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THE INFLUENCE OF SELECTED PARAMETERS OF CYCLIC PROCESS ON COHESIVE SOILS SHEAR CHARACTERISTICS AT SMALL STRAIN

M. JASTRZĘBSKA[1]

The subject of the paper comprises a cohesive soil response to a cyclic loading applied in the range of small strains ($10^{-5} \div 10^{-3}$). To this end tests of undrained cyclic shear in a triaxial compression apparatus were carried out on homogeneous material – kaoline from Tułowice. The tests were carried out on a modernised test bed, enabling full saturation of samples using the back pressure method, as well as a precise, intra-chamber measurement of small strains. Maintaining a constant deviatoric stress amplitude for NC and OC soils, the effect of its size (A = 0.75Δq or A = 0.375Δq) as well as the influence of strain rate on material characteristics "deviatoric stress (excess pore water pressure) – axial strain" and effective stress paths were tested. While analysing the results obtained, a phenomenon of closing and stabilising initially open and moving loops were found, in contrast to proposal by *Jardine* [8]. The observed increments in the axial strain during cyclic loading operation, at the same levels of lateral effective stress, were greater for normally consolidated than for overconsolidated soils. At the same time, at each next cycle, these increments were smaller and smaller, assuming even the value equal to zero for the tenth cycle. Similar relationships occurred during the increase in the pore water pressure during the cyclic load action. For the set number of cycles n = 10 they were that small – max. 46% (and decreasing with each consecutive cycle) that they did not result in weakening of the material. Taking the trend of decreasing Δu increments into account it was possible to accept that the conclusion considered was right irrespective of the cycles' number.

Key words: cohesive soils, small strain, cyclic loading, deviator stress amplitude, axial strain rate.

NOTATIONS

A	–	amplitude (deviator stress amplitude), kPa
N, n	–	number of cycles
v3, v4, v6,v8	–	loading rate in triaxial tests, mm/h
$\varepsilon_{1,unload}$ (EPS1,odpr)	–	axial strain initiating the cyclic load operation, %,
BS	–	bounding surface
CIU	–	testing with isotropic consolidation and shearing without drainage
CSL	–	critical state line
NC, OC	–	normally consolidated soil, overconsolidated soil

[1] PhD, Department of Geotechnics, Faculty of Civil Engineering, Silesian University of Technology, Gliwice, Poland, e-mail: malgorzata.jastrzebska@polsl.pl

1. Introduction

Development of research and numerical methods resulted primarily from the needs of modern geotechnical practice and from increase possibilities of implementation. These needs are related to new, larger and more difficult investment projects, such as high earth dams, open pit mines, heavy industrial buildings, off-shore drilling platforms, large bridges. The foundation on weak, seismically active and mining subsoils adds to the problem. A complex and peculiar behaviour of soils in the ranges of very small and small strain is today one of the most important research areas in the soil mechanics. Another priority consists of understanding and describing the response of the environment to cyclic loading, on various levels of stress, commonly existing in technology and nature. Both these cases concern exploration of areas poorly recognised because of its complexity, as well as the implementation of necessary progress in the geotechnical design.

While focusing on the analysis of the situation of research on mechanical properties of soils in the range of small strain, it may be stated that the results quoted in the literature apply basically to the response to a monotonic load. In the best case they apply to a single "loading – unloading" cycle (*Jardine,* [7]). This opens the field to original research aimed at investigating properties of soil response to cyclic loading in the range of small strain. The choice of cohesive soil is determined by the fact of dissimilarity of phenomena occurring in those conditions, in non-cohesive and cohesive soils, as well as by incomparably poorer knowledge of cyclic processes in the latter ones.

Tests of water drainage prevented from a sample are a natural implication of subject's narrowing to the cohesive soils. Free drainage conditions are possible only in the cases of especially slow cyclic processes, e.g. careful filling and emptying of tanks, and rather solely in poorly or medium cohesive subsoils. In overwhelming majority of cyclic influences at most the initial consolidation stages, reasonably represented by undrained conditions, are realistic. The tests of prevented water drainage provide also more information on the behaviour of environment subjected to cyclic loading in the range of small strain. After all, the soil response is reflected in the "$q - \varepsilon_s$" shear characteristic and in the "$q - p'$" effective stress path. The course of the latter may be especially interesting and dependent on the stress amplitude.

Both the phenomena in the area of small strain, as well as processes of soil response to cyclic loading play, crucial role in geotechnical designing and contracting; so it may be expected that the effect of their combined occurrence will be spectacular.

2. Changes in nature of mechanical phenomena within small deformations

The issues related to very small and small strain (of the order of 10^{-3} and less) have their roots in the beginning of the seventieth, associating with resonant columns and "dynamic" shear moduli (e.g. Hardin and Drnevich, [6]). The eightieth and later years have shown, firstly, a strong nonlinearity of "stress – strain" relationships, secondly, an identity of "dynamic" and "static" shear moduli, and thirdly a possibility to determine experimentally the relationship $G=f(p, \varepsilon_s)$ in the full strain range only on the basis of a combined database of experimental data originating from dynamic tests (bender elements and resonant columns), and precise triaxial tests with local measurement of strain and conventional triaxial tests with external measurement of strain (ATKINSON and SALLFORS [4]; BURLAND [5]; JARDINE [7]).

The technical progress and a broad experimental base provided foundations for the development of the theory explaining the physical nature of the stress-strain relationship nonlinearity at small strain, presented by JARDINE [7]. The theory is based on the postulate of the existence of four stress space zones around certain fixed point, beginning the process considered. The zones are situated around this point, approximately concentrically (Fig. 1). Each of them is characterised by a different deformation nature. The strains are measured by the distance from the beginning.

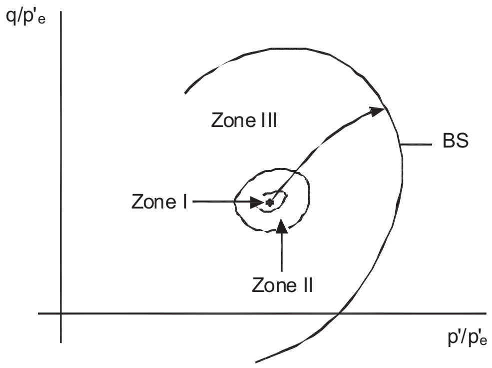

Fig. 1. Zones of the stress space differing in nature of deformations.
Rys. 1. Strefy przestrzeni naprężeń różniące się charakterem deformacji

Within zone I the soil behaves linearly elastically, but transversally anisotropically. In the surrounding zone II there occur transitional behaviour between the linear elasticity and plasticity. A characteristic feature consists in a different loading trajectory and different – unloading (Fig. 2) while maintaining full reversibility of the cycle. This phenomenon is macroscopically identified with an elastic hysteresis. From a microstructural perspective it is the implication of microslips between soil particles which result in energy dissipation (hence the hysteresis loop) and, what is more important, in strong sensitivity to the most recent loading history. This term is understood as a large influence of the last before the current state bend in the stress path on the stiffness change. The smaller the strain and the larger the bend angle, the larger (is a) positive increase in stiffness and the steeper its decline (ATKINSON *et al.*, [3]).

