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BEHAVIOUR OF COHESIVE SOIL SUBJECTED TO LOW-FREQUENCY CYCLIC LOADING IN STRAIN-CONTROLLED TESTS

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Abstract: The subject of the paper comprises tests of cohesive soil subjected to low-frequency cyclic loading with constant strain amplitude. The main aim of the research is to define a failure criteria for cohesive soils subjected to this type of load. Tests of undrained cyclic shear were carried out in a triaxial apparatus on normally consolidated reworked soil samples made of kaolinite clay from Tułowice. Analysis of the results includes the influence of number of load cycles on the course of effective stress paths, development of excess pore water pressure and stress deviator value. Observed regularities may seem surprising. The effective stress path initially moves away from the boundary surface and only after a certain number of load-unload cycles change of its direction occurs and it starts to move consequently towards the surface. At the same time, it has been observed that pore water pressure value decreases at the beginning and after few hundred cycles increases again. It is a typical behaviour for overconsolidated soil, while test samples are normally consolidated. Additionally, a similar change in deviator stress value has been observed – at first it decreases and later, with subsequent cycles, re-increases.

Key words: *cohesive soils, triaxial tests, cyclic loading, strain-controlled tests*

NOMENCLATURE

A_s	–	amplitude (stress amplitude), kPa
A_ε	–	amplitude (strain amplitude), %, –
B	–	Skempton equation parameter, –
D, H	–	sample diameter, sample height, cm
f	–	frequency, Hz
I_p	–	plasticity index, %
L_L	–	liquid limit, %
N	–	number of cycles, –
p'	–	mean effective stress, kPa
q, q_c	–	stress deviator, cyclic stress deviator, kPa
S_u	–	shear strength (monotonic load, undrained conditions), kPa
u, Δu	–	pore water pressure, excess pore water pressure, kPa
u_N*	–	normalized cyclic pore water pressure, kPa
v	–	velocity, mm/h
w_n	–	natural water content, %
$\gamma, \gamma, \gamma'_c$	–	shear strain, cyclic (effective) shear strain, %, –
γ_{tv}	–	the volumetric threshold cyclic shear strain, %, –
ε_1, ε_a	–	axial strain, %, –
$\varepsilon_{1,\mathrm{unload}}$	–	axial strain initiating the cyclic load operation, %, –
ε_s, $\varepsilon_{s,c}$	–	shear strain, cyclic shear strain, %, –
θ	–	moisture content, %
σ'_c, σ'_{vc}	–	initial effective stress, initial vertical effective stress, kPa
σ'_h, σ'_v	–	horizontal, vertical effective stress, kPa
σ'_p	–	maximum preconsolidation pressure in the stress history of a soil, kPa
σ_r	–	cyclic shear stress, kPa
τ, τ_c	–	shear stress, cyclic shear stress, kPa
τ_f	–	shear strength (monotonic load, undrained conditions), kPa
CIU	–	testing with isotropic consolidation and shear without drainage
CSL	–	critical state line
CSR	–	cyclic stress ratio
OCR	–	overconsolidation ratio

1. INTRODUCTION

Analysis of the influence of cyclic loading on soil is a common geotechnical issue, due to numerous loads of this type occurring in nature. Cyclic loading is a kind of influence in which alternating cycles of load–unload occur. It means that during cyclic loading there are numerous changes in stress path direction of 180 degrees. This type of load can be generated both by forces of nature or different types of machines. It is important to correctly classify and identify the nature of the type of load and then accurately reproduce it in the laboratory.

The influence of cyclic loading on cohesive soils has not been yet comprehensively studied and described in scientific publications (especially in the range of small strains). There are definitely more publications concerning cohesionless soils, which are far more recognized. Test results gathered so far on cohesive soils point out different, more complex behaviour of this kind of soil subjected to cyclic loadings (Jastrzębska [12]).

Therefore, the object of presented tests is cohesive soil subjected to cyclic loading with small strain amplitude (A_ε = 0.02%). The specificity of cohesive soils allows performance of many load-unload cycles before failure, in contrast to cohesionless soils, which are much more sensitive to this kind of load and are rapidly led to liquefaction. In conducted tests it was decided to perform as many load-unload cycles as possible to observe how their number affects behaviour of cohesive soil subjected to cyclical influences. It is a very long-term process, taking into account number of cycles and their low frequency, dictated by the desire to eliminate occurrence of dynamic influences.

The aim of the paper is to present the observed phenomena associated with changes of cohesive soil behaviour during cyclic loading. The results of the research may provide a basis for development of soil models describing cohesive soil behaviour. In the future a larger number of test results will allow an attempt to define a failure criterion for cohesive soils subjected to low-frequency cyclic loading. An example of such load in nature can be filling and emptying the tank or tidal variations. Tests are conducted on very soft soil for which no shallow foundation can be applied. This is why triaxial compression tests are performed, instead of oedometric ones.

2. BEHAVIOUR OF COHESIVE SOIL SUBJECTED TO CYCLIC LOADING

The term of cyclic loading is very general and it has many definitions available in the literature. Their common denominator are for sure multiply repeated load–unload cycles, which in the case of soil may result in their very different behaviour. An extremely important factor is the location of the beginning of cyclic process in a "stress–strain" relationship. If the process starts with loading, then it is situated in the beginning of coordinate system (q, ε_s) = {0, 0} (Fig. 1, Fig. 2). The situation is different if the cyclic process is preceded by monotonic trajectory of primary load and later starts with unloading (decrease in stress intensity).

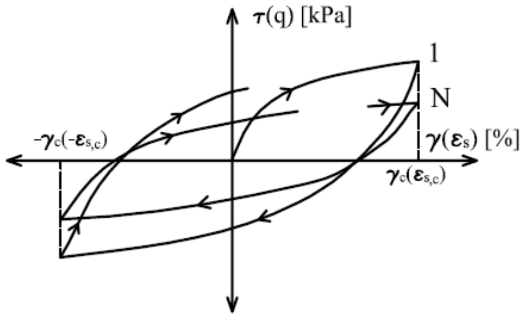

Fig. 1. An example of test with controlled strain state

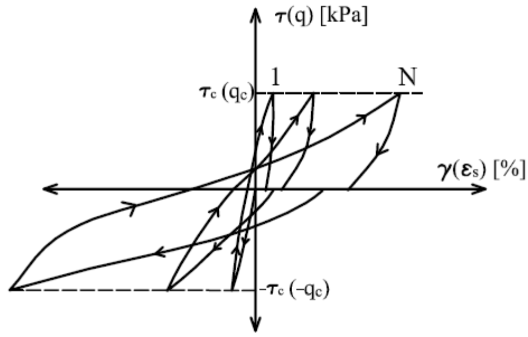

Fig. 2. An example of test with controlled stress state

In studies of cyclic processes there can be distinguished, respectively, as in the case of monotonic loads, strain-controlled tests (Fig. 1) and stress-controlled tests (Fig. 2).

To date, most studies on the behaviour of soil under cyclic loads are dedicated to the sands. First results related to cohesive soils (Drammen clay) appeared in a comprehensive report NGI (1975), which has since become one of the primary sources of information concerning the influence of cyclic loading on the behaviour of cohesive soils, in both experimental and numerical area (i.e., Andersen [2]; Andersen and Lauritzsen [3]; Sawicki [22]). Another excellent study is state of the art report by Wood [27], for laboratory tests of soils subjected to cyclic loadings of relatively high and variable amplitude. In this report he presented *inter alia* 30 publications dedicated to cohesive soils behaviour. Another review is the work of Sagaseta et al. [19] that summarizes 31 publications discussing issues of cyclic loading in relation to engineering issues and laboratory and field tests. The entirety refers to various sources causing variable loads, such as: wind, industrial machinery, piles, anchors, filling and emptying tanks. In view of the different nature of the cyclic load, its frequency is in a broad range: from a few cycles per second to one or two cycles per month. Cited publications contain both load conditions with and without drainage and refer to cohesive soils, as well as cohesionless soils. In later years, there are new works *inter alia* Hyodo et al. [9], Zergoun and Vaid [29] oraz Houlsby and Burd [8]. It seems that the focus of researchers studying cyclic processes today are boundary issues in undrained conditions, especially liquefaction of cohesionless soils as a result of water pore pressure accumulation. There are also attempts to explain the influence on cohesive soils of the mode of cyclic load (one-way[1] or two-way[2]) and other factors, such as: overconsolidation (defined as the ratio of maximum overconsolidation pressure occurring in the history of soil σ'_v to the current vertical stress acting on the soil σ'_p – $OCR = \sigma'_v/\sigma'_p$), the size of amplitude (stress or strain), number of cycles, load frequency and velocity.

