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THE INFLUENCE OF OVERCONSOLIDATION RATIO ON THE "$G_S - \varepsilon_1$" DEPENDENCE FOR CYCLIC LOADING OF COHESIVE SOILS IN THE RANGE OF SMALL STRAINS

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Abstract: The subject of the paper comprises testing of a cohesive soil response to cyclic loading applied in the range of small strains ($10^{-5}\div10^{-3}$). To this end, tests of undrained cyclic shear in a triaxial apparatus were carried out on homogeneous material – kaoline from Tułowice. The tests were carried out on a modernised test bed, enabling full saturation of specimens using the *back pressure* method as well as a precise, local measurement of small strains by means of contactless microdisplacement sensors. While maintaining a constant deformation rate, during 10 cycles of unloading and reloading, the influence of overconsolidation ratio (*OCR*) at various levels of the deviator stress amplitude ($A_1 = 0.75\Delta q$ and $A_2 = 0.375\Delta q$) on the "$G_s - \varepsilon_1$" stiffness characteristics was investigated. When analysing the results obtained it has been found that the overconsolidated soils feature higher G_s values as compared with normally consolidated soils, where the differences are more significant for higher deviator stress amplitudes. In addition, the deviator stress amplitude increase results in a decline in the shear modulus G_s. At the same time soil strengthening in consecutive cycles is observed.

NOMENCLATURE

A	– amplitude (deviator stress amplitude), kPa,
n	– number of cycles,
G_s	– secant shear modulus, Mpa,
$\varepsilon_{1,\text{unload}}$ (EPS1,odpr)	– axial strain initiating the cyclic load operation, %,
σ'_3	– effective lateral stress, kPa,
BS	– bounding surface,
CIU	– testing with isotropic consolidation and shearing without drainage,
CSR	– cyclic stress ratio,
NC, OC	– normally consolidated soil, overconsolidated soil,

1. INTRODUCTION

It is characteristic and surprising at the same time that a general advancement of studies on small strains does not translate into the case of cyclic loading. It is possible to say that the recognition and description of phenomena in the small strains range were limited generally to one "loading–unloading" cycle, even in such a representative publication as Jardine's paper [7]. Having in mind that the issues related to strains smaller than 10^{-3} and to cyclic loading were considered rather separately, it seems right that effects of their combined occurrence will be interesting from the point of view of geotechnical designing and contracting.

2. PHENOMENA IN THE RANGE OF SMALL STRAINS

Issues related to very small and small strains (of the order of 10^{-3} and less) date back with their roots to the beginning of the 1970's (e.g. HARDIN and DRNEVICH [6], ATKINSON and SALLFORS [4], BURLAND [5], JARDINE [7]).

Despite anisotropy the behaviour of soils in the range of small strains is relatively simple. The importance of research in this area is related to identification of initial "soil shear modulus and compression modulus–strain" characteristics. The environment's response to loading in the small strain range (from around 10^{-5} to around 10^{-3}) turns out to be far more complex. In quantitative terms it is characterised by a slump in isotropic and deviator stiffness. In the light of fundamental Jardine's theoretical investigations and experimental tests [7], [8] the reasons of this phenomenon should be sought in quantitative changes occurring in the small strain range. Linear elasticity transforms first into an elastic hysteresis. The "stress–strain" characteristics in the "loading–unloading" cycle have the form of closed loops. The soil shows here a high sensitivity to the course of loading path, preceding the current state (ATKINSON et al. [3]). An increasing deformation is accompanied by a decay of this sensitivity and wider and wider opening of the hysteresis loop and transformation into plasticity. As can be seen, the shift from linear elasticity to plasticity contains a complicated intermediate stage, which occurs just in the small strain range (in zone II and at the beginning of III, figure 1).

However, it is the application aspect that decides about the key importance of small strain issues in contemporary geotechnics. Settlement forecast without considering a sudden decrease of stiffness happens to be drastically overestimated (KRIEGEL and WEISNER [15], BURLAND, [5]). In fact, the strain decay with depth turns out to be "quicker", in particular, when the pressure from the foundation on the ground is relatively small, which is usually the case for buildings of extensive plan.