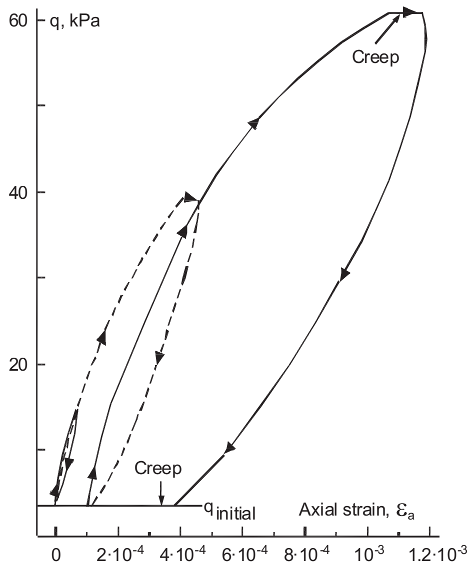

Fig. 2. Stress-strain behaviour in the range of transition from zone II to zone III (results of drained triaxial tests of Magnus till; after JARDINE [7])
Rys. 2. Cykle „obciążenie – odciążenie" na granicy stref II i III (wyniki badań trójosiowych gliny Magnus z odpływem wody; za JARDINEM [7])

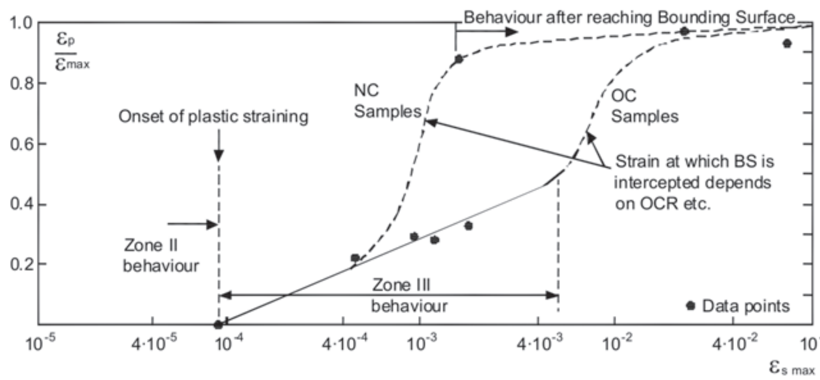

Fig. 3. Variations of the relative permanent shear strain (after JARDINE [7])
Rys. 3. Zmiany względnego trwałego odkształcenia postaciowego (za JARDINE [7])

The strain, at which the influence of the most recent history disappears, is considered as a point of external envelope of zone II (Fig. 1; ATKINSON *et al.*, [3]). Another criterion consists of the beginning of opening of hysteresis loop closed so far (Fig. 2) (JARDINE [7]; PUZRIN and BURLAND [14]). Up to now, no strict consistence of both criteria has been found. In points of zone II, the beginning of sudden decline in stiffness is observed. In zone III plasticity appears explicitly, manifesting itself in opening and shifting of the hysteresis loop and in a steep increase in the irreversible to total deformations ratio (Fig. 3). At the same time the dependence of the stiffness on the angle of the last stress path turn is no longer observed. The decrease of stiffness with increasing deformation diminishes. Cycles are open to a large extent.

3. Behaviour of the cohesive soils under cyclic loading

A cyclic process may be considered as a multiple change in the direction of load path by the angle of 180°. Its beginning is usually situated at the beginning of the first load curve.

In the next cycles the beginning of each of them is important at changes in the process course. The cycle amplitude is the second identifier of repeatable load in each cycle. In this case the soil response to the applied variable load consists of the drawing away of individual loops.

However, irrespective of the soil type, its response to cyclic loading in undrained conditions consists of loops shifting towards higher deformation values in the "$q - \varepsilon_s$" system, while in the "$q - p'$" stress space towards smaller values of mean stress p'.

Most studies related to soils behaviour under the influence of cyclic loading are devoted to sands. Only the construction, at the beginning of the seventieth, of drilling platforms on the North Sea, whose bottom is formed mainly of 'Drammen' clays, contributed to intensive tests related to cohesive soils. (e.g. Andersen [1]). Another perfectly prepared study is *state-of-the-art Wood* report [16], related to laboratory investigations of soils subjected to cyclic loading. In his report he (has) presented, inter alia, 30 publications on the behaviour of cohesive soils. The reading of this review reveals the main direction of investigations in the field considered till the eightieth – ninetieth of the twentieth century. The subject of most of them comprised soils subject to cyclic loading of relatively large and varying amplitude. Consecutive cycles of effective stress are spread between the passive and active boundary state, sometimes after a few process repetitions, and sometimes from its beginning (Takahashi *et al.*, [15]).

There is an impression, that the focus of cyclic processes researchers' interest comprises today on boundary problems in conditions of water drainage prevented and primarily the soil liquefaction resulting from pore water pressure accumulation. Two cases are distinguished here:

1) pulsating cyclic loading (cycles located on one side of static stress, Fig. 4),
2) oscillating cyclic loading (cycles located on both sides of static stress, Fig. 5).

There are significant differences between cohesive soil response to pulsating and oscillating loading, therefore it is necessary to identify the process in that respect.

Characteristic features of pulsating cyclic loading include the fact that cycles retain nearly the same shape, moving along ε_1 axis. The largest plastic strains occur in the

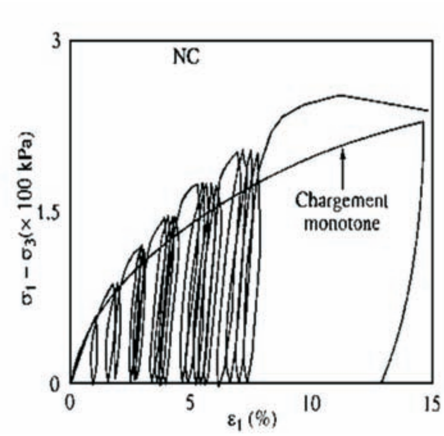

Fig. 4. Pulsating cyclic test of normally consolidated cohesive soil (ARTHUR *et al.* [2]).
Rys. 4. Badanie cykliczne pulsujące normalnie skonsolidowanego gruntu spoistego (ARTHUR *i in.* [2])

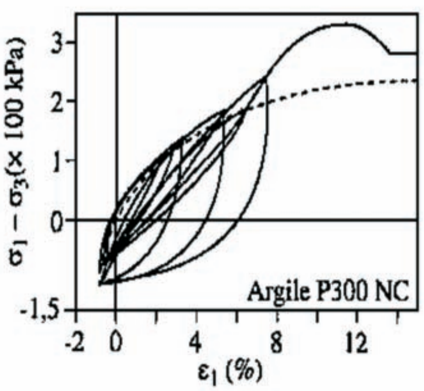

Fig. 5. Oscillating cyclic test of normally consolidated cohesive soil with drainage (ARTHUR *et al.* [2])
Rys. 5. Badanie cykliczne oscylujące normalnie skonsolidowanego gruntu spoistego z odpływem wody (ARTHUR *i in.* [2])

first cycle and they gradually decay with the following cycles. In turn, in the case of oscillating cyclic loading, in particular for tests without possibility of water draining from the sample, gradual soil degradation is observed. With each cycle the maximum and the secant shear modulus for the given cycle has a smaller value.

