent diameter (mm)	0.340
Sphericity	0.836
Circularity	0.515
Form coefficient	0.702
Angularity	0.410
Particle shape	

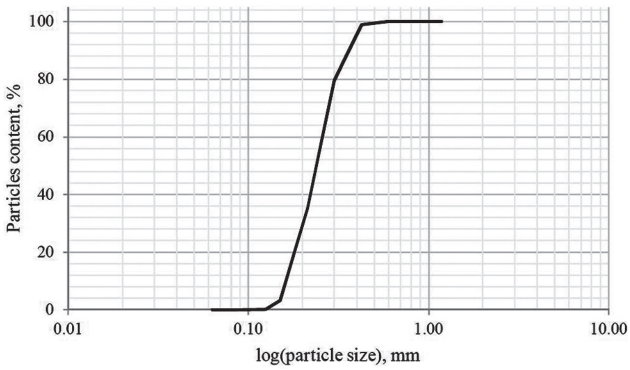

Fig. 1 Granulometry composition of investigated Holocene marine sand

Testing procedure

Experimental investigations were provided with direct shear device LO3550 with cell (Fig. 2). We applied 0.5 mm/min shearing velocity for sand specimens up to not less than total 8 mm horizontal displacement u_h. Before the shearing procedure, there was established a 1 mm gap between shearing rings. The shearing gap during the testing procedure was constant due to rigid fixations. All specimens were loaded with normal vertical stress σ_v (10 or 100 kPa). First of all, tests with air dry sand specimens were provided. According to these results, water influence on shearing strength was checked. Water influence was imitated fully saturating sand samples and establishing hydrostatic water pressure σ_H = 0, 30 and 100 kPa. Chosen hydrostatic water pressure values imitating soil under the water behaviour were in the depths of 0, 3 and 10 meters, respectively. The samples saturation process is very slow and takes around 30 min to fully fill the cell (water inlet is from the cell bottom) and to saturate the sand sample. According to slow saturation it is accepted that all the pores in the sand sample are fully saturated. During all tests there was applied the constant vertical normal stress, and for samples which were tested for water pressure influence, the applied hydrostatic pressure also was constant.

All soil samples investigated were of the same height (30 mm), shearing box shape was square, box length and width were 100 mm. Investigated samples were prepared with medium density (1.671 g/cm^3).