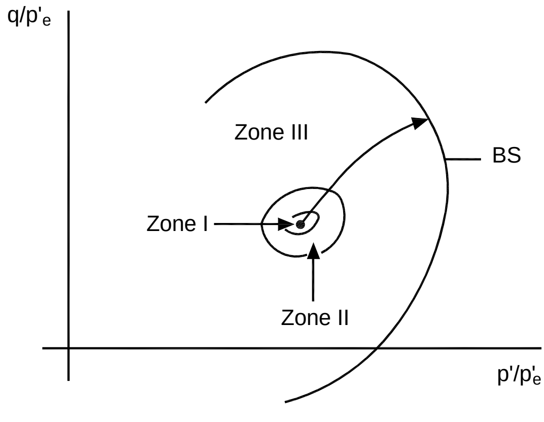


Fig. 1. Zones of the stress space differing in nature of deformations

3. COHESIVE SOILS BEHAVIOUR UNDER CYCLIC LOADING

Cyclic loading occurs commonly in technology and nature. Although each source has its own characteristic, the loading and unloading cycles alternating in time are their common denominator. A more precise description refers to the occurrence of multiple changes in the load path direction by 180°. The location of cyclic process beginning in the "stress–strain" system is a crucial element. In some papers devoted to this issue this state begins with a loading curve. In this approach it is usual to start the process in a natural state, i.e. at the origin of coordinates $(q, \varepsilon_s) = \{0,0\}$.

In an alternative, closer to reality approach, each cyclic process in the soil environment is preceded by a monotonic trajectory of primary loading, in other words, normal consolidation of the environment. The beginning of the process is the upper bound of the first loop. The process starts with unloading (a decline in stress intensity). In the next cycles the beginning of each consecutive cycle is important at changes in the process course. The cycle deviator stress amplitude is the second identifier of repeatable load in each cycle. In this case the soil response to the applied variable load consists in drawing away individual loops.

Most studies related to soil behaviour under the influence of cyclic loading are devoted to sands. The information related to cohesive soils originated at the beginning of the 1970's (e.g. ANDERSEN [1], ANDERSEN and LAURITZSEN [2]). Another excellent paper is the state-of-the-art report by WOOD [20].

There is an impression that within the focus of cyclic processes researchers' interest comprises today boundary problems in conditions of water drainage prevented and primarily the soil liquefaction resulting from pore pressure accumulation. There are also attempts to explain the influence of cyclic loading on cohesive soil behaviour as well as the influence of such factors as overconsolidation, the strain amplitude size, number and frequency of cycles. Speaking about the cyclic loading method people usually think of cycles' location against the initial static stress, representing the action of a steady load. Cases of pulsating (cycles situated on one side of static stress) and oscillating (cycles on both sides of static stress) cyclic loading are distinguished here.

The amplitude of cyclic loading is a basic parameter of testing, generally deciding about soil destruction. There is no doubt (acc. to SANGREY and FRANCE [18]) that if it is large enough (cyclic stress ratio $CSR > 0.7$), then irrespective of other factors the soil must get liquefied (in undrained conditions). Large amplitude causes a sudden increase in pore pressure, which results in a sudden increase in shear strains. The determination of the limit between soil stabilisation and liquefaction is a problem. Especially, as apart from the soil type it depends on many other factors, such as *OCR* or loading rate. Referring to stiffness characteristics it is possible to state that the influence of amplitude on the shear modulus is varied. At small amplitudes ($CSR < 0.3$) modulus G increases slightly or does not change its value during consecutive cycles.

Larger amplitude results in decreasing the shear moduli with cycles (soil degradation of parameters).

With regard to the overconsolitation ratio (*OCR*) it seems reasonable to assess the cyclic loading influence via the analysis of generated pore pressure (e.g. as suggested by MATASOVIC and VUCETIC [16]). In the case of overconsolidated soils and basically at $OCR > 2$, the originating negative pressure causes that we need much larger number of cycles to obtain an identical pressure level (responsible for soil liquefaction) as for the same soil, but normally consolidated (e.g. MATSUI et al. [17]).