4. DESCRIPTION OF LABORATORY EQUIPMENT

A conventional triaxial apparatus was used to perform tests on kaolin samples. The triaxial cell contains internal tie bars and a rigid connection between the top cap and the loading piston. The diameters of top cap and pedestal were equal to that of the sample.

Strips of filter paper along the sample and porous stones screwed on the top cap and bottom base were used for drainage. The pressure cell was filled with de-aerated water.

Two different measurements of axial strain ε_1 were taken:

– Internal ε_1 on the lateral surface of the specimen using two couples of high resolution submergible proximity transducers. The transducers were mounted at two positions, opposite to each other, around the specimen diameter. The range and resolution of these transducers were 2.0 mm and 0.01%, respectively.
– External ε_1 using the external displacement gauge fixed on the loading piston.

Lateral strain ε_z was directly and locally measured by means of a couple of proximity transducers placed in the central part of the sample. A piece of thin aluminium foil was used as a target. This target was attached to the external membrane with silicone grease.

The data reading took place at chosen time intervals.

5. Material and samples preparation for triaxial compression

The material used in this study came from the Porcelain Factory in Tułowice. Its basic properties are given in Table 1 (Jastrzębska [9]). The tested soil exhibited great homogeneity of structure. In all cases the samples for triaxial tests were made on a soil paste of $w \approx 50\%$ water content (what makes around $1.2w_L$), which was initially consolidated at isotropic pressure equal to 80kPa. The adopted minimum values of initial consolidating pressure were dictated by obtaining the sample ultimately in such a state as it would be possible to cut out from it a proper specimen without the risk of

Table 1

Values of some physical properties and classification characteristics of Tułowice kaolin (Jastrzębska [9]).
Wartości parametrów fizycznych i klasyfikacyjnych kaolinu z Tułowic (Jastrzębska [9])

Specific gravity	G_s	t/m^3	2.637
Natural water content	w_n	%	31.90-37.73
Liquid limit	w_L	%	42.2
Plastic limit	w_p	%	20.0
Plasticity index	I_p	%	22.2
Liquidity index	I_L	–	0.60-0.78
Skempton's coefficient	A	–	0.52-0.6
Void ratio	e	–	0.956-1.098
Clay fraction	CF	%	37.0-37.9
Silt-size fraction	SF	%	53.7-56.3
Effective cohesion	c'	kPa	10.7
Effective angle of internal friction	ϕ'	°	25
Poisson's ratio	ν		0.085

losing its shape during the preparation and placing in the test cell. Finally all triaxial tests were carried out on samples 50 mm in diameter, 100 mm high. Each sample was saturated. At first they were flushed with de-aerated water. Thereafter a high back pressure was applied. The Skempton's B-values obtained were greater than 0.95.

Then, the samples were isotropically consolidated to the value of effective mean pressure p'_c of about 310 kPa in the case of undrained tests no 12-2, 12-2a, 12-3, 12-3a, (13-1) ÷ (13-6).

Next, five of the chosen soil samples were unloaded to effective pressures p'_0 of about 110 kPa (tests 12-3, 12-3a, 13-2, 13-4 and 13-6). This value corresponds to the overconsolidation ratio equal to $OCR = p'_c/p'_0 = 2.8$. After isotropic consolidation undrained tests were carried out.

Monotonic loading was continued until the value of axial strain $\varepsilon_{1,unload} \approx 1.5\%$ was reached. Then 10 loading cycles were applied (9 cycles in the case of test 13-2).

Table 2

Characteristics of tests carried out within the study.
Charakterystyki badań wykonanych w ramach pracy

Symbol of test	Type of test	Overconsolidation ratio	Axial strain rate	Lateral pressure (consolidation)	Pore pressure (consolidation)	Lateral pressure (shearing)	Skempton parameter	Void ratio initial / final	moisture content initial / final	Axial strain-start of cyclic load	Deviator stress amplitude
		OCR	v	σ'_3	u_b	σ'_3	B	e_0 / e_k	w_0 / w_k	$\varepsilon_{1,unload}$	A
			[mm/h]	[kPa]	[kPa]	[kPa]	[–]	[–]	[%]	[%]	[–]
12-2	CIU sch. I	1	0.22 v6	308	442	308	0.98 (450)	1.098 0.810	37.73 28.34	1.5	0.75Δq
12-2a	CIU sch. III	1	0.22	315	435	315	0.95 (450)	0.886 0.618	31.90 25.73	1.5	0.375q
12-3	CIU sch. I	2.8	0.22	309 110	441	110	0.98 (450)	1.041 0.861	35.73 29.25	1.5	0.75Δq
12-3a	CIU sch. III	2.8	0.22	314 114	436	114	0.97 (450)	0.904 0.714	32.66 26.18	1.5	0.375q
13-1	CIU sch. I	1	1.8 v3	310	440	310	0.98 (450)	1.015 0.774	34.24 28.53	1.5	0.75Δq
13-2	CIU sch. II	2.8	1.8	309 110	441	110	0.99 (450)	0.980 0.736	34.27 28.74	3.0	0.75Δq
13-3	CIU sch. I	1	0.05 v8	307	443	307	0.98 (450)	0.986 0.851	33.93 28.49	1.5	0.75Δq
13-4	CIU sch. I	2.8	0.05	310 111	440	111	0.98 (450)	0.956 0.814	33.41 28.14	1.5	0.75Δq
13-5	CIU sch. I	1	0.9 v4	306	444	306	0.98 (450)	0.971 0.778	34.34 27.92	1.5	0.75Δq
13-6	CIU sch. I	2.8	0.9	308 108	442	108	0.99 (450)	1.057 0.757	37.02 28.43	1.5	0.75Δq

Cycles were performed each time with constant amplitude of deviator stress q = $\sigma_1 - \sigma_3$. Details of tests' conditions are specified in Table 2.

6. LABORATORY TESTING OF THE SOIL

After saturation and consolidation were completed, the cyclic triaxial compression was started in conditions of water drainage prevented from the sample, according to the assumed testing procedure (Table 2, Fig. 6). The most important criterion for all tests division consists of amplitude *A* value and of the strain rate increase during the shearing.

