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CYCLIC STRENGTH OF AN ORGANIC SOIL IN TERMS OF THE STATE PARAMETER

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ABSTRACT

The paper investigates the cyclic strength of an organic soil. The "organic" material was carbon made from carbonized wood fully pulverized. Oedometer tests illustrated the correlation between the compressibility and initial density. The critical state line was obtained from triaxial undrained tests. Cyclic undrained triaxial tests were performed at three different densities and three different consolidation stresses $\sigma'_{c\text{-}o}$. As the void ratio increases, the cyclic strength at similar $\sigma'_{c\text{-}o}$ decreases with a progressively smaller rate. For similar void ratio, the cyclic strength decreases as $\sigma'_{c\text{-}o}$ increases. As the void ratio increases, the effect of $\sigma'_{c\text{-}o}$ on the cyclic strength is less pronounced. The relationship between the state parameter and the cyclic strength shows a decrease with a progressively smaller rate and a strong correlation coefficient.

Keywords: liquefaction, carbon, state parameter, organic soil, laboratory testing.

INTRODUCTION

Organic deposits start as accumulations of vegetable matter in lakes or shallow seas. They undergo changes due to decomposition by bacteria and pass through the phases of bog, humus, turf, peat and lignite or other forms of coal. The first phases are transient but coal is chemically stable under normal temperatures. The whole process takes a few tens of millions of years (Kirkaldy, 1963). This paper deals with the chemically stable variety of organic material.

The stability of organic soils is a major concern at some regions. For example, in Greece, the stability of open pit lignite mines is of major concern because most of the electric power comes from thermal plants. Accumulated volumes of tailings are of the order of hundreds of millions of cubic meters. The composition and descriptive properties of tailings from coal mines in the U.S.A. and U.K. are presented by Vick (1983).

The problem of assessing the geotechnical properties and strength of deposits of lignite is treated by Stamatopoulos and Kotzias (1981), for the case of an open pit mine near the thermoelectric station of Megalopolis, Greece. Yet, the cyclic strength was not measured.

If a general study of the properties of organic deposits is to be addressed, standardization is needed. In the present study the organic soil is made from carbonized wood fully pulverized in order that all the material to pass through the No 200 sieve. The density of grains, maximum and minimum void ratio, water content and compressibility of the obtained carbon are similar to those of the lignite at Megalopolis.

The state parameter (Been and Jefferies, 1985) has been correlated to a number of soil parameters measured in the laboratory, such as the peak friction angle. Recently, the cyclic soil strength has also been correlated to the state parameter (Chen and Liao, 1999, Boulonger, 2003, Stamatopoulos et al,

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2004). Such correlations have the advantage that they have a strong theoretical basis and that they describe with a single parameter the effect of both the confining stress and the void ratio on the cyclic soil strength.

The paper studies the cyclic strength of the organic soil described above and its relation to the state parameter. For this purpose, an elaborate testing program of cyclic triaxial tests at different cyclic stress ratios, initial void ratios and initial confining stresses was performed. Undrained triaxial tests were also performed at different initial densities to establish the critical state line and thus estimate the state parameter at different states of void ratio and confining stress.

CHARACTERISTICS OF THE MATERIAL

Carbon used was derived from manufactured carbonized wood commercially available because it is used for outdoor barbecuing. It was fully pulverized in order that all the material passes through the No 200 sieve. It is chemically stable with liquid limit 110%, plasticity index zero, specific gravity 1.62 and carbon content (by ASTM D2974) 95%. Figure 1a gives the appearance of the carbon saturated with water. Grain size distribution of the carbon is given in figure 1b.

Comparisons between the properties of the carbon and lignite from the power plant of Megalopolis, Greece, reported by Stamatopoulos and Kotzias (1981) are shown in Table 1. Specific gravity of grains, water content, liquid limit and plasticity index of the carbon fall within the range of the values of lignite.

Maximum and minimum density tests were carried out firstly on dry constituents. Then, they were repeated after increasing the water content to 30%. Results are shown in Table 2. It is noted that the mixtures with water content 30% give higher maximum density than the dry mixtures.