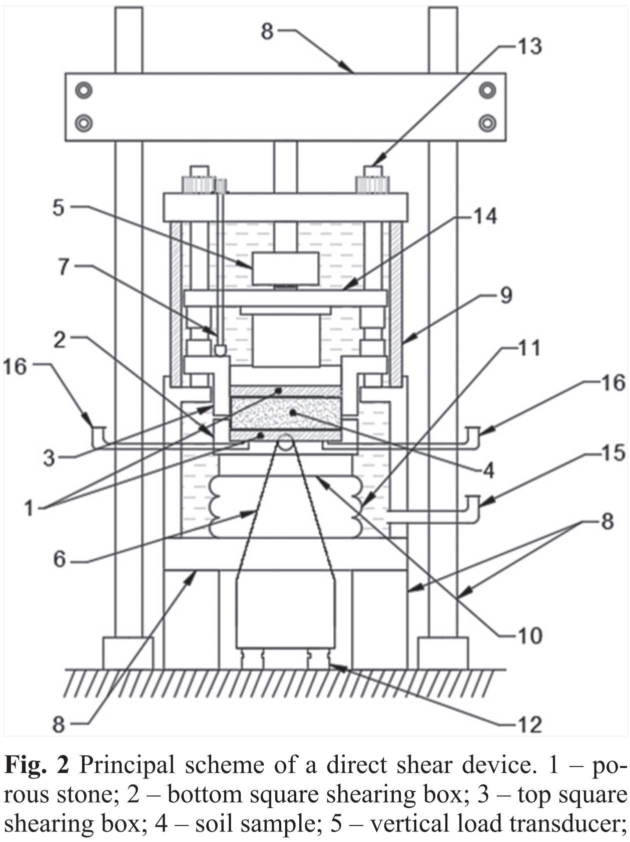

Fig. 2 Principal scheme of a direct shear device. 1 – porous stone; 2 – bottom square shearing box; 3 – top square shearing box; 4 – soil sample; 5 – vertical load transducer; 6 – horizontal load transducer; 7 – bolt for shearing gap establishment; 8 – rigid frame element; 9 – water cell; 10 – bottom horizontal plate; 11 – uniaxial connecting joint; 12 – horizontal track; 13 – bolts for water cell tightening; 14 – load directing element; 15 – water inlet channel to water cell; 16 – water inlet channel to sample

OBTAINED RESULTS

The direct shear strength results obtained for air dry and fully saturated sand samples which were affected with hydrostatic pressures (σ_H = 0, 30 and 100 kPa) were not the same. The results were different when peak $\tau_{f(p)}$ and residual $\tau_{f(r)}$ values of shear-

Table 2 Shearing strength of investigated sand with different testing conditions

σ_H, kPa	$\sigma_{v(p)}$, kPa	$\tau_{f(p)}$, kPa	$u_{h(p)}$, mm	$\sigma_{v(r)}$, kPa	$\tau_{f(r)}$, kPa	$u_{h(r)}$, mm	$\phi_{mean(p)}$, °	$c_{mean(p)}$, kPa	$\phi_{mean(r)}$, °	$c_{mean(r)}$, kPa
Air dry	10.06	10.08	3.83	10.10	7.52	8.15	25.81	5.43	19.03	4.49
Air dry	9.99	10.68	2.73	10.08	8.50	6.52				
Air dry	10.28	10.25	3.12	10.34	8.39	5.44				
Air dry	10.49	10.15	2.37	10.52	8.15	4.95				
Air dry	9.89	10.50	3.08	9.97	7.45	8.29				
Air dry	99.99	50.14	3.61	100.02	40.21	8.26				
Air dry	99.95	50.48	3.78	100.21	36.68	8.55				
Air dry	100.05	54.03	3.60	100.13	41.23	8.41				
Air dry	99.93	55.07	3.71	100.03	42.83	8.56				
Air dry	100.00	59.24	4.07	100.40	34.23	8.56				
0	8.72	10.25	2.96	8.83	8.24	8.76	24.05	6.66	19.89	4.49
0	10.34	12.33	2.44	10.68	9.82	5.84				
0	7.43	9.25	8.04	8.32	9.35	4.43				
0	99.84	49.76	4.43	100.07	39.56	8.53				
0	99.73	54.03	4.39	100.69	46.43	8.42				
0	99.69	49.71	3.66	101.05	40.52	8.25				
30	99.05	68.38	3.70	99.57	50.44	5.71	29.92	3.30	22.49	3.44
30	99.68	77.03	3.23	101.84	51.55	8.40				
30	100.00	55.90	2.89	100.56	41.24	8.52				
30	99.79	47.12	3.22	100.07	38.58	8.53				
30	99.68	64.23	4.18	99.21	48.49	8.28				
30	99.93	51.36	2.95	100.33	39.43	8.52				
30	9.86	6.13	2.29	10.62	3.97	6.45				
30	9.70	8.55	8.34	9.87	7.63	5.02				
30	11.01	11.59	2.25	11.16	10.36	4.34				
30	9.85	10.08	3.27	10.09	9.10	6.49				
100	99.92	54.84	3.30	99.09	43.23	8.31	32.77	5.88	26.85	5.17
100	100.04	58.65	2.74	101.61	44.17	8.27				
100	99.12	85.31	4.49	100.00	72.01	8.34				
100	99.38	84.56	3.65	101.12	64.11	8.30				
100	99.87	67.07	3.15	100.05	56.41	8.05				
100	10.01	12.29	3.51	10.04	9.84	8.68				
100	10.03	12.79	2.79	10.28	10.88	5.45				
100	10.23	12.70	3.54	10.20	10.51	8.71				
100	10.04	11.46	2.60	9.99	9.96	6.46				