Taking into account the nature of amplitude, the presented tests were carried out at its constant value (A=0.75Δq or A=0.375Δq). Such a case is most popular in nature and technology. Examples comprise the action of a storm wave, vibrations caused by an earthquake. Their impacts are obviously irregular, but even they, simplified, may be expressed as harmonic vibrations. Vibrations forced by machines operation are a similar case.

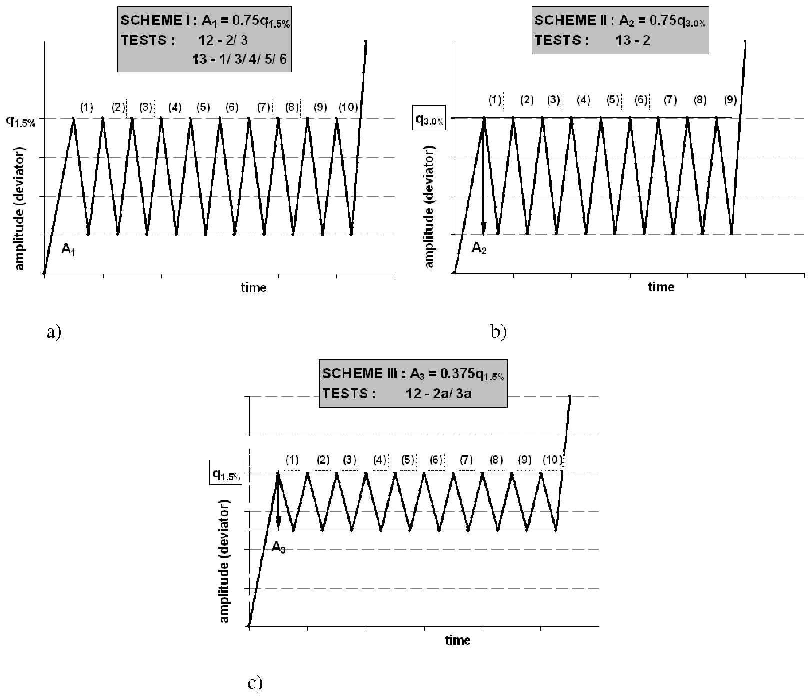

Fig. 6. Cyclic load schemes at constant amplitude: a) scheme I: A_1= 0.75$q_{1.5\%}$, 10 cycles; b) scheme II: A_2= 0.75$q_{3.0\%}$, 9 cycles; c) scheme A_3= 0.375$q_{1.5\%}$, 10 cycles.

Rys. 6. Schematy obciążenia cyklicznego przy stałej amplitudzie: a) schemat I: A_1= 0.75$q_{1.5\%}$, 10 cykli; b) schemat II: A_2= 0.75$q_{3.0\%}$, 9 cykli; c) schemat A_3= 0.375$q_{1.5\%}$, 10 cykli

Because of instrument capabilities, the starting point conditioning the beginning of cyclic load action consists of current axial strain and corresponding at specific moment deviator stress, against which the amplitude value is determined $\varepsilon_{1,unload}$ = 1.5% (in one case $\varepsilon_{1,unload}$ = 3.0%; test 13-2) was taken as characteristic axial strain.

Soil samples shearing was carried out at a constant strain rate ("*strain controlled*") equal to: v_3= 1.8 mm/h (tests 13-1 and 13-2); v_4= 0.9 mm/h (tests 13-5 and 13-6); v_6= 0.22 mm/h (tests 12-2/ 2a/ 3/ 3a) and v_8= 0.05 mm/h (tests 13-3 and 13-4). In the further part, to simplify the notation, the following symbols will be used: v3, v4, v6 and v8.

Fig. 6 shows schemes, according to each individual tests were carried out. They present the number and arrangement of cycles (n), amplitude size (A_i) including its upper and lower bound, the axial strain value initiating the cyclic loading ($\varepsilon_{1,unload}$) and corresponding deviator stress value (q_i). Assuming the denotations $q_{1.5\%}$ and $q_{3.0\%}$, i.e. the deviator stress value at vertical strain equal to ε_1 =1.5% and ε_1=3.0%, respectively, the load amplitude value was defined as: A_1= 0.75$q_{1.5\%}$, A_2= 0.75$q_{3.0\%}$ and A_3= 0.375$q_{1.5\%}$.

For the needs of the results of the experiment analysis, it is convenient to consider the load in the form of "deviatoric stress – time" diagrams. However, the response is then represented by two characteristics: "deviatoric stress – axial strain" and "deviatoric stress – average stress". This information is supplemented by "excess pore water pressure – axial strain" characteristic. The overconsolidation ratio OCR plays the role of curves parameter. As the aforementioned characteristics proceed in the full deformation range (ε_1 = 0% ÷ 15%), their graphical interpretation is possible due to external readings ($\varepsilon_1 = \varepsilon_{1,ext}$). However, due to the interest in small strains zone, their presentation was omitted in this paper. In turn, parts of graphs corresponding to cyclic loading were presented as locally enlarged, using the internal measurement of sample height and diameter change, indispensable in this case. It is worth emphasising that the strains generated for each of secondary load curves (between two sharp stress turns) fall in the small strain range, in addition, they decline with each cycle. For example, in test 12-2 during the first four cycles the following increments of axial strain were generated:

$$\varepsilon_1^{1wt} = 0.00116 > \varepsilon_1^{2wt} = 0.00098 > \varepsilon_1^{3wt} = 0.00086 > \varepsilon_1^{4wt} = 0.00072$$

Similar relationships were observed in all tests carried out. So, it is not advisable to be influenced by high values of axial strain $\varepsilon_{1,unload}$ = 1.5% and 3.0%, which determine only the beginning of the cyclic process, while the whole inference is based on phenomena occurring in the small strains zone, determined by secondary load curves of each cycle.

Figures 7 and 8 below present the influence of amplitude size (A = 0.75Δq and A = 0.375Δq) on "q - ε_1" and "q - p'" characteristics. Loops visible in Fig. 7 show a slight inclination towards higher ε_1 values. Moreover, their concentration with increasing cycles number is clearly seen. At the same time the effective paths loops run on the lower values side (Fig. 8). In the case of tests carried out at a smaller amplitude (12-2a and 12-3a; A = 0.375Δq), due to negligibly small changes, the presentation of pore water pressure increment during the variable load operation was given up. The impact of higher amplitude on Δu in tests 12-2 and 12-3 is presented in Figs. 13 and 14.