4. DESCRIPTION OF LABORATORY EQUIPMENT

A conventional triaxial apparatus has been 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 are 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 deaerated 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 are 2.0 mm and 0.01%, respectively.

• External ε_1 using the external displacement gauge fixed on the loading piston.

Lateral strain ε_3 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. PREPARATION OF MATERIAL AND SAMPLES FOR TRIAXIAL TEST

The material used in this study comes 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 the cases the samples for triaxial tests were made on a soil paste of $w \approx 50\%$ water content (which makes around $1.2w_L$), which was initially consolidated at isotropic pressure equal to 80 kPa. 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 fear of losing its shape during the preparation and placing in the test cell. Finally, all triaxial tests were carried out on samples of 50 mm in diameter and 100 mm in height. Each sample was saturated. At first they were flushed with deaerated water. Thereafter a high back pressure was applied. The Skempton's B-values obtained were greater than 0.95.

Then, the specimens 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/ 2a/ 3/ 3a/ 4/ 7; 114 kPa for test no. 12-3b and 29 kPa for test no. 12-6.

Table 1

Values of some physical properties and classification characteristics of Tułowice kaolin (Jastrzębska [9])

Specific gravity	G_s	t/m^3	2.637
Natural water content	w_n	%	20.7
Liquid limit	w_L	%	42.2
Plastic limit	w_p	%	20.0
Plasticity index	I_p	%	22.2
Liquidity index	I_L	–	0.03
Skempton's coefficient	A	–	0.52–0.6
Void ratio	e	–	0.886–1.098
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

Next, two of the chosen soil samples were unloaded to effective pressures p'_0 of about 110 kPa (tests 12-3 and 12-3a) and two next soil samples to 28 kPa (tests 12-4 and 12-7). This value corresponds to the overconsolidation ratio equal to $OCR = p'_c / p'_0 =$ 2.8, 11 and 12.8. After isotropic consolidation undrained tests were carried out.

Monotonic loading had been continued until the value of axial strain $\varepsilon_{1,\text{unload}} \approx$ 1.5% was reached. Then each loading cycle comprised 10 cycles. Cycles were performed in each case with constant deviator stress amplitude. Details of the tests conditions are specified in table 2.

6. LABORATORY TESTING OF SOIL

After completing saturation and consolidation, cyclic triaxial test was started in conditions of water drainage prevented from the sample, according to the assumed testing procedure (table 2, figure 2). The value of overconsolidation ratio (*OCR*) is the most significant criterion for all tests division. In addition, the way in which the constant deviator stress amplitude value ($A = 0.75\Delta q$ and $A = 0.375\Delta q$) affects the obtained secant shear moduli has been analysed.

Because of instrument capabilities, the starting point – conditioning the beginning of cyclic load action – consists of current axial strain and corresponding at specific moment stress deviator, against which the amplitude value is determined. $\varepsilon_{1,\mathrm{unload}}$ = 1.5% was taken as the characteristic value of axial strain.

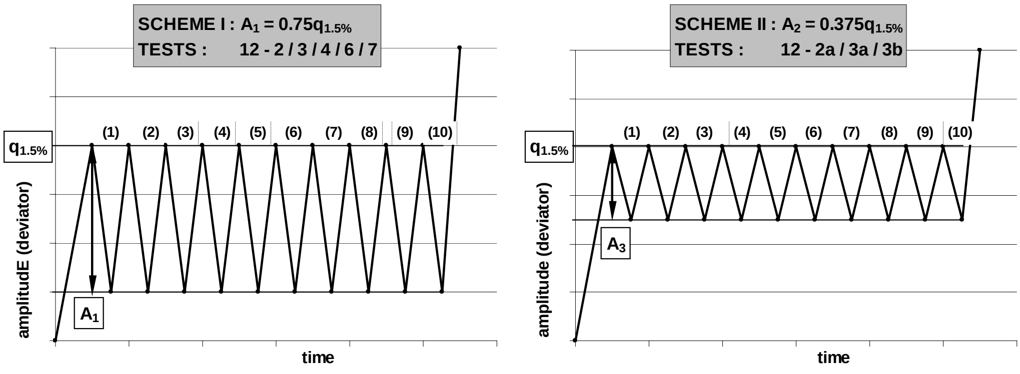
Soil samples shear was carried out at a constant strain rate ("strain controlled") equal to v_6 = 0.22 mm/h.