Here:
σ_H – the hydrostatic pressure;
$\sigma_{v(p)}$ – the vertical normal stress;
$\tau_{f(p)}$ – the peak value of the shearing strength;
$u_{h(p)}$ – the horizontal displacement at the peak shearing strength;
$\sigma_{v(r)}$ – the vertical normal stress;
$\tau_{f(r)}$ – the residual value of the shearing strength;
$u_{h(r)}$ – the horizontal displacement at the residual shearing strength;
$\phi_{mean(p)}$ – the mean value of the peak angle of internal friction;
$c_{mean(p)}$ – the mean value of the peak cohesion;
$\phi_{mean(r)}$ – the mean value of the residual angle of internal friction;
$c_{mean(r)}$ – the mean value of the residual cohesion.

ing strength were compared (Table 2). Peak shearing strength was obtained by the maximum ratio of tangential and normal stresses. Residual shearing strength was obtained by the minimum ratio of tangential and normal stresses.

Analysis of the obtained results in Table 2 shows that the determined mean value of the peak angle of internal friction $\phi_{mean(p)}$ for air dry sand samples was 25.81°. When the soil samples were affected with hydrostatic pressure, the higher the hydrostatic pressure, the bigger was the increment of the angle of internal friction. The highest peak value of internal friction $\phi_{mean(p)}$ obtained was 32.77° with hydrostatic pressure of 100 kPa. This value of the angle of internal friction is 21.24% higher compared with air dry sand results.

Analyzing direct shear test peak shearing strength results (see Fig. 3–6 and Table 2), it was obtained that if the determination coefficient R^2 decreases, the

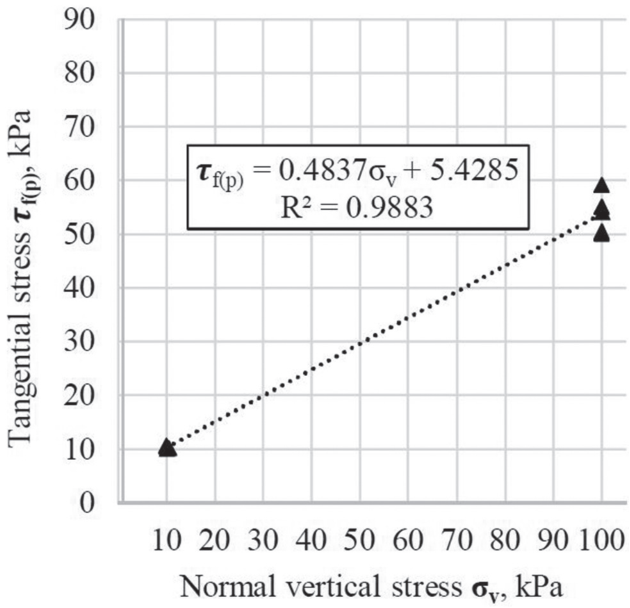

Fig. 3 Tangential stress versus normal vertical stress (air dry sand)

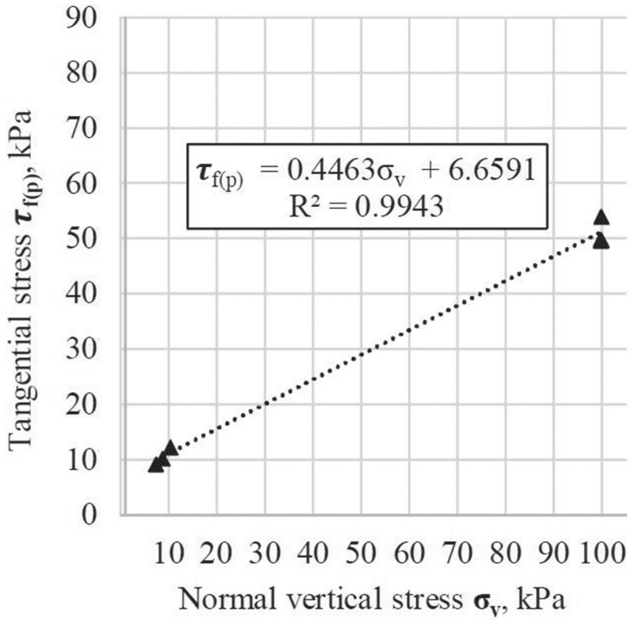

Fig. 4 Tangential stress versus normal vertical stress (fully saturated sand with hydrostatic pressure $\sigma_H = 0$ kPa)