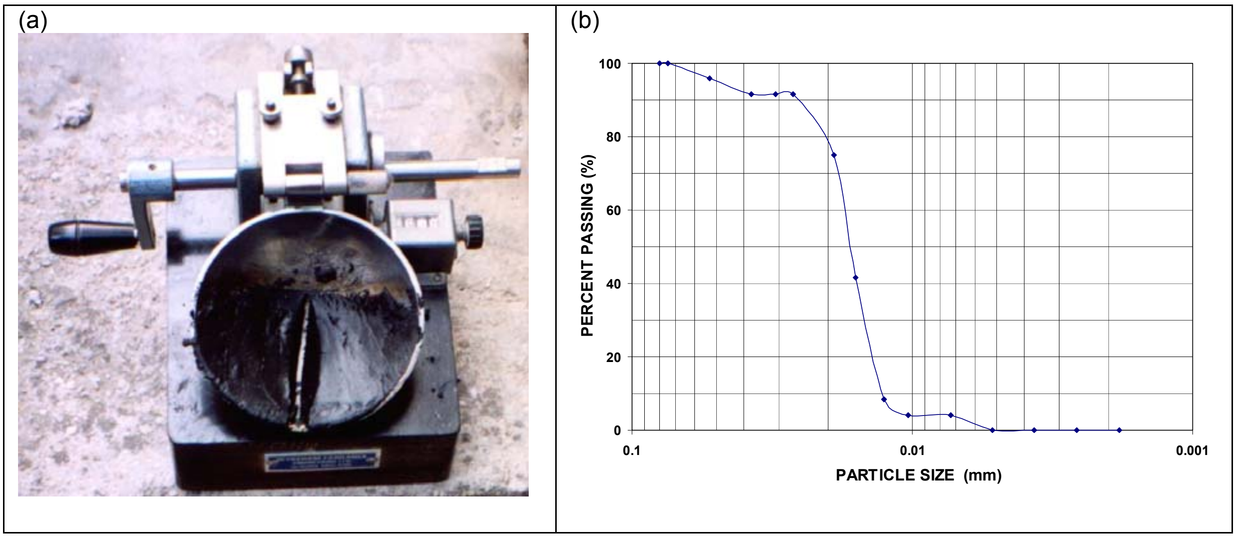

Figure 1. (a) Appearance of carbon saturated with water, (b) Grain size distribution of the carbon

Table 1. The properties of the carbon used in the current study and comparison with the properties of the field lignite from the power plant of Megalopolis

	Current carbon	Lignite from Megalopolis
Specific gravity of grains	1.6	1.5 to 2.5
Water content %	120 to 140	130 to 240
Liquid Limit	110	100 to 250
Plasticy Index	0	0 (mainly)
Coefficient C_C	0.8	1.0
Coefficient C_R	0.09	0.15

Table 2. Maximum and minimum dry densities and corresponding void ratios

	Maximum density (kN/m^3)	Corresponding Minimum void ratio	Minimum density (kN/m^3)	Corresponding Maximum void ratio
Dry	4.3	2.8	3.4	3.8
30% water	5.7	1.8	3.4	3.8

COMPRESSIBILITY

Compressibility was studied in the oedometer device for varying initial dry density ($\gamma_{d\text{-}o}$). Samples were prepared as described in Appendix A. Test results are given in figure 3. They illlustrate that the loosely compacted specimen behaves as normally consolidated with compression index $C_C = 0.8$. The denser specimens show preconsolidation up to a certain stress and then they converge with the line of the loose specimen. The swelling index C_R is equal to 0.08 for all specimens. The measured (virgin) compression index λ (= C_C/2.3) and the swelling index κ (=C_R/2.3), defined e.g. by Atkinson (1993) are 0.35 and 0.035 respectively.

It is of interest to compare the compressibility of the carbon with the compressibility of the lignite of Megalopolis whose properties are described by Stamatopoulos and Kotzias (1981). The comparison is given in table 1. It can be observed that the carbon prepared in the laboratory is, similarly to the Megalopolis lignite, very compressible.