Table 2

Characteristics of tests carried out within the study

Symbol of test	Type of test	Overconsolidation ratio	Axial strain rate	Lateral pressure (consolidation)	Pore pressure (consolidation)	Lateral pressure (shearing)	Skempton's 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	$\varepsilon_{1,\mathrm{unload}}$	*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	$0.75\Delta q$
12-2a	CIU sch. II	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	0.75q
12-3a	CIU sch. II	2.8	0.22	314 114	436	114	0.97 (450)	0.904 0.714	32.66 26.18	1.5	0.375q
12-3b	CIU sch. II	1	0.22	114	435	114	0.95 (450)	0.907 0.799	33.16 30.31	1.5	0.375q
12-4	CIU sch. I	11	0.22	308 28	442	28	0.97 (450)	1.010 0.856	36.91 31.07	1.5	0.75q
12-6	CIU sch. I	1	0.22	29	441	29	0.97 (450)	1.021 1.003	35.94 34.57	1.5	0.75q
12-7	CIU sch. I	12.8	0.22	358 28	392	28	0.99 (450)	1.054 0.888	36.13 30.53	1.5	0.75q

Figure 2 shows schemes according to which individual tests were carried out. They present the number and arrangement of cycles (n), amplitude size (A_i) including its upper and lower bounds, the axial strain value initiating the cyclic loading ($\varepsilon_{1,\mathrm{unload}}$) and corresponding stress deviator value (q_i). Assuming denotation $q_{1.5\%}$, i.e. the stress deviator value at vertical strain equal to $\varepsilon_1 = 1.5\%$, the load deviator stress amplitude value was defined as: $A_1 = 0.75q_{1.5\%}$ and $A_2 = 0.375q_{1.5\%}$.



a) b)

Fig. 2. Cyclic load schemes at constant amplitude:
a) scheme I: $A_1 = 0.75q_{1.5\%}$, 10 cycles; b) scheme II: $A_2 = 0.375q_{1.5\%}$

Triaxial tests carried out within the study apply to cohesive soil behaviour under repeatable loading, which in each cycle cause small strains falling within a narrow range of $0 \div 10^{-3}$. The results of monotonic triaxial tests with local measurements of small strains within one cycle and within the range between two sharp turns of the stress, in accordance with zone theory by JARDINE [7], were used to determine stiffness changes within any cycle. The secant shear modulus G_s versus axial strain (maximum main strain ε_1 within any cycle) was determined in a local strain system, counted from the beginning of specific cycle.

For the sake of transparency of the analysis performed, tables 3 and 4 show the obtained shear modulus values for individual curves of secondary loading. To this end characteristic, from the point of view of stiffness distribution, places of G_s estimation were selected, i.e. $\varepsilon_1 = 0.00005$, 0.0001 and 0.001.

For the sake of clarity, the conclusions presented below refer to averaged maximum shear moduli values, determined from individual cycles of specific test at axial strain $\varepsilon_1 = 0.00005$.

The analysis of the influence of overconsolidation ratio (*OCR*) on the shear modulus G_s has shown that it was consistent with expectations. The overconsolidated soils feature higher G_s values, regardless of the amplitude size. It is worth noticing that the differences between the obtained shear modulus values, at gradually increasing overconsolidation ratio, are smaller and smaller. This is confirmed by tests (12-3b and 12-3a) as well as (12-6, 12-3 and 12-7). The above comments may be written as follows:

$$\underbrace{(\varepsilon_{1,\text{unload}} = 1.5\%,\ \sigma_3' = 114\ \text{kPa},\ A = 0.375\Delta q)}_{(12\text{-}3b)\,\text{and}\,(12\text{-}3a)} \rightarrow G_{OCR=1} = 380\ \text{MPa} < G_{OCR=2.8} = 700\ \text{MPa},$$

$$\underbrace{(\varepsilon_{1,\text{unload}} = 1.5\%,\ \sigma_3' = 28\ \text{kPa},\ A = 0.75\Delta q)}_{(12\text{-}6),\ (12\text{-}4)\,\text{and}\,(12\text{-}7)} \rightarrow G_{OCR=1} = 130\ \text{MPa} < G_{OCR=11\ \text{and}\ 12.8} \approx 300\ \text{MPa}.$$