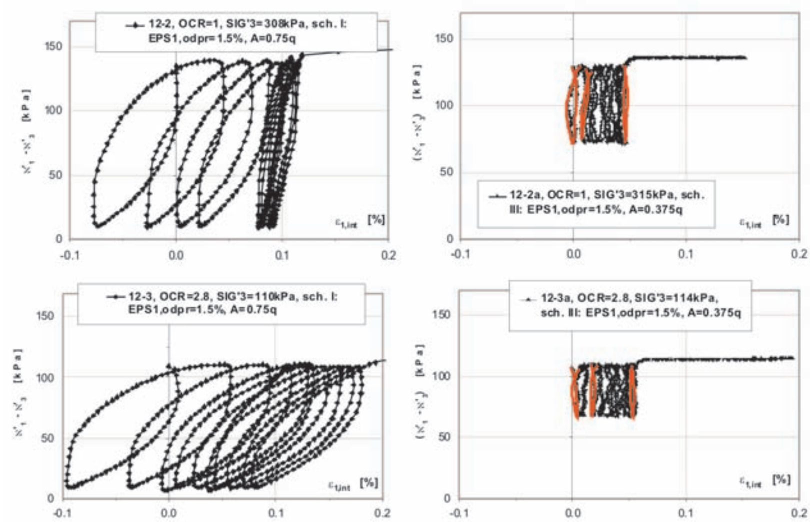

Fig. 7. Deviator stress vs. axial strain – results of tests acc. to scheme I ($\varepsilon_{1,unload}$ =1.5%, A=0.75Δq) and III ($\varepsilon_{1,unload}$ =1.5%, A=0.375Δq).
Rys. 7. Dewiator naprężenia w funkcji odkształcenia osiowego – wyniki badań wg schematu I ($\varepsilon_{1,unload}$ =1.5%, A=0.75Δq) i III ($\varepsilon_{1,unload}$ =1.5%, A=0.375Δq)

The characteristics presented further on, taking into consideration the strain rate influence, were grouped separately for normally consolidated (Figs. 9, 11 and 13) and overconsolidated soils (Figs. 10, 12 and 14). Figures 9 and 10 present deviator stress changes versus the axial strain, in the part corresponding to cyclic loading. Like in Fig. 7, loops visible there show a slight inclination towards higher ε_1 values. Moreover, the higher rate we have (v3) the more "stretched" they are. This translates into smaller strain increments ε_1 during cyclic loading at a lower rate (v8). Irrespective of the shear rate value, the loops are gradually more concentrated and get "closed" with each next cycle of unloading and secondary loading. Loops of effective stress paths observed in Figs. 11 and 12 in "q - p'" system with increasing strain rate gradually "open up", where the opening is larger for overconsolidated soils.

Further on, Tables 3 and 4 present excess pore water pressure or axial strain, respectively, generated during cyclic loading at various amplitudes and various shear rates. The data presented in Table 3 shows that at higher amplitudes there are higher

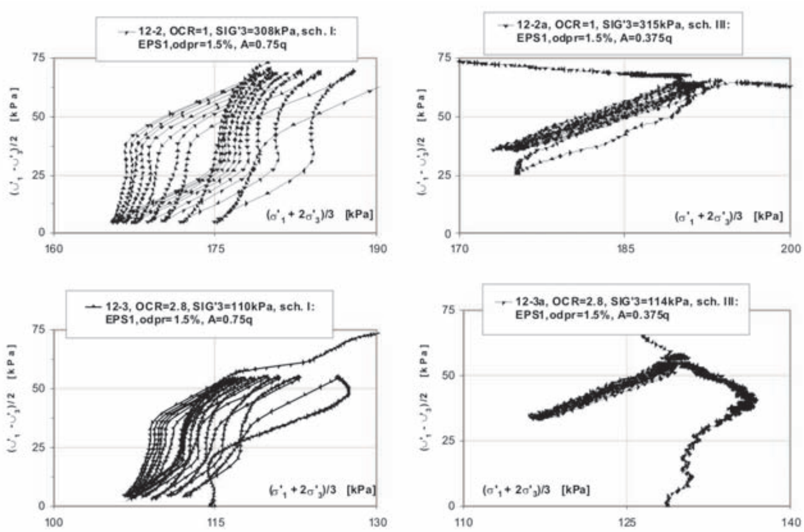

Fig. 8. Effective stress paths in tests acc. to scheme I ($\varepsilon_{1,unload}$ =1.5%, A=0.75Δq) and III ($\varepsilon_{1,unload}$ =1.5%, A=0.375Δq).

Rys. 8. Ścieżki naprężenia efektywnego w badaniach wg schematu I ($\varepsilon_{1,odpr}$ =1.5%, A=0.75Δq) i III ($\varepsilon_{1,odpr}$ =1.5%, A=0.375Δq)

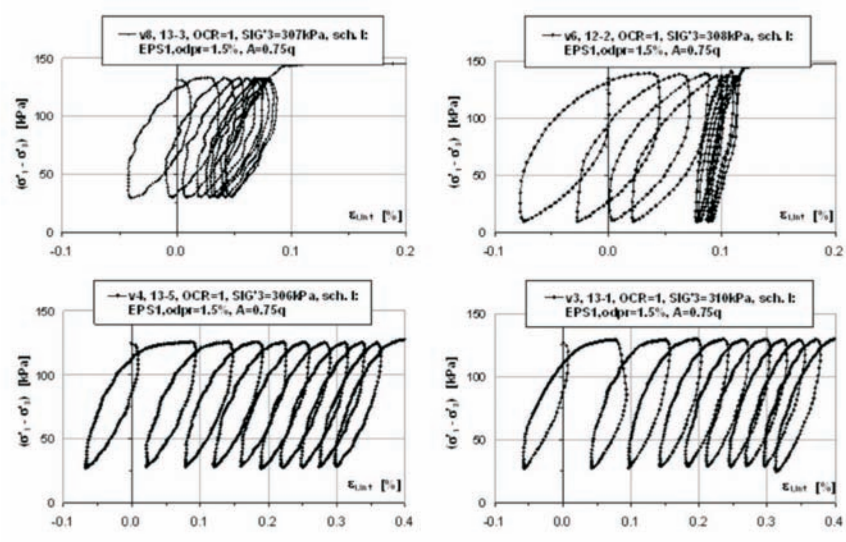

Fig. 9. Deviator stress vs. axial strain – results of NC soils shear tests at various rates (scheme I – $\varepsilon_{1,unload}$ =1.5%, A=0.75Δq).