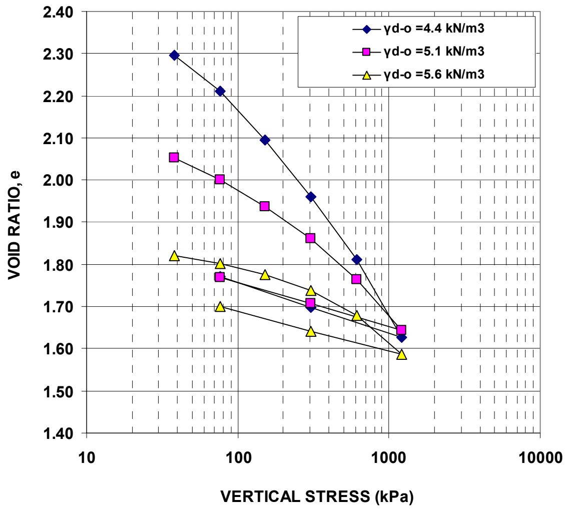

Figure 3. Results of consolidation tests for carbon at different initial densities

UNDRAINED STRENGTH

Been and Jefferies (1985) define the steady-state void ratio in terms of the octahedral stress, σ'_{oct} , as

$$e_{ss} = \Gamma_{CS} - \lambda_{CS} \ln(\sigma'_{oct}) \tag{1a}$$

In the above equation, the notation used by Bouckovalas et al (2003) is adopted and σ'_{oct} is in kPa.

Equation (1a) can be rewritten as

$$\sigma'_{oct\text{-}ss} = \exp\left[(\Gamma_{CS} - e) / \lambda_{CS} \right] \tag{1b}$$

The steady-state shear strength q_{ss} of soils can be obtained from the steady-state octahededral stress and the factor M, or equivalently the steady-state friction angle, φ_{ss}, as:

$$q_{ss} = \sigma'_{oct\text{-}ss} / M \quad \text{where} \tag{2}$$
$$q = (\sigma_1 - \sigma_3)$$
$$M = 6 \sin\varphi'_{ss} / (3 - \sin\varphi'_{ss})$$

In this study, the undrained strength is described by the factors $\sigma'_{oct\text{-}ss}$ and φ'_{ss}.

Undrained tests on carbon specimens were performed in the triaxial device. Appendix A describes the methodology of sample preparation. In all tests the effective consolidation stress ($\sigma'_{c\text{-}o}$) was 200kPa. The tests performed and results of the tests are summarized in table 3. Figure 4 gives the results of two typical tests one on loose and one on dense samples of carbon. It can be observed that in the dense sample (figure 4 – upper right), unlike the loose, lower pore pressure develops, and thus the final shear strength is larger (figure 4 – upper left). On the other hand, the final effective stress ratio is similar (figure 4 – lower left).

Figure 5 gives the final measured effective octahedral stress $\sigma'_{oct\text{-}ss}$ in terms of the void ratio after consolidation. The parameters Γ_{CS} and λ_{CS} of Equation (1) obtained were 0.39 and 4.00 respectively, while the coefficient of correlation R^2 was 0.87.

According to critical state theory (e.g. Atkinson 1993), the value of the parameter λ_{CS} obtained from undrained triaxial tests is the same as the parameter λ obtained from consolidation tests. The estimated ratio λ_{CS} / λ equals 1.11, reasonably close to unity.

Table 3 gives the measured friction angle φ_{ss} in terms of the initial density. It can be observed that φ_{ss} does not change considerably with initial density and takes the average value 36^o (or M=1.44).

Table 3. Summary of results of undrained triaxial tests. In all tests consolidation was at $\sigma'_{c\text{-}o}$=200kPa

Date	$\gamma_{d\text{-}o}$ [kN/m^3]	$\sigma'_{oct\text{-}ss}$ [kPa]	φ'_{ss} ^{o}
12.04.05	5.10	198	37
13.04.05	5.59	270	35
19.04.05	5.69	217	35
20.04.05a	5.10	193	36
20.04.05b	5.59	313	36
11.05.05	5.59	368	35
26.04.06a	3.60	82	35
26.04.06c	4.00	127	35