For the purposes of interpretation of test results there have been introduced *inter alia* such indicators as:

- CSR (cyclic stress ratio, i.e., according to Green and Terry, [7]):

$$\mathrm{CSR} = \frac{\tau_c}{\tau_f} \quad \text{or} \quad \mathrm{CSR} = \frac{\tau_c}{S_u} \tag{1}$$

where τ_c is cyclic shear stress amplitude, τ_f or S_u – shear strength of soil determined in the process of monotonic load (in undrained conditions).

- γ_{tv} (the volumetric threshold cyclic shear strain introduced for sands by Dobry et al. [6]).

For example, in the opinion of Andersen et al. [4], Jacobsen and Ibsen [10]; Sangrey and France [21]; Sangrey et al. [20] the highest level of amplitude related to stress (CSR) at which strain stabilization occurs, which means that there is no failure of soil, for OCR = 1 is about 0.68, for OCR = 4 is 0.5, while for OCR = 10 it is just 0.42. Furthermore, the limit amplitude for one-way load corresponds to the CSR = 0.54, whereas in the same conditions for the soil under two-way load the limit is much lower and amounts to CSR = 0.4. In turn Vucetic [24]; Vucetic and Dobry [25] proved that in the case of constant strain amplitude, for soils (cohesive and cohesionless) tested in different conditions (with or without drainage) there is (strongly dependent of plasticity index I_p, or type of soil) a certain threshold value of strain amplitude (higher for cohesive soils), after which soil is sensitive to the effects of cyclic loading. Generally, γ_{tv} is between 0.01% and 0.1% and increases with increasing value of I_p.

Interesting approach to the issue was presented in the work of Yashuara et al. [28], where it was stated that in the case of normally consolidated soils subjected to cyclic loading in undrained conditions pore water pressure is generated, which causes a decrease in effective stress. Stress path is moving towards the critical state line CSL. In the case of stress-controlled cyclic load it is possible to determine the beginning of the failure in p'-q coordinate system, as shown in Fig. 3, while in the case of strain-controlled load the value of stress deviator q varies, because of which the designation of the start of failure is impossible. It should be noted that in the case of two-way load soil failure occurs sooner than for one-way load (Fig. 3b).

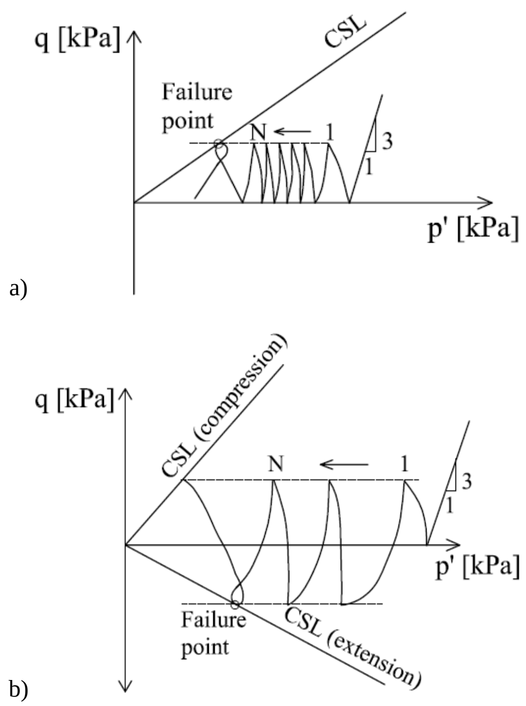

Fig. 3. Stress paths for normally consolidated soil during stress-controlled cyclic loading (a) one-way, (b) two-way

In Fig. 4, there is shown the normalized pore water pressure increase ($\Delta u/\sigma'_c$) for subsequent number of cycles N. Based on the presented results it can be seen that failure occurs at the value of pore water pressure (Δu) at level of 0.6–0.8 of initial mean effective stress (σ'_c). It is one of the features that differentiates the behaviour of cohesive soils from cohesionless soils, for which the failure (complete liquefaction) occurs when $\Delta u = \sigma'_c$. It can also be observed that with the increasing normalized stress amplitude ($\alpha = \sigma_r / 2\sigma'_c$)

[1] According to authors more accurate wording is – oscillating, meaning a kind of load in which cycles are located on both sides of monotonic stress.

[2] According to authors – pulsating, which means that cycles are realized on one side of monotonic stress.

excess pore water pressure at the moment of failure decreases.

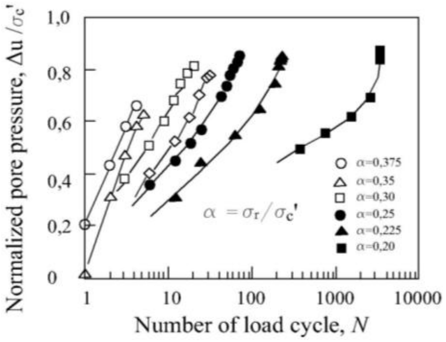

Fig. 4. Development of excess pore water pressure for normally consolidated cohesive soil during cyclic loading in undrained conditions (after Yashuara et al. [28])

Development of pore water pressure in saturated cohesive soils depends on strain (or stress) amplitude. Below a certain value after the termination of cyclic loading generated excess pore water pressure disappears (Fig. 5). In contrast, above the value in each subsequent cycle pore water pressure is generated and it persists after the termination of the load.

It is significant that in the case of normally consolidated soil the value of pore water pressure is always positive, while in the case of overconsolidated soil negative pore water pressure may occur (Fig. 5 and Fig. 6). The higher overconsolidation ratio OCR, the higher values of negative pore water pressure. In subsequent cycles pore water pressure may become positive. In the case of overconsolidated soil, generation of excess pore water pressure depends largely on load history, which makes it much more complicated than in the case of normally consolidated soil.

The influence of load velocity, or frequency, on cohesive soil behaviour is not obvious. Besides the aspect of strengthening there can be found in the literature other research, the results of which are largely inconsistent as to the influence of frequency on soil parameters (i.e., Ansal and Erken [5]). Studies of Ansal and Erken [5] show that the influence of frequency is the greater, the greater the value of CSR and thus in general the greater the amplitude. In addition, there is a certain limit value of CSR, below which there is no frequency effect. On the other hand, rapid loading causes a delay in water pore water pressure generation, which is identical to the case of monotonic load by Vucetic and Dobry [25]. Slightly different observations are contained in the publications of Matsui et al. [16] and also Procter and Khaffaf [18], where no visible effects of frequency on soil behaviour under cyclic loading has been observed. Yasuhara et al. [28] state that there is no apparent effect of frequency on shear strength of soil subjected to cyclic loading in undrained conditions.

In the light of all the above observations the authors will present later other, unusual behaviour of cohesive soil subjected to low-frequency cyclic loading of low amplitude. Attention will be focused