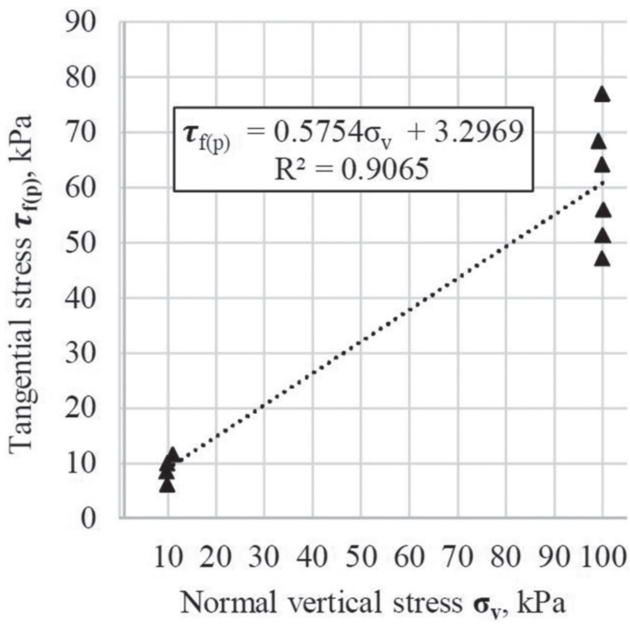

Fig. 5 Tangential stress versus normal vertical stress (fully saturated sand with hydrostatic pressure $\sigma_H = 30$ kPa)

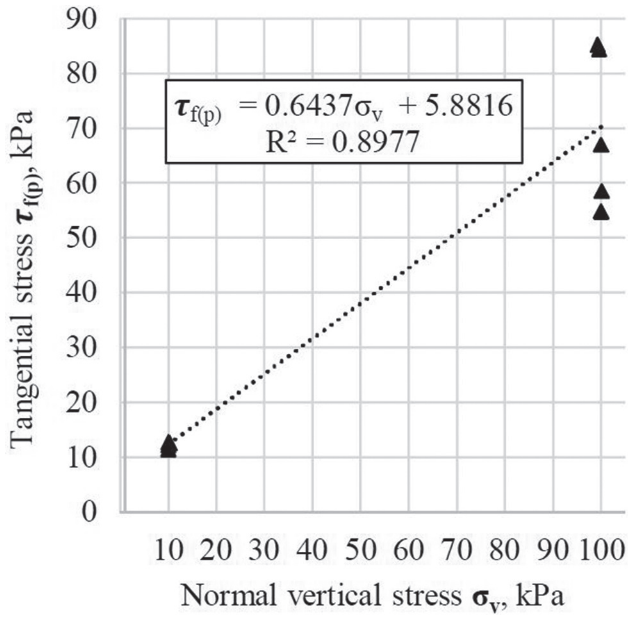

Fig. 6 Tangential stress versus normal vertical stress (fully saturated sand with hydrostatic pressure $\sigma_H = 100$ kPa)

angle of internal friction increases. Also, the determination coefficient R^2 depends on hydrostatic pressure applied during testing. When hydrostatic pressure σ_H decreases, the determination coefficient R^2 decreases. This tendency has an influence on the angle of internal friction. For air dry sand samples and for samples with hydrostatic pressure $\sigma_H = 0$ and 30 kPa, the angle of internal friction has a very small increment. But when hydrostatic pressure $\sigma_H = 100$ kPa is applied, the angle of internal friction increment is obvious. There were no tendencies obtained for cohesion, only for the angle of internal friction.