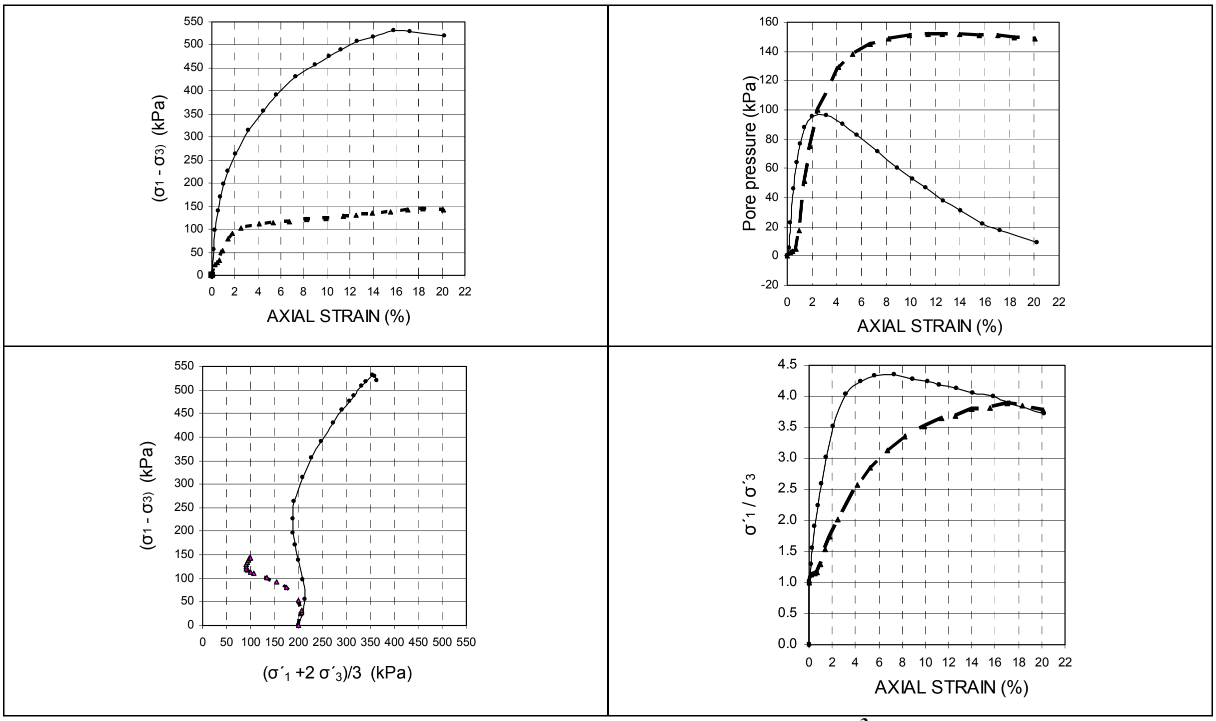

Figure 4. Response of undrained triaxial tests of loose (γ_{do}= 3.6kN/m^3 - dotted thicker lines) and dense ($\gamma_{d\text{-}o}$= 5.7kN/m^3) carbon samples.

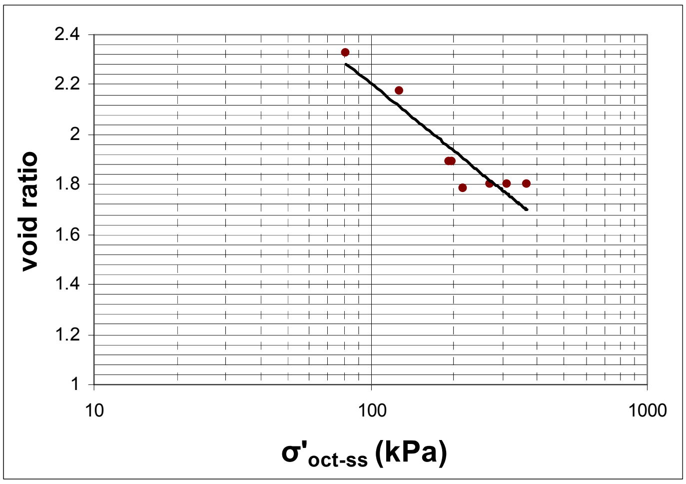

Figure 5. Measured critical state line of the carbon

CYCLIC STRENGTH

In the triaxial device liquefaction is simulated by applying in isotropically-consolidated specimens shear stress τ_{cyc}=[(σ_1 - σ_3)/ 2] about a zero mean shear stress value. We define a cycle of loading as the complete change of the shear stress (i) from zero to τ_{cyc} , (ii) from τ_{cyc} to -τ_{cyc} and (iii) from -τ_{cyc} back to zero.