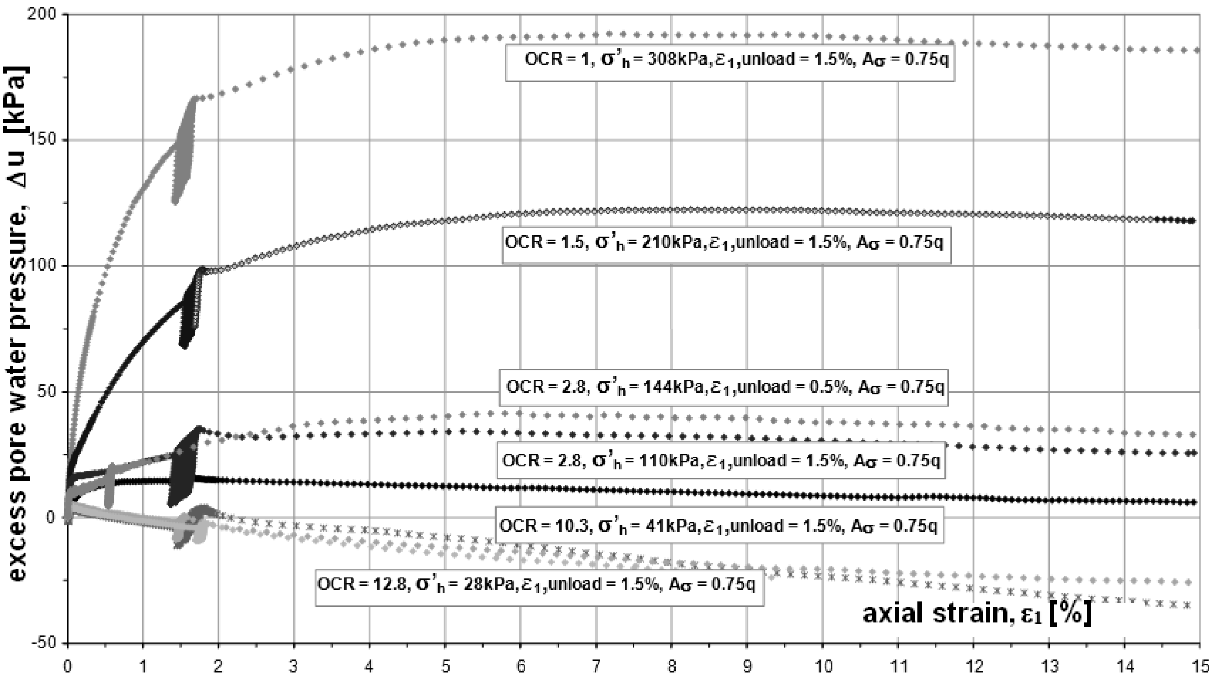

Fig. 5. Increase of pore water pressure in function of axial strain for cohesive soil with variable values of overconsolidation ratio OCR, effective stress σ'_h, axial strain initiating the cyclic load operation $\varepsilon_{1,\mathrm{unload}}$ and cyclic load amplitude A_σ (after Jastrzębska [12])

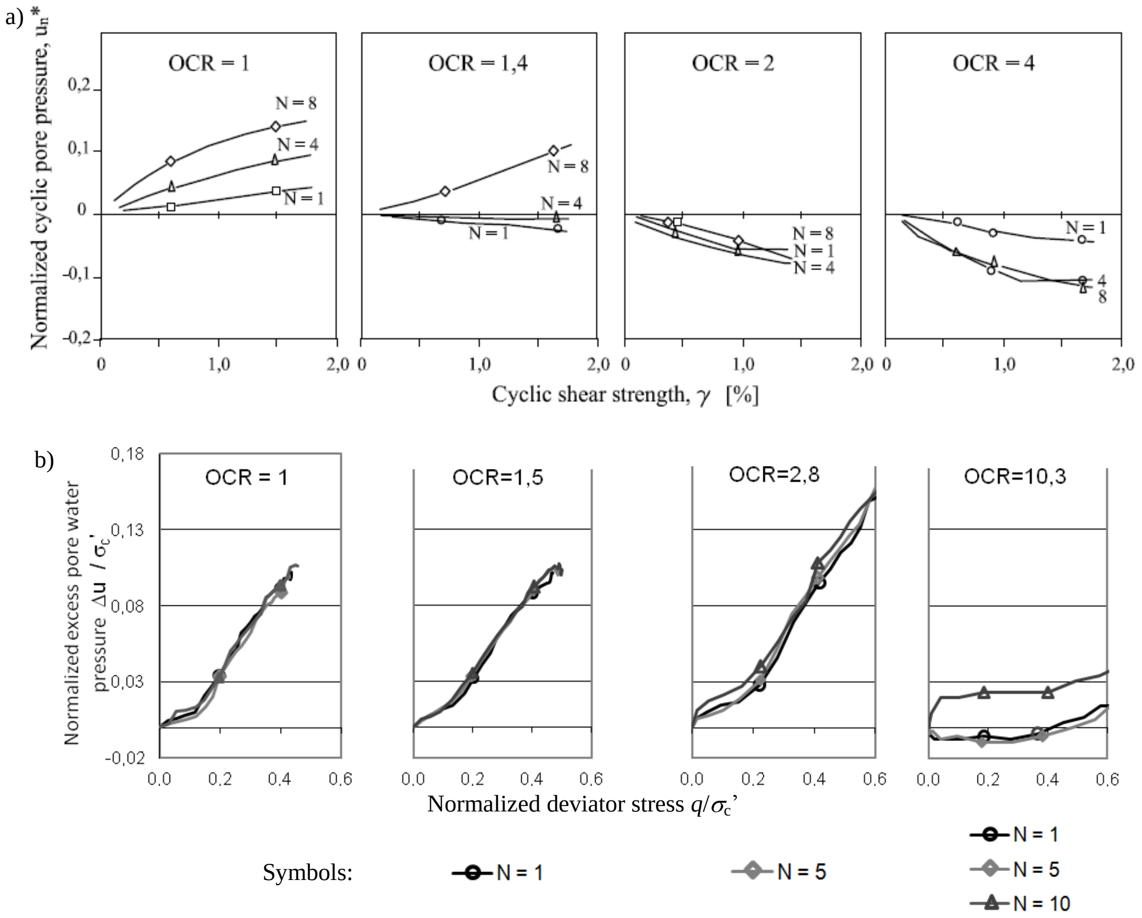

Fig. 6. Development of pore water pressure for soil samples with different values of overconsolidation ratio *OCR* depending on: (a) the level of shear strain (after Vucetic and Dobry [25]), (b) normalized stress deviator (after Jastrzębska [12])

on the effective stress paths course, development of excess pore water pressure and deviator stress value.

3. DESCRIPTION OF LABORATORY EQUIPMENT

Laboratory tests were conducted in a triaxial apparatus with specially modified cell, designed by Lipiński [14]. Modification of the cell is schematically shown in Fig. 7. Introduced innovations helped eliminate the errors resulting from the lack of complete contact surface and coaxiality of the sample and the piston, inaccurate application of the top can to the piston and also enabled to determine the exact value of the vertical stress acting on the sample, according to the formula (2)

$$\sigma_1 = \frac{P + Q + \sigma_3 (A_p - A_t)}{A_p} \quad (2)$$

where σ_1 – principal stress (axial stress), σ_3 – principal stress (radial stress–water pressure in the cell), P – force actinf on the piston, Q – self-weight of the piston and top can, A_t – cross-sectional area of the piston, A_P – cross-sectional area of the sample.

During the laboratory tests the following values are being monitored: water pressure in the cell σ_3,

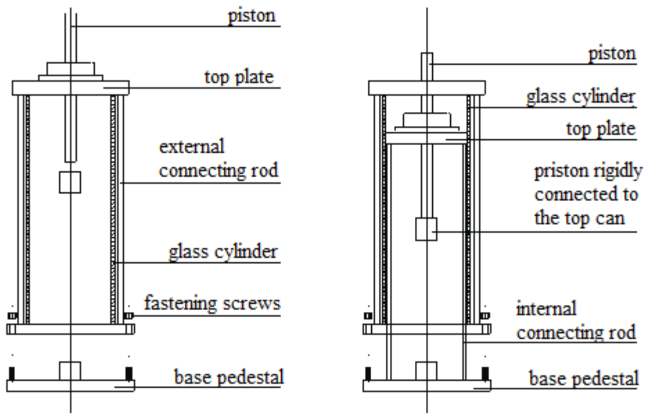

Fig. 7. Differences in the construction of traditional and modified TX 93 cell: (a) scheme of traditional cell, (b) scheme of modified TX 93 cell

pore water pressure u, force acting on the piston P, volumetric changes ΔV (by measuring the volume of water flowing out of the sample and stored in a volumeter) and axial deformation Δh_i (on the base of measurements of external strain sensor).

Data from the measurements are being recorded continuously on a computer.

4. MATERIAL AND SAMPLE PREPARATION

The subject of the research is kaolinite clay: a soil density ρ_s = 2.64 t/m^3, natural water content w_n = 35% and plasticity index I_P = 22.2%. Its other parameters, quoted many times, may be found *inter alia* in Jastrzębska's dissertation [11].

Tests are conducted on reworked soil samples, made of ground paste with a moisture content θ ≈ 50% (~1.2 L_L). Before receiving proper samples, ground paste is consolidated by the isotropic stress equal to 80 kPa.

Afterwards, the samples are trimmed to their proper dimensions: diameter D ≈ 50 mm and H/D ratio ≈ 2. After placing the sample in a triaxial apparatus cell the process of sample saturation begins. Initially samples are saturated by gravity and then by *back pressure method*, thanks to which in realized tests the obtained values of Skempton's parameter were B = 0.95 ÷ 0.98.

After completion of saturation, samples are subjected to proper isotropic consolidation by the effective stress equal to 300 kPa (without unloading – normally consolidated samples, OCR = 1).