Rys. 9. Dewiator naprężenia w funkcji odkształcenia osiowego - wyniki badań ścinania gruntów NC z różnymi prędkościami (schemat I – $\varepsilon_{1,odpr}$=1.5%, A=0.75Δq)

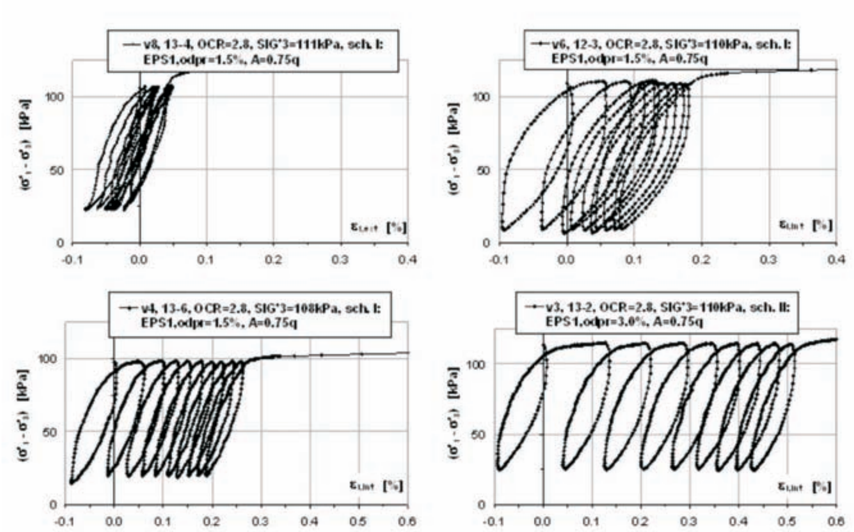

Fig. 10. Deviator stress vs. axial strain – results of OC soils shear tests at various rates (scheme I – $\varepsilon_{1,unload}$=1.5%, A=0.75Δq) and scheme II – $\varepsilon_{1,unload}$=3.0%, A=0.75Δq).

Rys. 10. Dewiator naprężenia w funkcji odkształcenia osiowego – wyniki badań ścinania gruntów OC z różnymi prędkościami (schemat I – $\varepsilon_{1,odpr}$=1.5%, A=0.75Δq) oraz schemat II – $\varepsilon_{1,odpr}$=3.0%, A=0.75Δq)

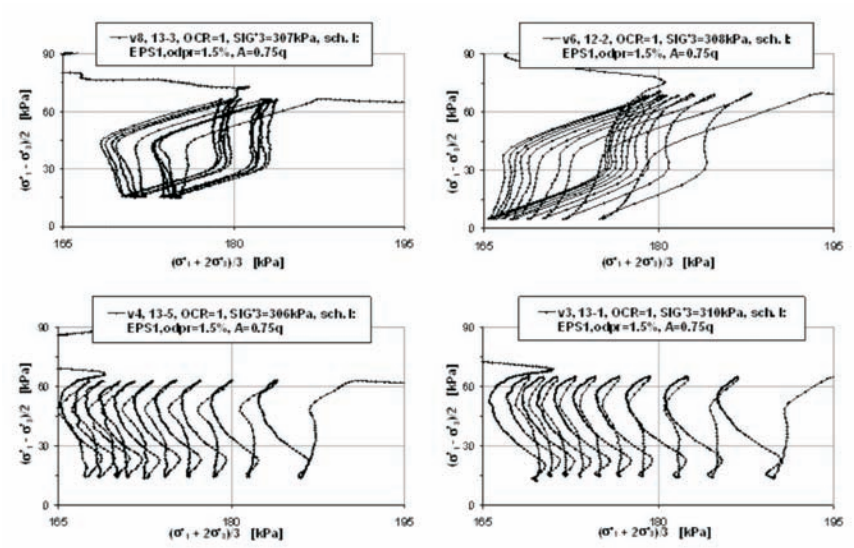

Fig. 11. Effective stress paths in NC soils shear tests at various rates (scheme Ia – $\varepsilon_{1,unload}$=1.5%, A=0.75Δq).

Rys. 11. Ścieżki naprężenia efektywnego w badaniach ścinania gruntów NC z różnymi prędkościami (schemat Ia – $\varepsilon_{1,odpr}$=1.5%, A=0.75Δq)

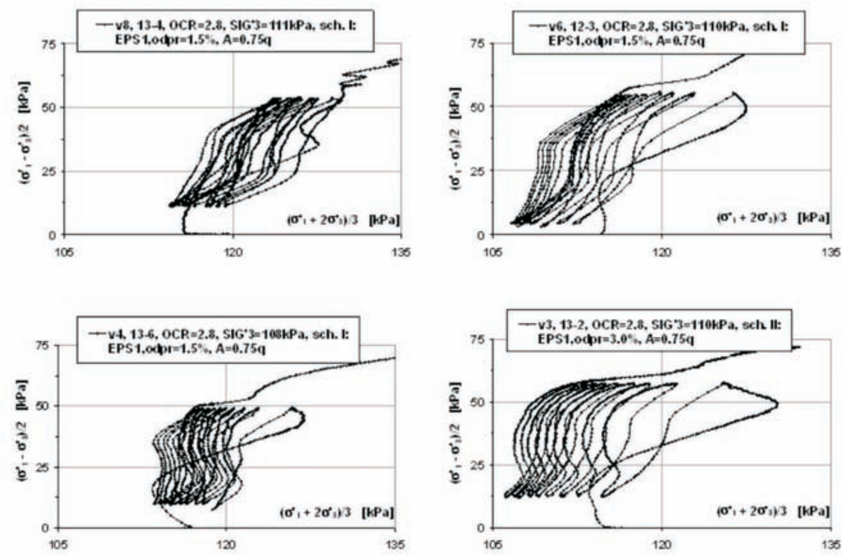

Fig. 12. Effective stress paths in OC soils shear tests at various rates (scheme I – $\varepsilon_{1,unload}$=1.5%, A=0.75Δq) and scheme II – $\varepsilon_{1,unload}$=3.0%, A=0.75Δq).

Rys. 12. Ścieżki naprężenia efektywnego w badaniach ścinania gruntów OC z różnymi prędkościami (schemat I – $\varepsilon_{1,odpr}$=1.5%, A=0.75Δq) oraz schemat II – $\varepsilon_{1,odpr}$=3.0%, A=0.75Δq)

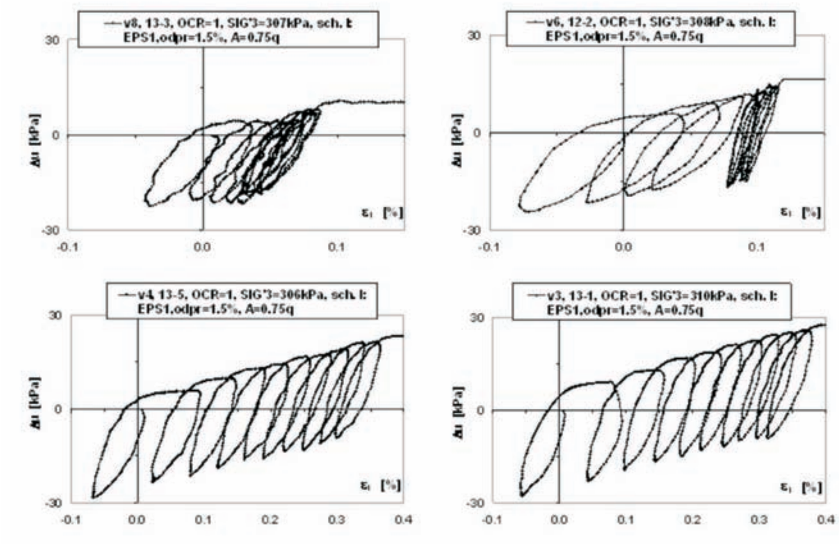

Fig. 13. Excess pore water pressure vs. axial strain – results of NC soils shear tests at various rates (scheme I – $\varepsilon_{1,unload}$=1.5%, A=0.75Δq).