Table 3

Specification of shear moduli G_s values during triaxial tests:
one series – 10 cycles at deviator stress amplitude $A = 0.75\Delta q$

Number of cycle	G_s [MPa]									Number of cycle
	12-6			12-4			12-7			
	$OCR = 1$, $\sigma_3' = 29$ kPa, v6; $\varepsilon_{1,\text{unload}} = 1.5\%$, $A = 0.75\Delta q$			$OCR = 11$, $\sigma_3' = 28$ kPa, v6; $\varepsilon_{1,\text{unload}} = 1.5\%$, $A = 0.75\Delta q$			$OCR = 12.8$, $\sigma_3' = 28$ kPa, v6; $\varepsilon_{1,\text{unload}} = 1.5\%$, $A = 0.75\Delta q$			
	strain level ε_1 [–]									
	0.00005	0.0001	0.001	0.00005	0.0001	0.001	0.00005	0.0001	0.001	
1wt	180	90	–	321	160		300	170	–	1wt
2wt	130	75	–	320	170	–	300	165	–	2wt
3wt	120	65	–	300	170	–	290	160	–	3wt
4wt	130	75	–	300	165	–	300	160	–	4wt
5wt	132	75	–	300	170	–	290	160	–	5wt
6wt	130	75	–	320	170	–	290	150	–	6wt
7wt	130	75	–	300	170	–	270	150	–	7wt
8wt	135	80	–	280	160	–	280	170	–	8wt
9wt	130	75	–	314	170	–	290	160	–	9wt
10wt	135	80	10	300	165	25	300	170	25	10wt

Number of cycle	12 - 2			12 - 3			Number of cycle
	$OCR = 1$, $\sigma_3' = 308$ kPa, v6; $\varepsilon_{1,\text{unload}} = 1.5\%$, $A = 0.75\Delta q$			$OCR = 2.8$, $\sigma_3' = 110$ kPa, v6; $\varepsilon_{1,\text{unload}} = 1.5\%$, $A = 0.75\Delta q$			
	strain level ε_1 [–]						
	0.00005	0.0001	0.001	0.00005	0.0001	0.001	
1wt	407	204	47	311	168	35	1wt
2wt	412	217	–	300	182	35	2wt
3wt	430	230	–	280	177	36	3wt
4wt	435	240	–	280	162	–	4wt
5wt	450	353	–	300	143	35	5wt
6wt	415	303	–	–	150	35	6wt
7wt	422	300	–	–	140	34	7wt
8wt	496	330	–	339	180	35	8wt
9wt	520	320	–	–	–	35	9wt
10wt	530	340	48	327	180	37	10wt

Table 4

Specification of shear moduli G_s values during triaxial tests:
one series – 10 cycles at deviator stress amplitude $A = 0.375\Delta q$

Number of cycle	G_s [MPa]									Number of cycle
	12-2a			12-3b			12-3a			
	$OCR = 1$, $\sigma_3' = 315$ kPa, v6 $\varepsilon_{1,\mathrm{unload}} = 1.5\%$, $A = 0.375\Delta q$			$OCR = 1$, $\sigma_3' = 114$ kPa, v6 $\varepsilon_{1,\mathrm{unload}} = 1.5\%$, $A = 0.375\Delta q$			$OCR = 2.8$, $\sigma_3' = 114$ kPa, v6 $\varepsilon_{1,\mathrm{unload}} = 1.5\%$, $A = 0.375\Delta q$			
	strain level ε_1 [–]									
	0.00005	0.0001	0.001	0.00005	0.0001	0.001	0.00005	0.0001	0.001	
1wt	750	400		380	–	–	690	340	38	1wt
2wt	–	–	–	380	–	–	700	–	–	2wt
3wt	840	–	–	380	–	–	700	–	–	3wt
4wt	830	–	–	380	–	–	700	–	–	4wt
5wt	–	–	–	380	–	–	700	–	–	5wt
6wt	–	–	–	380	–	–	680	350	–	6wt
7wt	830	–	–	–	–	–	700	–	–	7wt
8wt	840	–	–	–	–	–	–	–	–	8wt
9wt	830	–	–	–	–	–	700	–	–	9wt
10wt	840	450	45	400	200	20	700	350	38	10wt