5. RESEARCH PROGRAM

As mentioned in the introduction, soil reaction under the impact of low-frequency cyclic loading was monitored in the conducted research. Cyclic load was implemented after monotonic load applied until predetermined value of axial strain ($\varepsilon_{1,\mathrm{unload}}$) was reached. Therefore, cyclic load was carried out at a low velocity v = 0.22 mm/h (tests no. 3 and 4) and 0.45 mm/h (tests no. 5 and 6) and frequency f ≈ 0.001 Hz. Such frequency is low enough to exclude the presence of dynamic phenomena.

Tests were conducted with constant strain amplitude (A_ε = 0.02%). The research program assumed execution of the bigger number of load–unload cycles that would lead to soil sample failure. So far six tests have been performed. In the first two, the number of load cycles achieved up to 155. Therefore, results of these tests have not been taken into further consideration. In subsequent tests a few thousand cycles of load–unload were performed, although in none of them decrease in strength of the material was observed. Because of the external reasons (power outage or computer failure), none of the samples was led to failure. However, it does not change the fact that during experiments there have been observed interesting regularities (see Section 6).

To the further analysis there have been selected tests in which at least 1000 cycles of load–unload were performed:

- Test no. 3 – 1593 cycles of load–unload with strain amplitude A_ε = 0.02% and axial strain initiating the cyclic load operation $\varepsilon_{1,\mathrm{unload}}$ = 0.5% ($\Delta\varepsilon_1$ = 0.5%÷0.48%), duration of the test: 28 days,
- Test no. 4 – 2286 cycles of load–unload with strain amplitude A_ε = 0.02% and axial strain initiating the cyclic load operation $\varepsilon_{1,\mathrm{unload}}$ = 0.7% ($\Delta\varepsilon_1$ = 0.7%÷0.68%), duration of the test: 41 days,
- Test no. 5 – 2552 cycles of load–unload with strain amplitude A_ε = 0.02% and axial strain initiating the cyclic load operation $\varepsilon_{1,\mathrm{unload}}$ = 0.7% ($\Delta\varepsilon_1$ = 0.7%÷0.68%), duration of the test: 28 days,
- Test no. 6 – 5134 cycles of load–unload with strain amplitude A_ε = 0.02% and axial strain initiating the cyclic load operation $\varepsilon_{1,\mathrm{unload}}$ = 1.0% ($\Delta\varepsilon_1$ = 1.0%÷0.98%), duration of the test: 49 days.

6. RESULTS

The object of results analysis is to determine the influence of number of load cycles on the course of stress paths, development of excess pore water pressure and stress deviator value.

Effective stress paths

The course of stress paths is shown in Fig. 8 (tests: 3 – marked with black colour and 4 – with grey colour) and Fig. 9 (tests: 5 – marked with grey colour and 6 – with black colour). On the charts there are presented significant load–unload cycles, meaning such, after which change in direction of stress path occurs or which are the last ones for each test. These loops have been described with numbers (n_1, n_2, ...), where the subscripts indicate the order of particular direction changes. Moreover, arrows below the descriptions

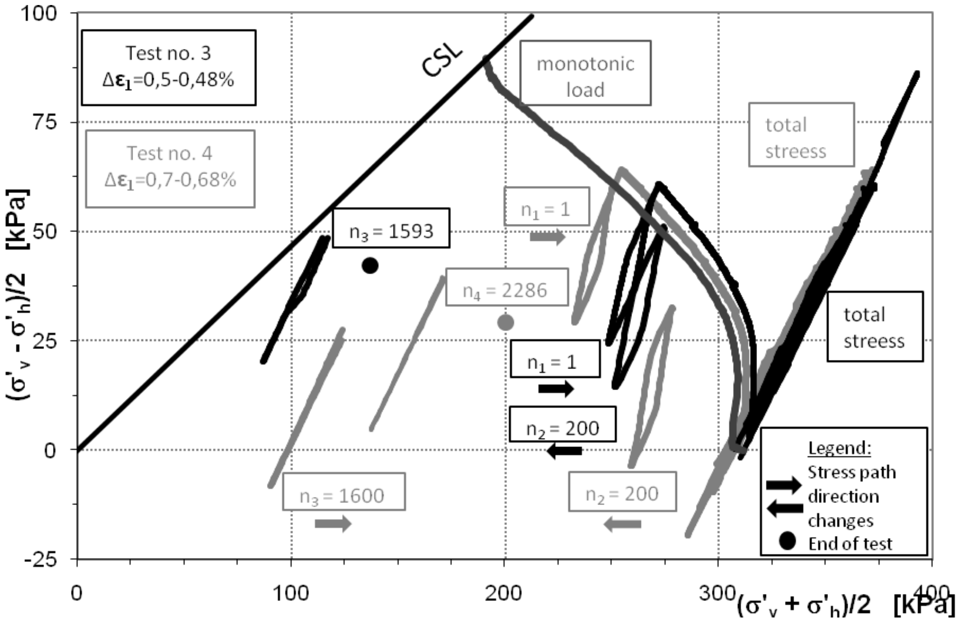

Fig. 8. Effective and total stress paths in tests no. 3 and 4 presented for significant load–unload cycles, after which change in direction of stress path occurs or which are the last ones for each test

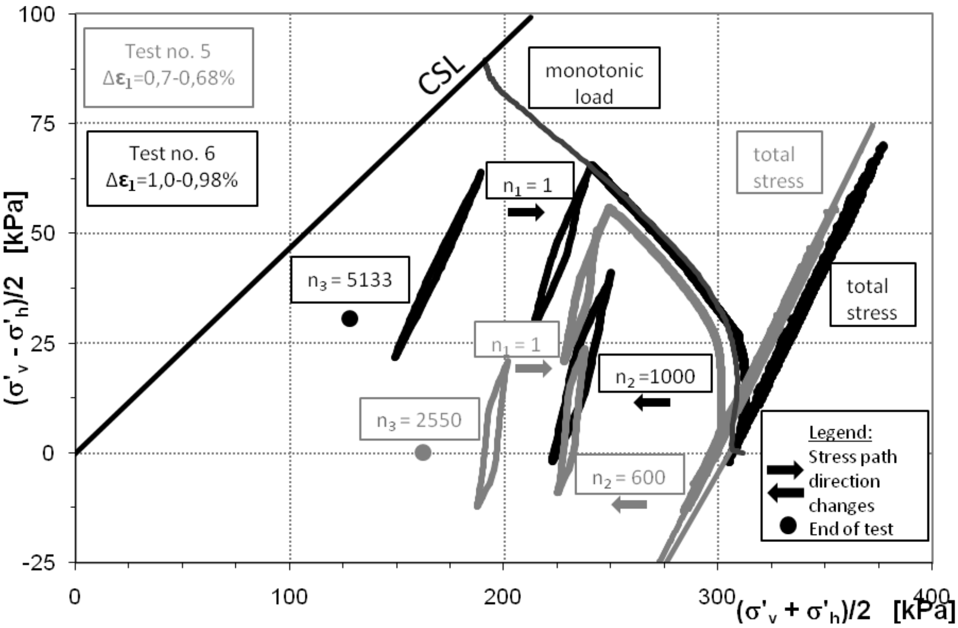

Fig. 9. Effective and total stress paths in tests no. 5 and 6 presented for significant load–unload cycles, after which change in direction of stress path occurs or which are the last ones for each test

indicate the direction of the stress path course after particular load–unload cycle. Additionally, next to each loop there is a number corresponding to number of load–unload cycle in particular test.

It should be noted that in all tests the effective stress path initially moves away from the boundary surface. Only after a certain number of load-unload cycles (in the case of test no. 3 and 4 – after about 200 cycles, 5 – after 600 cycles, 6 – 1000 cycles) the stress path changes its direction and moves towards the failure surface. Consistent approaching of stress path to the boundary surface suggests that if the test had not been interrupted, applied cyclic load would have led to the sample failure.

Excess pore water pressure

Changes in effective stress value are closely related to the changes in value of pore water pressure (Terzaghi's formula).