Rys. 13. Nadwyżka ciśnienia wody w porach w funkcji odkształcenia osiowego – wyniki badań ścinania gruntów NC z różnymi prędkościami (schemat I – $\varepsilon_{1,odpr}$=1.5%, A=0.75Δq)

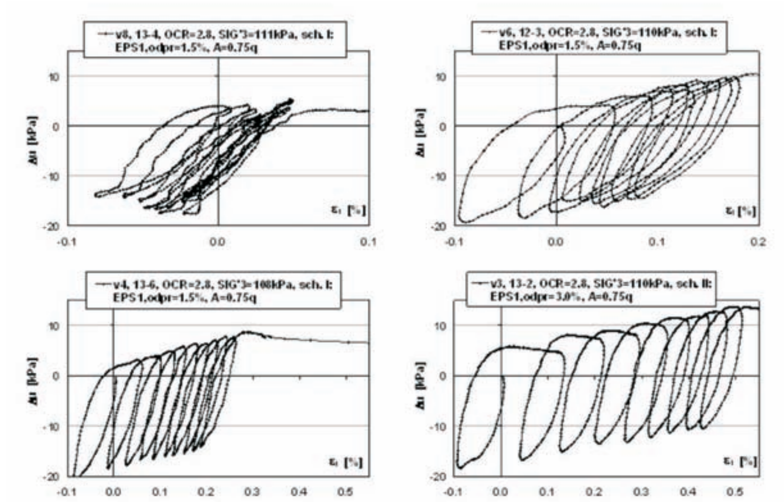

Fig. 14. Excess pore water pressure vs. axial strain – results of OC soils shear tests at various rates (scheme I – $\varepsilon_{1,unload}$=1.5%, A=0.75Δq) and scheme II – $\varepsilon_{1,unload}$=3.0%, A=0.75Δq).
Rys. 14. Nadwyżka ciśnienia wody w porach w funkcji odkształcenia osiowego – wyniki badań ścinania gruntów OC z różnymi prędkościami (schemat I – $\varepsilon_{1,odpr}$=1.5%, A=0.75Δq) oraz schemat II – $\varepsilon_{1,odpr}$=3.0%, A=0.75Δq)

Table 3

Percentage increase in pore water pressure during cyclic loading versus the sample strain rate.
Procentowy przyrost ciśnienia wody w porach podczas działania obciążenia cyklicznego w zależności od prędkości odkształcenia próbki

<table>
<tr><td colspan="7">Δu [%] (after 10 cycles)</td></tr>
<tr><td>Amplitude</td><td>Strain rate</td><td colspan="2">NC - OCR=1</td><td></td><td colspan="2">OC - OCR=2.8</td></tr>
<tr><td rowspan="9">0.75Δq</td><td rowspan="2">v8</td><td rowspan="12">σ'_3= 308 ÷ 315 kPa</td><td>13-3</td><td rowspan="6">$\varepsilon_{1,unload}$= 1.5%</td><td>13-4</td><td rowspan="12">σ'_3= 110 ÷ 114 kPa</td></tr>
<tr><td>5.6</td><td>18</td></tr>
<tr><td rowspan="2">v6</td><td>12-2</td><td>12-3</td></tr>
<tr><td>10.8</td><td>36</td></tr>
<tr><td rowspan="2">v4</td><td>13-5</td><td>13-6</td></tr>
<tr><td>15.7</td><td>40</td></tr>
<tr><td rowspan="3">v3</td><td>13-1</td><td rowspan="3"></td><td>13-2</td></tr>
<tr><td>$\varepsilon_{1,unload}$= 1.5%</td><td>$\varepsilon_{1,unload}$= 3.0%</td></tr>
<tr><td>17.8</td><td>46</td></tr>
<tr><td rowspan="3">0.375Δq</td><td rowspan="3">v6</td><td colspan="3">$\varepsilon_{1,unload}$ = 1.5%</td></tr>
<tr><td>12-2a</td><td rowspan="2"></td><td>12-3a</td></tr>
<tr><td>3.4</td><td>5.3</td></tr>
</table>

pore water pressures increments, while poorer diversification is seen for normally consolidated soils. Moreover, irrespective of the soil overconsolidation ratio, an increase in the strain rate results in higher Δu increment, where for the same rates increments are higher for overconsolidated soils. Based on the example of results presented in Table 4 it may be stated that an increase in the shear rate results in higher $\Delta\varepsilon_1$ values for all soil types (NC and OC), where the phenomenon is more noticeable for normally consolidated soils. Moreover, during cyclic loading at a lower amplitude (12–2a, $A = 0.375\Delta q$, $\Delta\varepsilon_1 = 3.3\%$) changes in ε_1 are much smaller than at amplitude $A = 0.75\Delta q$ (12–2, $\Delta\varepsilon_1 = 7.8\%$).

Table 4

Percentage increase in sample's axial strain during cyclic loading versus the shear rate.
Procentowy przyrost odkształcenia osiowego próbki podczas działania obciążenia cyklicznego w zależności od prędkości ścinania

<table>
<tr><td colspan="7">$\Delta\varepsilon_1$ **[%]** (after 10 cycles)</td></tr>
<tr><td>Amplitude</td><td>Strain rate</td><td colspan="2">**NC** - OCR=1</td><td></td><td colspan="2">**OC** - OCR=2.8</td></tr>
<tr><td rowspan="9">0.75Δq</td><td rowspan="2">**v8**</td><td rowspan="12">$\sigma'_3 = 308 \div 315$ kPa</td><td>13-3</td><td rowspan="6">$\varepsilon_{1,unload} = 1.5\%$</td><td>13-4</td><td rowspan="12">$\sigma'_3 = 110 \div 114$ kPa</td></tr>
<tr><td>**5**</td><td>**2.6**</td></tr>
<tr><td rowspan="2">**v6**</td><td>12-2</td><td>12-3</td></tr>
<tr><td>**7.8**</td><td>**11**</td></tr>
<tr><td rowspan="2">**v4**</td><td>13-5</td><td>13-6</td></tr>
<tr><td>**22.6**</td><td>**15.7**</td></tr>
<tr><td rowspan="3">**v3**</td><td>13-1</td><td rowspan="3"></td><td>13-2</td></tr>
<tr><td>$\varepsilon_{1,unload} = 1.5\%$</td><td>$\varepsilon_{1,unload} = 3.0\%$</td></tr>
<tr><td>**25**</td><td>**19.8**</td></tr>
<tr><td rowspan="3">0.375Δq</td><td rowspan="3">**v6**</td><td colspan="3">$\varepsilon_{1,unload} = 1.5\%$</td></tr>
<tr><td>12-2a</td><td rowspan="2"></td><td>12-3a</td></tr>
<tr><td>**3.3**</td><td>**3.9**</td></tr>
</table>

7. CONCLUSIONS

This paper aimed at presentation of selected properties of cohesive soil response to cyclic loading in the range of small strain, i.e. $10^{-5} \div 10^{-3}$.