After the analysis of peak shearing strength results, the analysis of residual shearing strength results was provided (see Fig. 7–10 and Table 2). The same tendencies were obtained as in the analysis of peak shearing strength. If the determination coefficient R^2 decreased, the angle of internal friction increased. Also, the determination coefficient R^2 had a direct dependence on the hydrostatic pressure applied during testing. For residual shearing strength, the same tendencies as for peak shearing strength were valid. All analyses of peak and residual shearing strengths show that the applied hydrostatic pressure during the test

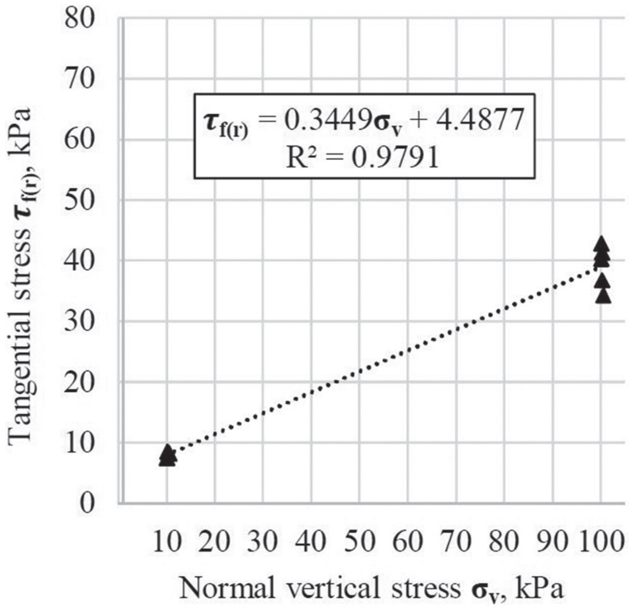

Fig. 7 Residual tangential stress versus normal vertical stress (air dry sand)

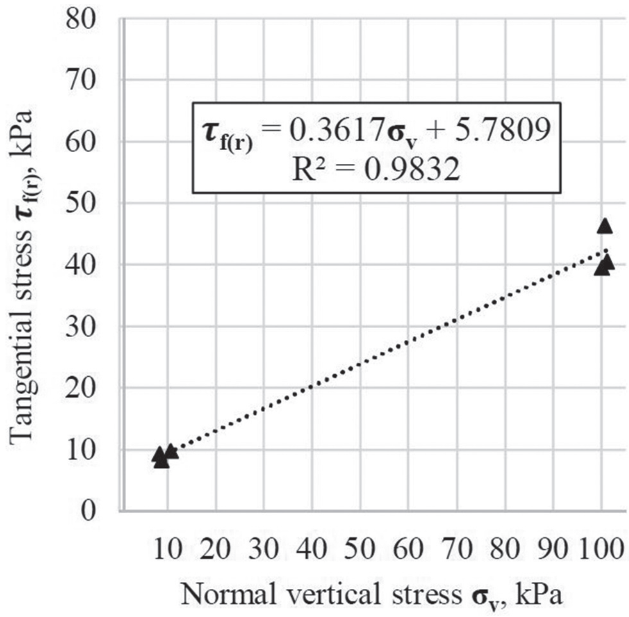

Fig. 8 Residual tangential stress versus normal vertical stress (fully saturated sand with hydrostatic pressure σ_H = 0 kPa)