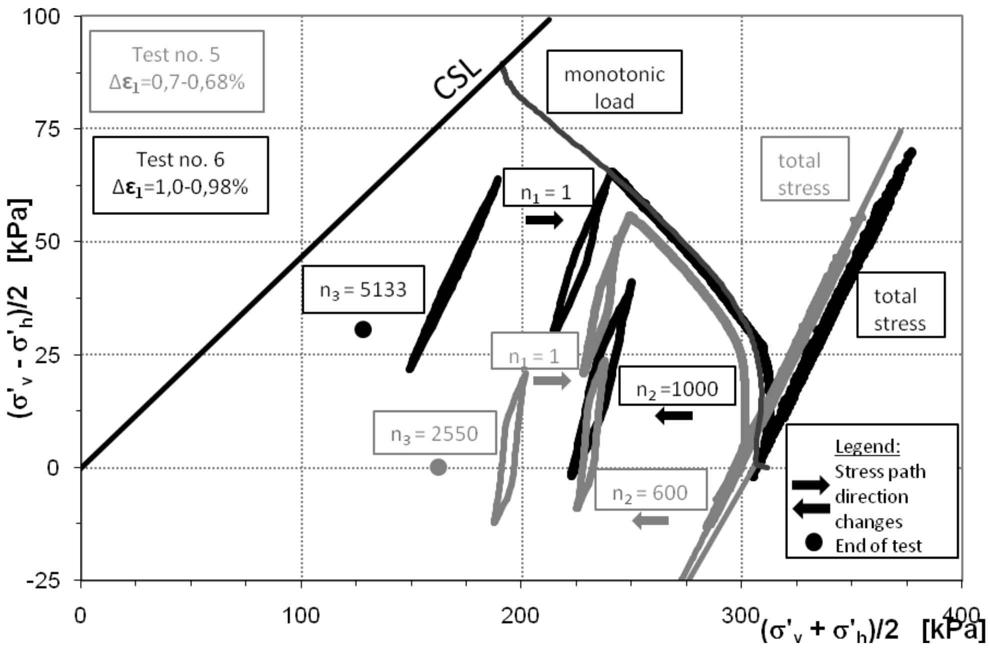

Fig. 10. Development of excess pore water pressure as a function of axial strain in tests 3–6

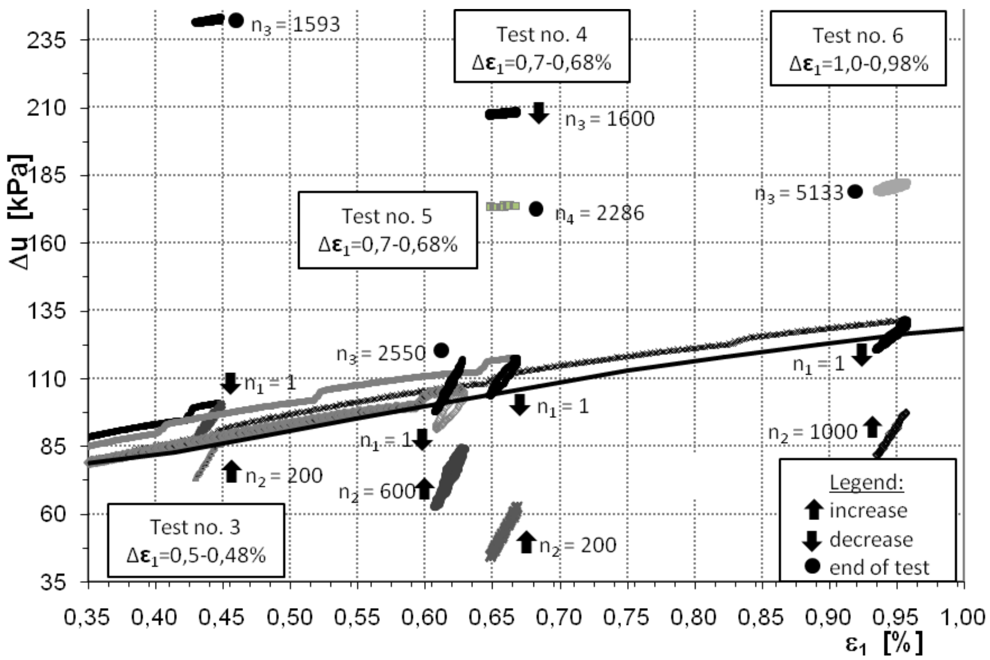

Fig. 11. Development of excess pore water pressure as a function of axial strain in tests 3–6 presented for significant load–unload cycles, after which change in direction of stress path (and respectively an increase or decrease of pore water pressure value) occurs or which are the last ones for each test

Development of excess pore water pressure for all load–unload cycles is presented in Fig. 10, while for significant cycles (meaning, the same for which the direction of effective stress path changes, according to Fig. 8 and Fig. 9) is shown in Fig. 11. As in the case of representation of stress paths course, characteristic load-unload cycles are described with numbers (n_1, n_2, ...), while the increase or decrease of pore water pressure value after each of them, is graphically shown with the use of arrows.

It can be observed that value of excess pore water pressure initially decreases and after a certain number of load-unload cycles (in the case of test no. 3 and 4 – after about 200 cycles, 5 – after 600 cycles, 6 – 1000 cycles) begins to increase again. It is an increasing value of excess pore water pressure that causes approaching of the stress path to the boundary surface. Finally, if continuing the loading, it could lead to failure of the material.

Initial decrease in value of pore water pressure might be a little bit surprising, because it is a typical behaviour for overconsolidated soil (see Section 2), while the test samples are normally consolidated. However, it should be noted that cyclic process with constant strain amplitude begins with unloading, and in the initial phase subsequent cycles are realized inside the history surface (cycles move away from the primary envelope), in the range of secondary loads and thereby overconsolidation.

Stress deviator

The variation of stress deviator for all load-unload cycles is presented in Fig. 12, while for significant cycles, which are followed by the change trend of deviator stress value (increase or decrease) is shown in Fig. 13. These loops are described with numbers (n_1, n_2, ...), which indicate the order of these changes and there are also numbers corresponding to the number of cycle in proper test. Increase or decrease of deviator stress value after each specified load–unload cycles is graphically shown with the use of arrows.

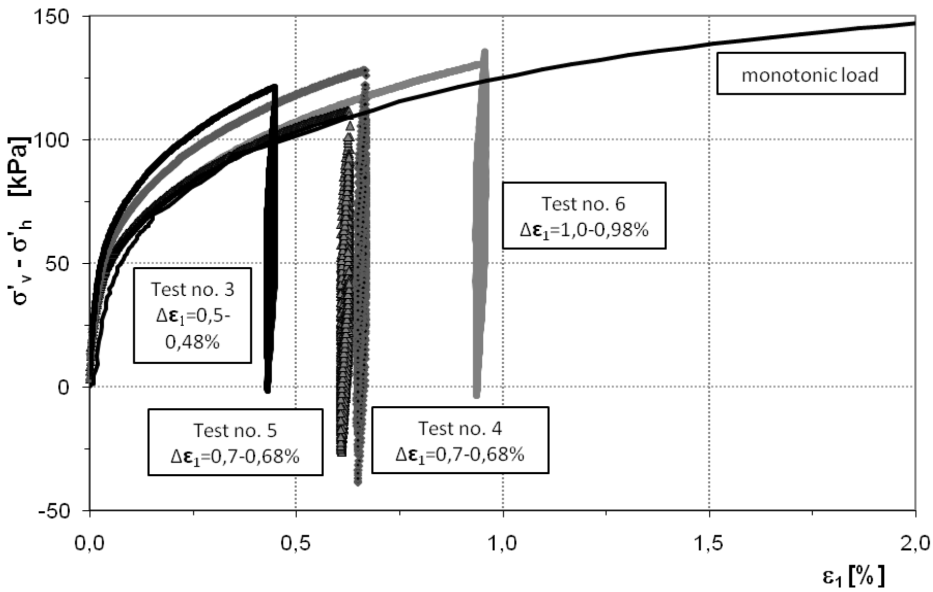

Fig. 12. Variation of stress deviator as a function of axial strain in tests 3–6

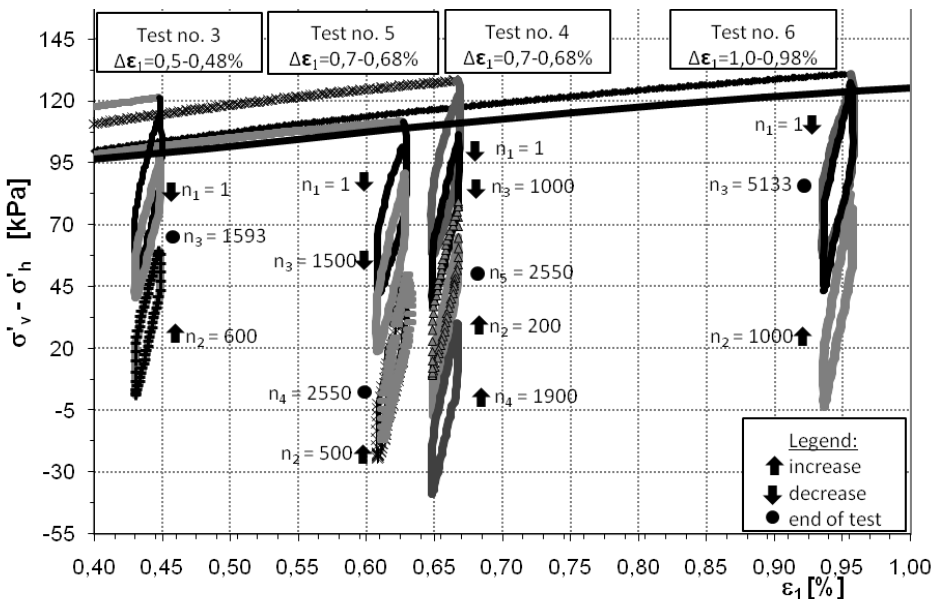

Fig. 13. Variation of stress deviator as a function of axial strain in tests 3–6
presented for significant load–unload cycles, after which trend change of deviator stress value (increase or decrease) occurs or which are the last ones for each test