Cyclic stress ratio SR (e.g Ishihara, 1993) is defined as

$$SR = (\sigma_1 - \sigma_3)_{cyc} / (2\, \sigma'_{c\text{-}o}) \qquad (3)$$

where the subscript "cyc" indicates maximum value attained during cyclic loading. In addition, cyclic shear strain during a loading cycle is defined as the maximum value of shear strain attained.

Permanent strain, or pore water pressure, is the strain, or pore water pressure that accumulates at the end of each cycle of loading.

As a result of cyclic harmonic loading, permanent pore pressure and cyclic shear strain build up with cycle number, while due to one-dimensional symmetry, considerable permanent horizontal movement does not accumulate (Committee on Earthquake Engineering et al, 1985, Ishihara, 1993). Cyclic strength for N cycles of harmonic loading, SR_N, is defined as the value of the cyclic stress ratio SR causing liquefaction in N uniform cycles. Liquefaction is the state where the effective stress becomes very small and the cyclic shear strain very large. Ishihara (1993) defines this state as when the cyclic shear strain exceeds a specific value. Consistently, in the present work, liquefaction is defined as when the cyclic shear strain exceeds 5%. This value for loose samples corresponds to dramatic loss of strength, while for dense samples to excessive deformation for civil engineering design purposes.

In liquefaction analyses, usually a reference earthquake of magnitude M=7.5, that corresponds to 15 cycles of uniform cyclic loading is used (e.g. European Prestandard, 1994, Seed et al, 1983). For this reason, the cyclic strength SR_{15} will be used below as index of the cyclic soil strength. Yet, it can be noted that it was observed that if the cyclic strength is taken as SR_{10} or SR_{20} instead of SR_{15}, the correlations presented below also hold.

Isotropically-consolidated cyclic undrained tests were performed in the triaxial device at three different densities and three different consolidation stresses. Appendix A describes the methodology of sample preparation. Table 4 gives the initial density and consolidation stress and partial results of the tests performed. Figure 6 presents the results of a typical test. Figure 7 presents the liquefaction curves obtained in each case of density and consolidation stress. Table 5 gives the value of the cyclic strength SR_{15} (defined previously) obtained from the liquefaction curves of figure 7. The factor SR_{15} was obtained from linear regression of the cyclic stress ratio and the logarithm of the number of cycles to liquefaction for all states considered in the present study. Table 5 also gives the coefficient of correlation (R^2) of the liquefaction curves and the void ratio, e, after consolidation for each case. The void ratio after consolidation is the average value estimated in all tests at similar initial density and consolidation stress. Figure 8a gives the cyclic strength in terms of initial density and confining stress.

CYCLIC STRENGTH IN TERMS OF THE STATE PARAMETER

According to Been and Jefferies (1985), the state parameter ψ is defined as the difference between the void ratio at the current state, e, and the void ratio at the steady-state e_{ss}, or:

$$\psi = e - e_{ss} \tag{4}$$

where e_{ss} is given by equation (1a).

Table 5 gives the state parameter prior to shearing, ψ, as a function of the void ratio and the consolidation stress $\sigma'_{c\text{-}o}$ for all states considered in the present study, obtained directly using equation (5). Figure 8b plots the cyclic strength in terms of ψ for all pairs of table 5. It can be observed that the cyclic strength increases as ψ decreases. A closer examination of the data reveals that the increase is more rapid for lower values of ψ. This is consistent with observations of other researchers (Boulanger, 2003) and explains the correlations between the cyclic strength, confining stress and void ratio described above (Boulanger, 2005).

If a linear relationship is assumed for the pairs (SR_{15}, ψ) of table 5 we can write

$$SR_{15} = a_1 + a_2\,\psi \qquad \text{for } 0.3>\psi>-0.7 \tag{5a}$$

Alternatively, if a second order polynomial relationship is assumed we can write

$$SR_{15} = b_1 + b_2\,\psi + b_3\,\psi^2 \qquad \text{for } 0.3>\psi>-0.7 \qquad (5b)$$

The coefficient of correlarion R^2 equals 0.85 and 0.95 for equations (5a) and (5b). It is inferred that equation (5b) better describes the data and is preferred. The best - fit equation is given in figure 8b. The corresponding parameters b_1 , b_2 , b_3 equal 0.17, -0.11 and 0.16 respectively.