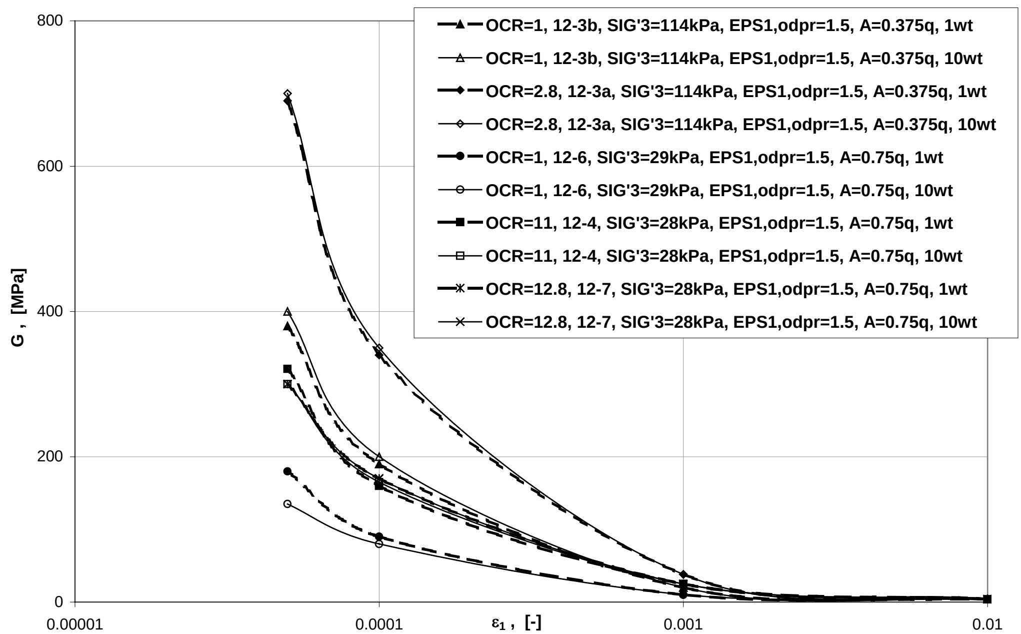


Fig. 3. The influence of overconsolidation ratio *OCR* on the "shear modulus–axial strain" characteristic for NC and OC soils, for the first and the tenth cycle in the series (acc. to data from tables 3 and 4)

Moreover, it has been noticed that in similar conditions of tests (12-3a) and 12-3, featuring the following values: OCR = 2.8, $\varepsilon_{1,\text{unload}}$ = 1.5% and $\sigma'_3 \approx$ 110 kPa, the obtained shear moduli G_s are nearly two and a half times higher for the smaller deviator stress amplitude ($A = 0.375\Delta q$). Similar relationships have been observed for normally consolidated soils (cf. tests 12-2 and 12-2a). This could be written as follows:

$$\underbrace{(OCR = 2.8; \sigma'_3 = 114\ \text{kPa})}_{(12\text{-}3\text{a})\ \text{and}\ (12\text{-}3)} \to G_{A_2 = 0.375\Delta q} = 700\ \text{MPa} > G_{A_1 = 0.75\Delta q} = 305\ \text{MPa},$$

$$\underbrace{(OCR = 1; \sigma'_3 \approx 310\ \text{kPa})}_{(12\text{-}2\text{a})\ \text{and}\ (12\text{-}2)} \to G_{A_2 = 0.375\Delta q} = 820\ \text{MPa} > G_{A_1 = 0.75\Delta q} = 340\ \text{MPa}.$$