It can be observed that value of stress deviator initially decreases, but after a few hundred load–unload cycles it begins to increase again. Analyzing the results, it can be noted that in the case of test no. 6, stress deviator re-increase occurs after about 1000 load-unload cycles, which is analogous to the increase of pore water pressure (Fig. 11). However, it is not a rule. For instance, in test no. 4 re-increase in value of stress deviator occurs after 600 load–unload cycles, while for comparison an increase in pore water pressure value proceeds after 200 cycles.

7. CONFRONTATION OF THE RESULTS

Because of the surprising test results, it has been decided to confront the observed phenomena with

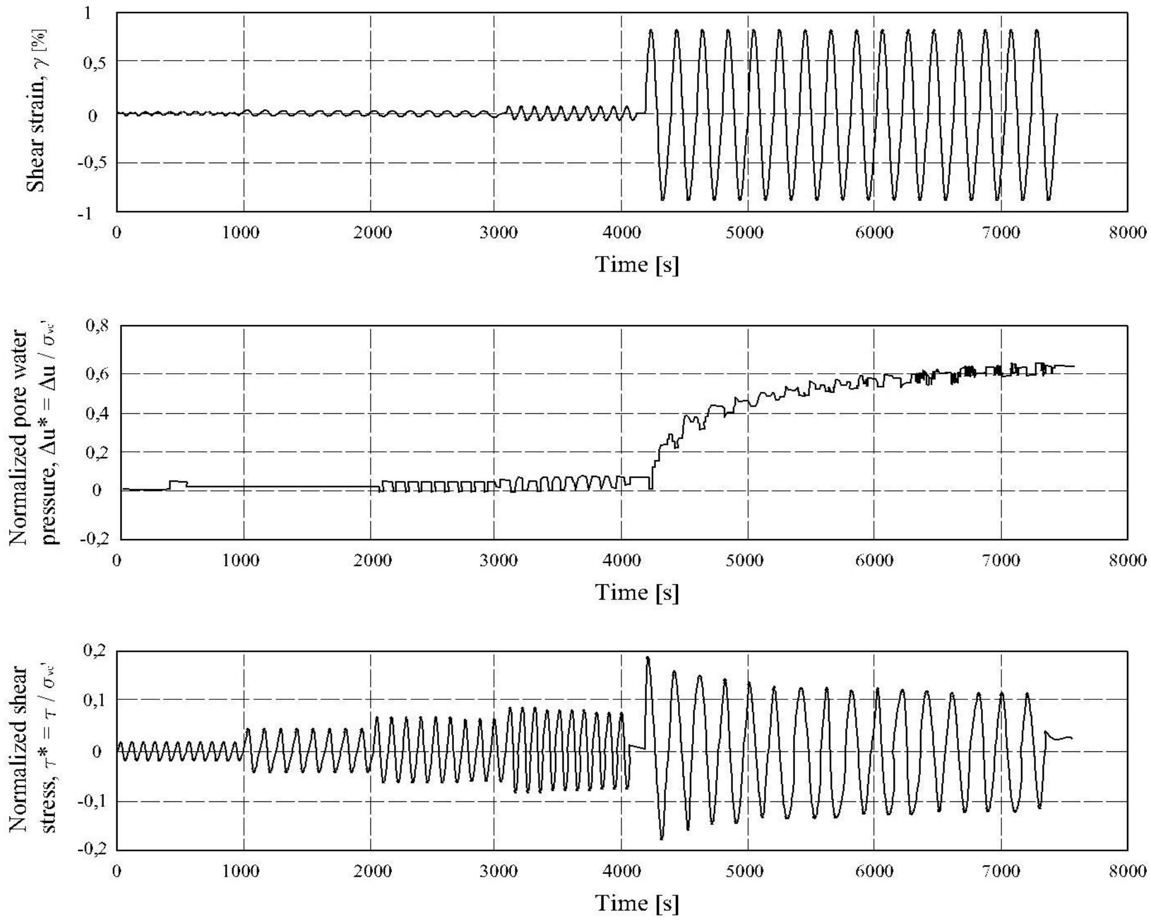

Fig. 14. Variation of strain, pore water pressure and stress with time in a cyclic strain-controlled test on kaolinite Clay with OCR = 1, σ'_{vc} = 213 kPa and f = 0.01 Hz (after Mortezaie [17])

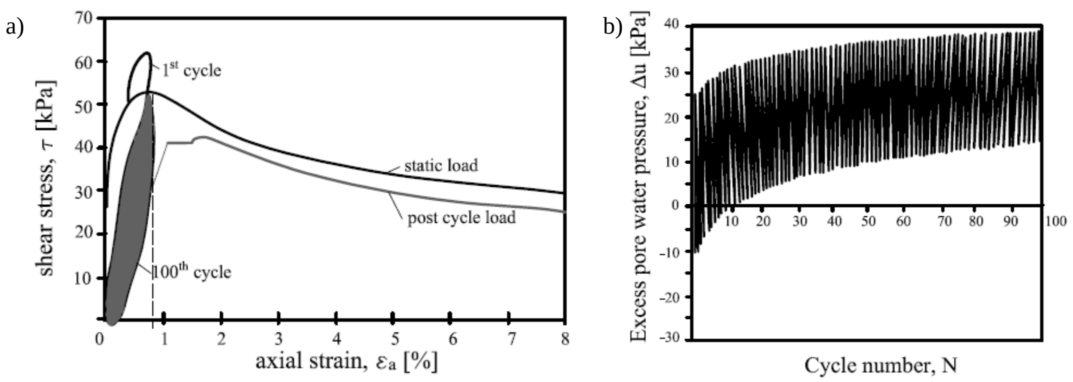

Fig. 15. Measured (a) stress-strain response (b) pore pressure increase in strain-controlled cyclic tests at 0.01 Hz with cyclic strains equal to the failure strain in static tests (after Ahnberg et al. [1])