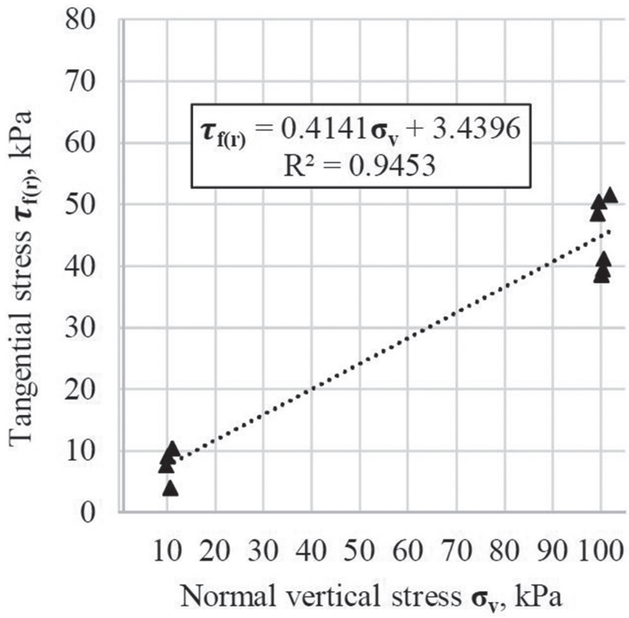

Fig. 9 Residual tangential stress versus normal vertical stress (fully saturated sand with hydrostatic pressure σ_H = 30 kPa)

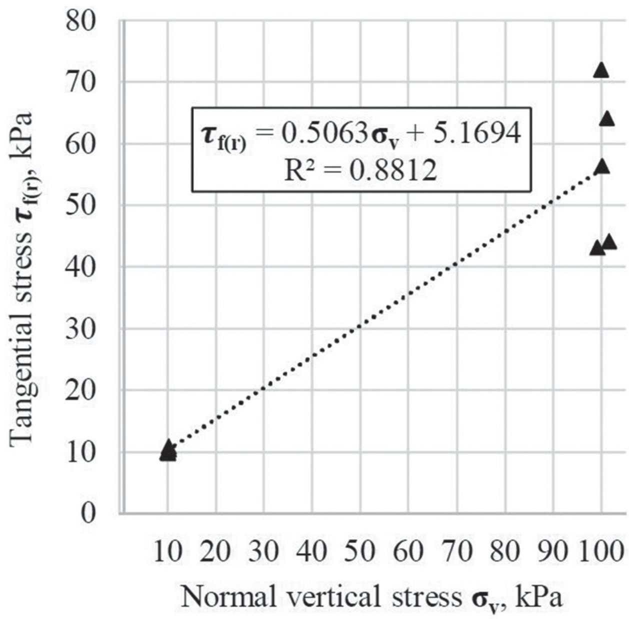

Fig. 10 Residual tangential stress versus normal vertical stress (fully saturated sand with hydrostatic pressure σ_H = 100 kPa)

had influence only on the angle of internal friction increment. No tendencies were found for cohesion.

According to obtained results, which are presented in Figures 3–10, it is obviously seen that when the hydrostatic pressure increases, the same happens with the angle of internal friction. This phenomenon of the angle of internal friction strengthening can be related with the testing procedure. When a prepared soil sample is in the direct shear device and the vertical porous stone is in contact with the soil sample, the density of the sample is not changing. The next procedure step is to saturate the soil sample. When water starts to flow into the device of direct shear in order to saturate the sample, due to the water which penetrates into the soil, the soil particles compact on their own (Han, Vanapalli 2016; Grujicic *et al.* 2008; Heitor *et al.* 2013). In this research the sample height change during the sample saturation procedure was not measured (Vogel *et al.* 2000; Sakaki, Illangasekare 2007; Ajayi *et al.* 2016).

CONCLUSIONS

The direct shear test performed with and without hydrostatic pressure represents the differences in the results of the study. When the investigated sand sample was loaded with maximum hydrostatic pressure (σ_H = 100 kPa), the peak value of the angle of internal friction (ϕ'_{mean}) was 21.24% higher compared to that of dry soil. No tendencies were found for cohesion. Confined hydrostatic pressure may cause reorientation of sand particles into a denser structure (more contacts between particles). The density of a sand sample during the saturation process may increase, and due to this reason the increment of the angle of internal friction is obtained. In the authors' opinion, the evaluation of in-situ hydrostatic pressure and reconstruction of it in the laboratory tests is a very important step in the determination of the engineering geological properties of sand strength and its behaviour.

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