The task set combined problems of cohesive soils response to repeatable loading and their behaviour at small strains. The paper considered the process in a way close to reality, starting from unloading. Experimental and theoretical grounds for the results interpretation within one cycle had been provided by fundamental JARDINE [7] paper quoted several times. However, it should be strongly emphasised here that not many of the papers on the phenomena in the range of small strains went beyond one cycle. This study, covering a larger number of repetitions, contributed to significant progress by capturing the difference in behaviour in consecutive narrow cycles, in particular the trends observed.

Because of own tests carried out, a number of regularities, based on clay from Tułowice example, were observed. The whole analysis considered the influence of such factors, as: deviator stress amplitude (A=0.75Δq or A=0.375Δq) and the shear rate (v). It was observed that at relatively large values of constant deviator stress amplitude of around 0.75 of the initial value (i.e. deviator value at the moment of starting the first unloading), the shear cycles were clearly inclined from the vertical towards higher values and the stress paths loops were oriented close to the vertical. For smaller amplitudes, of around 0.375 of deviator stress initial value, the situation was opposite (Fig. 7 and 8). The above feature should be related to a different degree of development of plastic deformation, the larger it was, the larger deviator stress amplitude was, and ultimately the different degree of physical non-linearity progression in final sections of consecutive cycles occurred. The accompanying increase in pore water pressure depended on the overconsolidation ratio and the amplitude size. For normally consolidated soils it amounted, for example, to 9% at A = 0.75Δq and 3.3% at A = 0.375Δq. For poorly overconsolidated soils this amounted to 30% and 5.3%, respectively.

At the same time it was noticed that a 36-time increase in the strain rate generated around 3 times larger Δu increments, what was understandable in the case possibilities of lack of water drainage from the sample. If the excessive generation of pore water pressure was considered as one of the reasons for the loss of soil shear strength, then in the case of clay from Tułowice it could be stated that the increase (higher for OC soils) with progressing cycles number was small, therefore it did not imply material's weakness. In addition, after stopping the cyclic loading, it went down to the level prior to its start. Such conclusion basically requires confirmation by further studies, for larger numbers of cycles. However, it is worth noting that if with each consecutive cycle the Δu increase was smaller and the total excess was generally not large (max. 46%), then the conclusion in question remains valid irrespective of the cycles' number.

Similar relations occurred during the increase in the axial strain during cyclic loading , which at the same levels of lateral effective stress were greater for normally consolidated than for overconsolidated soils, while at smaller amplitude the differences were less important. At the same time, with each next cycle the increments observed were smaller and smaller, even assuming the value equal to zero for the tenth cycle.

Both situations should be clearly distinguished when comparing this property with the results of Jardine [7] investigations into the behaviour of soil subject to monotonic loading in the range of small strains. In Jardine investigations an increase in strain responds to the increasing deviator stress (Fig. 2). It is accompanied by the development of irreversible strain with implications in the form of widening and opening of a narrow closed hysteresis loop, and its shifting in the "deviator stress – non-dilatational (and also axial) strain" space. In the studies presented here, a cyclic process of constant amplitude occurs, which with its development de facto goes away from the primary envelope. A visible effect is the phenomenon of closing and stabilising of initially open and shifting loops, opposite to that observed by Jardine. This applies, of course, to sufficiently large deviator stress amplitudes. For smaller amplitudes the loops are initially closed and stable.

The issues presented in the paper are part of a wide cycle of studies on the behaviour of cohesive soils under cyclic loading in the range of small and moderate strains and studies related to mathematical modelling (Jastrzębska and Łupieżowiec [10 ÷ 12] Jastrzębska and Sternik[13]).

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WPŁYW WYBRANYCH PARAMETRÓW PROCESU CYKLICZNEGO NA CHARAKTERYSTYKI ŚCINANIA GRUNTÓW SPOISTYCH W ZAKRESIE MAŁYCH ODKSZTAŁCEŃ

S t r e s z c z e n i e

Przedmiotem dociekań i badań przedstawionych w artykule jest odpowiedź gruntu spoistego w strefie małych odkształceń ($10^{-5} \div 10^{-3}$) na zadawane obciążenia cykliczne. Postawione zadanie łączy problemy reakcji gruntów spoistych na obciążenia powtarzalne i ich zachowania w zakresie małych odkształceń. Każdy z osobna jest przedmiotem wielu dociekań w ostatnich dekadach, jednakże niewiele z prac wychodziło poza jeden cykl. W związku z tym na potrzeby pracy wykonano badania cyklicznego ścinania bez odpływu wody w aparacie trójosiowego ściskania na jednorodnym materiale – kaolinie z Tułowic, obejmujące większą liczbę powtórzeń. Badania prowadzone były na zmodernizowanym stanowisku badawczym, umożliwiającym pełne nasycenie próbek metodą „*back pressure*" oraz precyzyjny, wewnątrzkomorowy pomiar małych odkształceń za pomocą bezkontaktowych czujników mikroprzemieszczeń. Przy zachowaniu stałej amplitudy dewiatora naprężenia, dla gruntów NC i OC, badano wpływ prędkości odkształcenia na charakterystyki materiałowe „dewiator naprężenia (zmiana ciśnienia wody w porach) – odkształcenie osiowe" i ścieżki naprężenia efektywnego. Analizując otrzymane wyniki stwierdza się inne w stosunku do JARDINE'A [7, 8] zjawisko zamykania się i stabilizacji otwartych początkowo i przemieszczających się pętli. Obserwowane przyrosty odkształcenia osiowego w trakcie działania obciążenia cyklicznego, przy tych samych poziomach bocznego naprężenia efektywnego są większe dla gruntów normalnie skonsolidowanych niż prekonsolidowanych. Jednocześnie z każdym kolejnym cyklem przyrosty te są coraz mniejsze, przyjmując nawet wartość równą zero dla cyklu dziesiątego. Podobne relacje zachodzą podczas przyrostu ciśnienia wody w porach w trakcie działania obciążenia cyklicznego. Dla zadanej liczby cykli n=10 są one na tyle niewielkie – max. 46% (i malejące z każdym kolejnym cyklem), że nie prowadzą do osłabienia materiału. Uwzględniając trend malejących przyrostów Δu można uznać słuszność przedmiotowego wniosku niezależnie od liczby cykli.

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