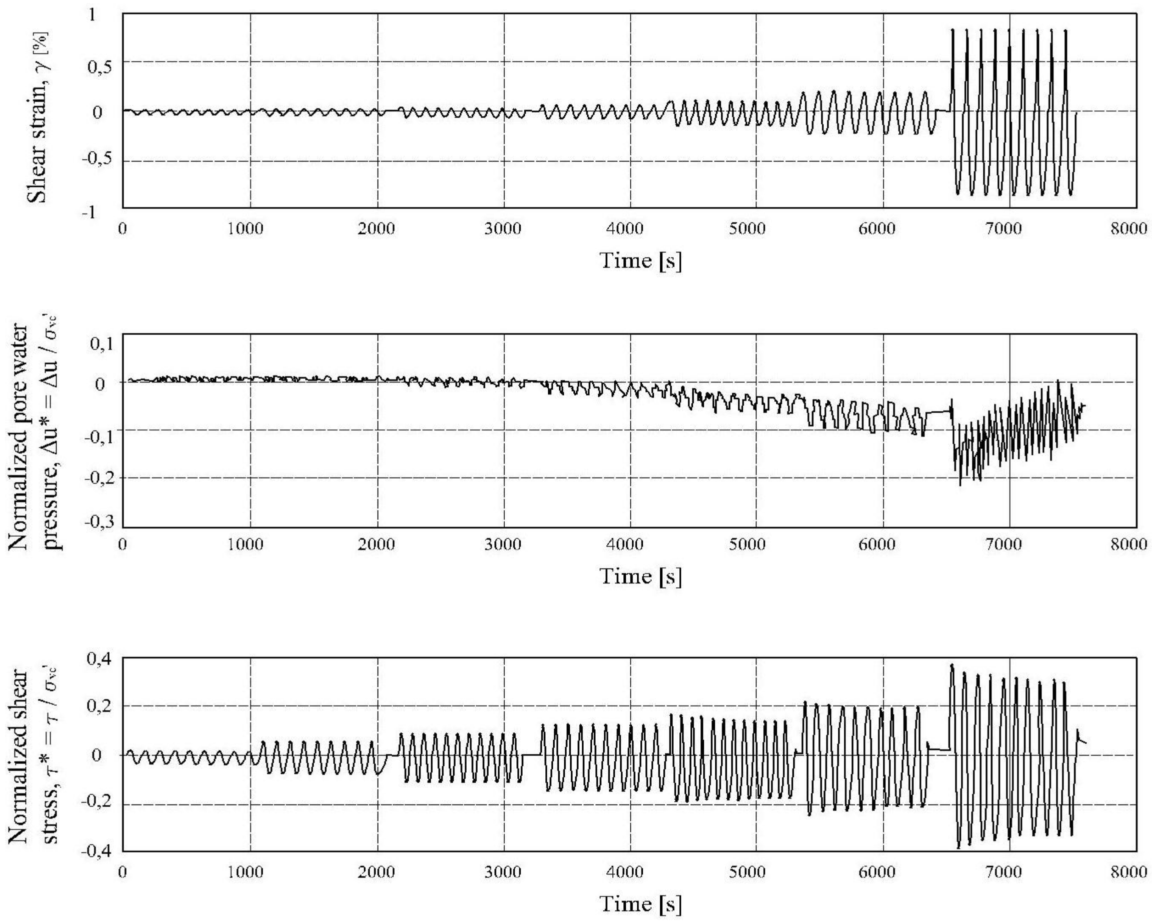

Fig. 16. Variation of strain, pore water pressure and stress with time in a cyclic strain-controlled test on kaolinite Clay with OCR = 4, σ'_{vc} = 211 kPa and f = 0.01 Hz (after Mortezaie [17])

results of other similar tests (with constant strain amplitude), available in the literature (i.e., Mortezaie [17], Ahnberg et al. [1]).

The following relations were stated:

1. Normally consolidated soil:

a) small strain amplitude ($A_\varepsilon \le 0.5\%$) – slight and slow increase in excess pore water pressure value Δu, decrease in value of shear stress τ (Fig. 14).

b) large strain amplitude ($A_\varepsilon > 0.5\%$) – substantial and rapid increase in excess pore water pressure value Δu, decrease in value of shear stress τ (Fig. 14, Fig. 15).

2. Overconsolidated soil:

a) small strain amplitude ($A_\varepsilon \le 0.5\%$**)** – decrease in excess pore water pressure value Δu, decrease in value of shear stress τ (Fig. 16).

b) large strain amplitude ($A_\varepsilon > 0.5\%$) – increase in excess pore water pressure value Δu, decrease in value of shear stress τ (Fig. 16).

The results presented in the literature do not confirm, but also do not exclude the correctness of the phenomena observed in the presented own study, the results of which are additionally presented in systems: *"normalized stress deviator (q/σ_c) – time"* and *"normalized excess pore water pressure ($\Delta u/\sigma_c$) – time"* (Fig. 17–Fig. 20).

In the case of research conducted by Mortezaie [17] and Ahnberg et al. [1], a decrease in value of shear stress can be observed in subsequent cycles of load-unload. On the other hand, there is no its further re-increase, that occurs in presented own research (Fig. 17–Fig. 20). However, it should be noted that the number of load–unload cycles in case of these studies (Mortezaie [17]; Ahnberg et al. [1]) is much smaller and it cannot be excluded that in subsequent load-unload cycled re-increase in value of shear stress would not occur.

Initial decrease in pore water pressure value, in the case of normally consolidated samples, on the first sight may seem unlikely. Nevertheless, as mentioned is Section 6, in presented tests the cyclic process begins with unload and initial load–unload cycles are performed in the range of secondary loads and thereby overconsolidation. This is a different situation than in the case of research presented above (Mortezaie [17]; Ahnberg et al. [1]), in which the cyclic process starts from the axial strain $\varepsilon_1 = 0$.

On the basis of Mortezaie's research [17] it can be concluded that in the case of overconsolidated soil, decrease or increase in excess pore water pressure value is dependent on amplitude (Fig. 16): At small strain amplitude pore water pressure value decreases in subsequent cycles, while in the case of large strain amplitude it increases. In presented own research (Kalinowska

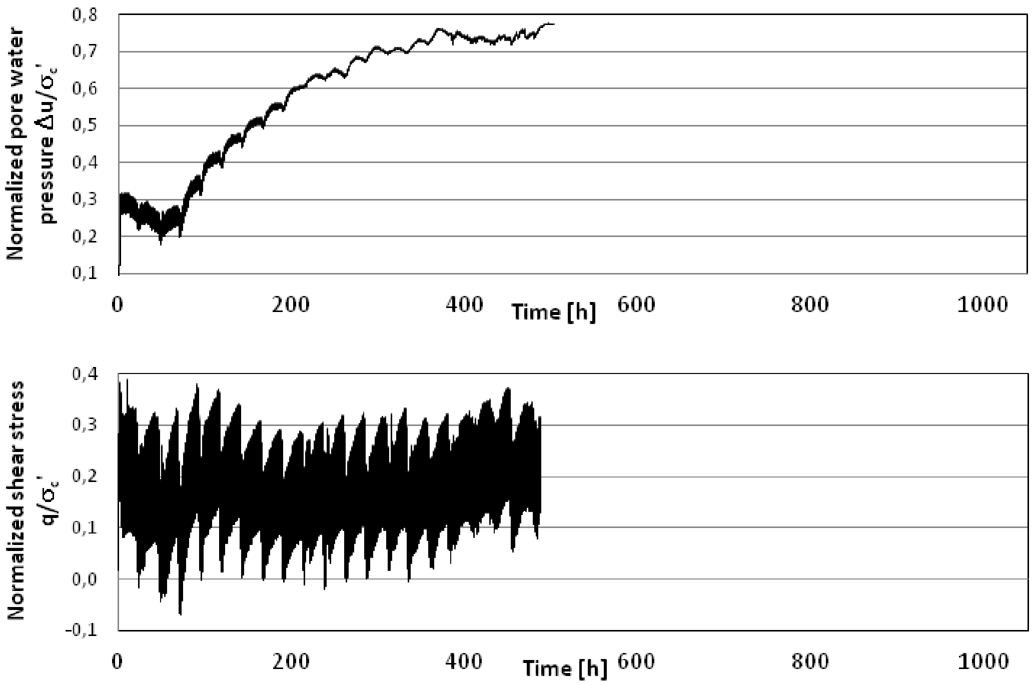

Fig. 17. Variation of pore water pressure and stress with time in a cyclic strain-controlled test on kaolinite Clay with OCR = 1, $\sigma_c' = 300$ kPa and $f = 0.001$ Hz, $\Delta\varepsilon_1 = 0.5 – 0.48\%$ (test no. 3)

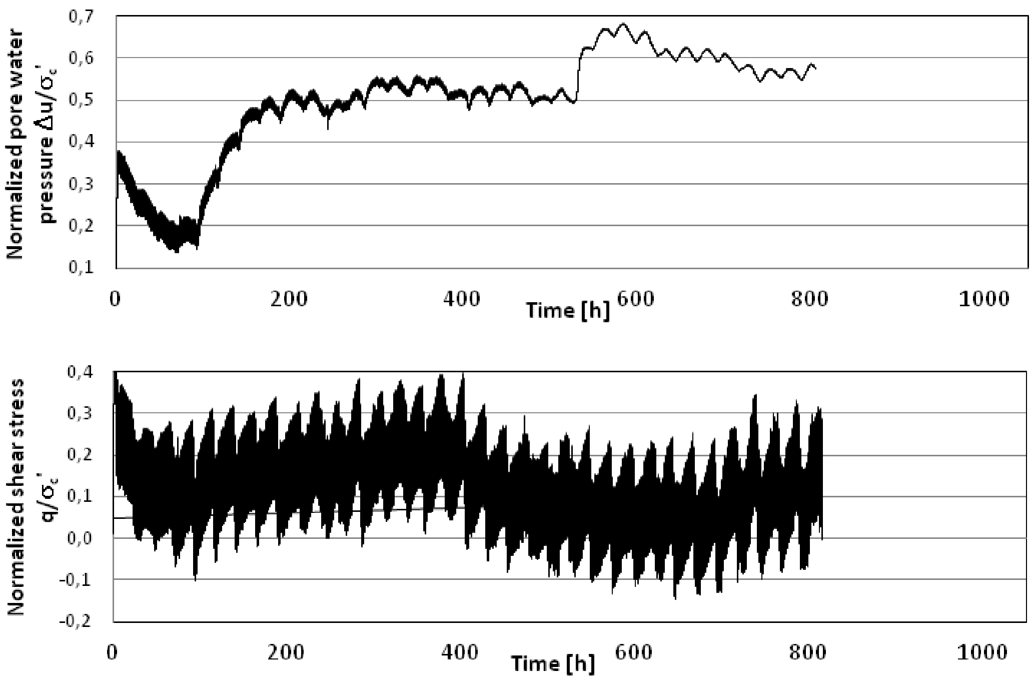

Fig. 18. Variation of pore water pressure and stress with time in a cyclic strain-controlled test on kaolinite Clay with OCR = 1, $\sigma_c' = 300$ kPa and $f = 0.001$ Hz, $\Delta\varepsilon_1 = 0.7 – 0.68\%$ (test no. 4)