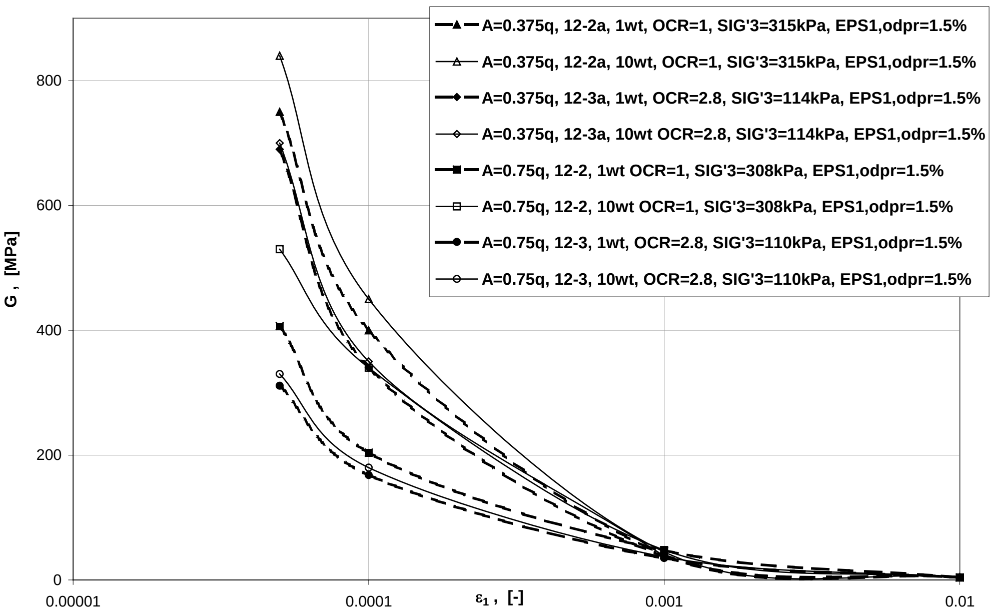


Fig. 4. The influence of deviator stress amplitude size ($A = 0.375\Delta q$ or $A = 0.75\Delta q$) on the "shear modulus–axial strain" characteristic for NC and OC soils, for the first and the tenth cycle in the series (acc. to data in tables 3 and 4)

7. CONCLUSIONS

This paper aimed at investigation of the overconsolidation ratio (OCR) influence on the secant shear modulus G_s of cohesive soil cyclically loaded in the range of small strains dependence on the maximum main strain. The paper has considered the process in a way close to reality, starting from unloading. In view of simultaneous focus of attention on the small strains range, narrow cycles, each of which fell within those limits, were considered especially interesting.

Experimental and theoretical grounds for interpretation of the results within one cycle have been provided by fundamental JARDINE's paper [7] quoted several times. The tests presented, covering a larger number of repetitions, have contributed to significant progress by capturing the difference in behaviour in consecutive narrow cycles, in particular the trends observed.

During the tests carried out by the author a number of regularities, based on clay from Tułowice example, have been observed. The influence of overconsolidation ratio (*OCR*) at various levels of deviator stress amplitude ($A = 0.75\Delta q$ and $A = 0.375\Delta q$) was considered in the entire analysis.

Within classical approach to phenomena in the range of small strains the interpretation of tests was related to the evaluation of shear modulus dependence on the axial strain, where cyclic process indices play the role of parameters. The overconsolidation ratio was the leading variable in this paper. It has been noticed that overconsolidated soils feature higher shear moduli than normally consolidated soils. In the same testing conditions the OC soils featured 1.8 times higher shear modulus in the case of the smaller deviator stress amplitude ($A = 0.375\Delta q$). At the higher deviator stress amplitude ($A = 0.75\Delta q$) the relation was 2.4 times. At the same time the increase in G_s modulus value translates into a steeper course of the shear curve (figure 3). This results in a higher positive jump of stiffness and its sharper reduction. At the same time, at a twofold increase in the deviator stress amplitude the shear modulus value decreases 2.5 times, both for NC and OC soils, although the accompanying increments of axial strains are smaller (figure 4).

In summary, it has been stated that the tests carried out have shown some properties of cohesive soil subject to cyclic loading in a way raising no doubt.

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 [11]–[13], JASTRZĘBSKA and STERNIK [14]). In turn, another publication of the author [10] presents the course of material characteristics versus the strain amplitude and rate. This paper and [10] are complementary.

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