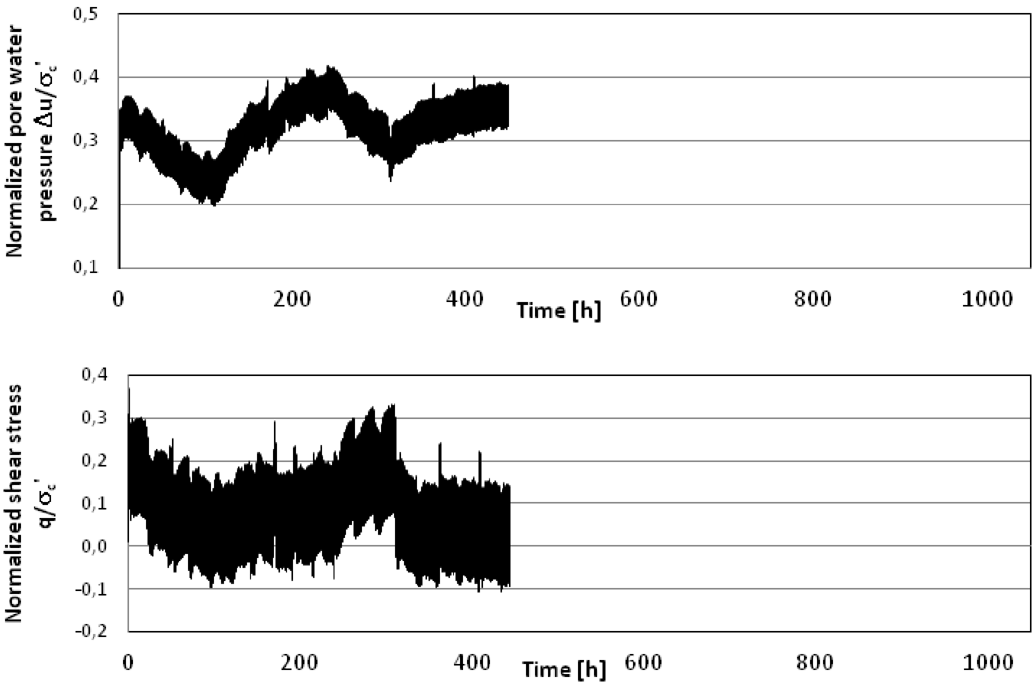

Fig. 19. Variation of pore water pressure and stress with time in a cyclic strain-controlled test on kaolinite Clay with OCR = 1, σ_c' = 300 kPa and f = 0.001 Hz, $\Delta\varepsilon_1$ = 0.7 – 0.68% (test no. 5)

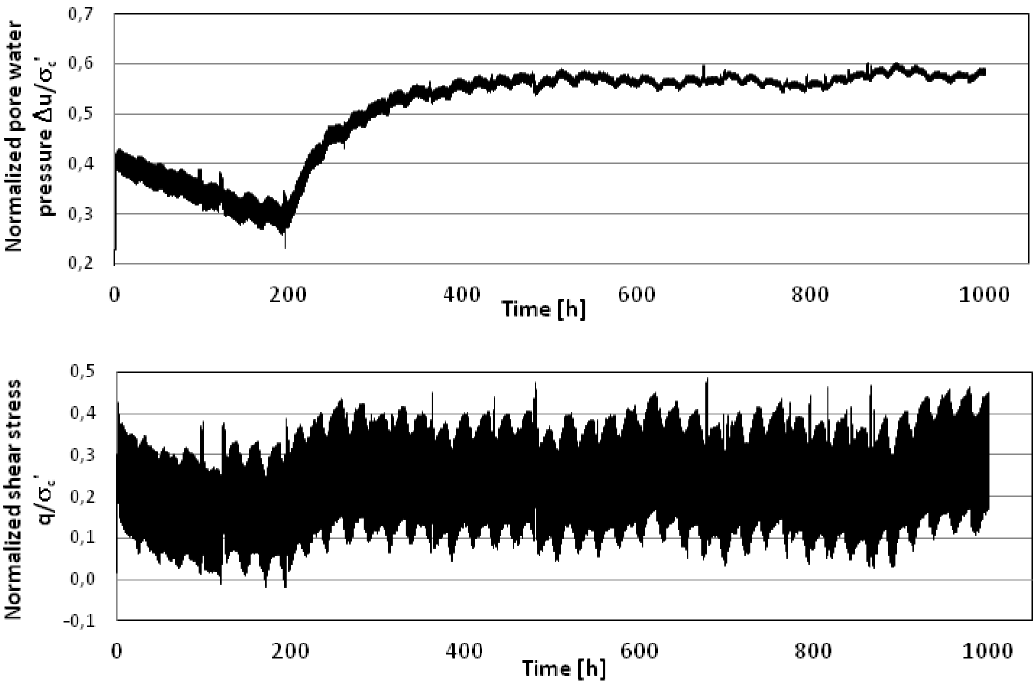

Fig. 20. Variation of pore water pressure and stress with time in a cyclic strain-controlled test on kaolinite Clay with OCR = 1, σ_c' = 300 kPa and f = 0.001 Hz, $\Delta\varepsilon_1$ = 0.1 – 0.98% (test no. 6)

[13]), in which strain amplitude is small (A_ε = 0.02%), there can be observed an initial decrease in excess pore water pressure value and after a certain number of load –unload cycles its re-increase (Fig. 17–Fig. 20). It is quite possible that excess pore water pressure value in each case would ultimately increase and it depends on the amplitude (and overconsolidation ratio OCR) after how many load–unload cycles this would occur.

8. CONCLUSIONS

The presented experiments are part of the research conducted by Jastrzębska, presenting behaviour of cohesive soil subjected to low-frequency cyclic loading. Unlike the previous tests (Jastrzębska [12]), which were stress-controlled, the current ones are performed with constant strain amplitude. Moreover, the number of load–unload cycles has been increased. In the latest tests presented by Kalinowska and Jastrzębska (Kalinowska [13]), there were even 5000 load–unload cycles.

During experiments, the object of which was normally consolidated soil, there have been observed interesting regularities. The most interesting finding is an observation of initial decrease in pore water pressure value (after activation of cyclic loading) and then its successive, slow re-increase. Similar behaviour was observed in the case of stress deviator: initial decrease and then its re-increase. These observations, appropriate for all the six tests, are surprising because they refer to normally consolidated soil.

These phenomena reveal the complexity of the behaviour of cohesive soil subjected to cyclic loading. It is obvious that it is very difficult to predict a cohesive soil response to any cyclic load, because it depends on many factors and differs from behaviour of cohesionless soil subjected to similar loads.

Initial decrease in pore water pressure value (Fig. 11), although the sample is normally consolidated, can be explained by the fact that initial axial strain value $\varepsilon_{1,\text{unload}} \neq 0$.

Observed regularities may complement the description of the behaviour of cohesive soil subjected to low-frequency cyclic loading. They can also be the basis for verification and development of the NAHOS model, which is the subject of scientific works in the Department of Geotechnics on Silesian University of Technology (Jastrzębska [11]; Uliniarz [23]). Development of the model can contribute to the general development of numerical description of cohesive soil, especially in the range of small strain and under the impact of low-frequency cyclic loads.

Observed phenomena and potential benefits of conducted research lead to the conclusion that it should be continued. Conducting more tests in the future will contribute to the formulation of failure criterion for clay subjected to low-frequency cyclic loading.

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