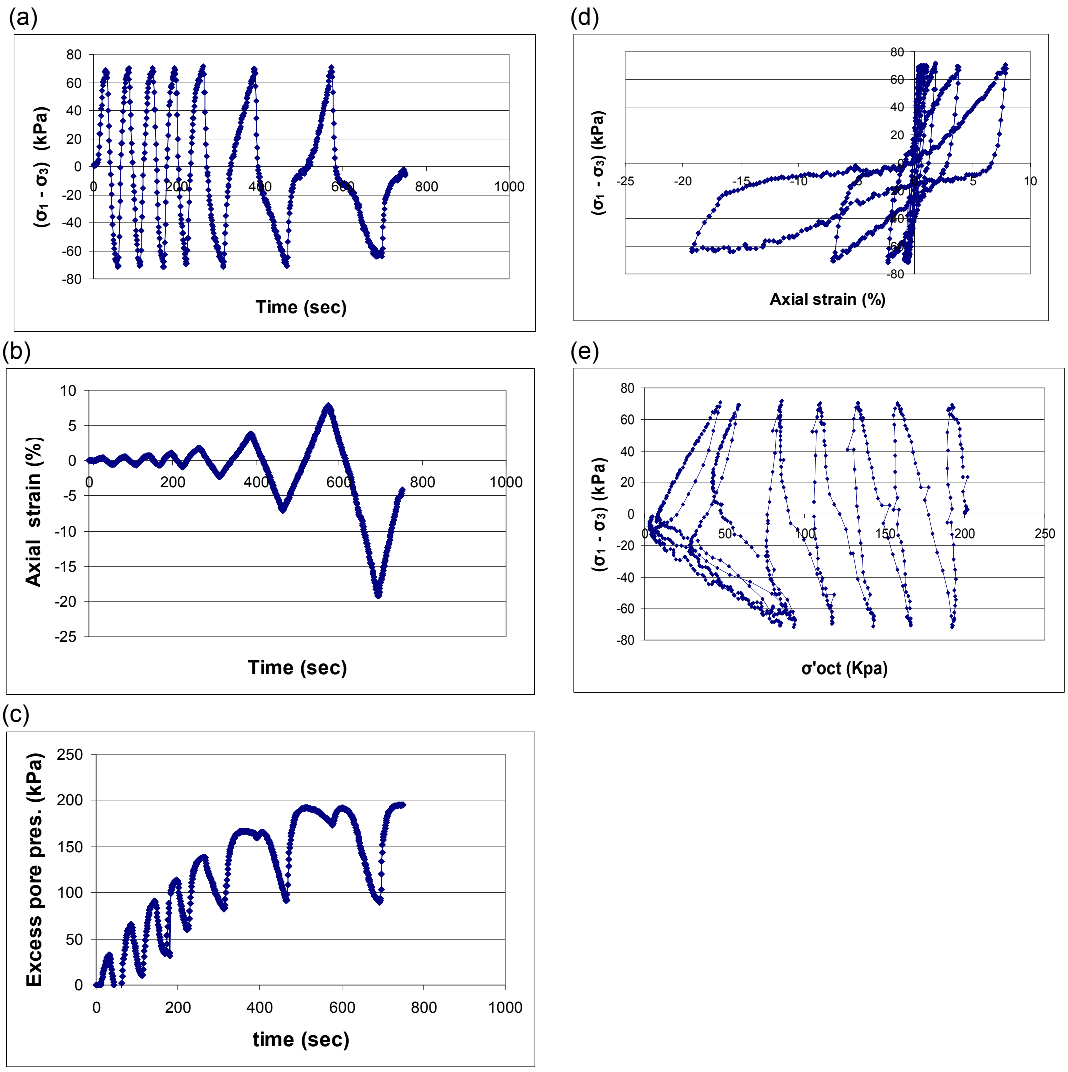

Figure 6. Results of a typical test. In the test presented $\gamma_{d\text{-}o}$=4.5KN/m^3, $\sigma'_{c\text{-}o}$=200kPa: (a), (b) (c) give axial stress, axial strain and pore pressure versus time and (d) and (e) give axial stress versus axial strain and σ'_{oct}

Table 4. Cyclic undrained triaxial tests performed on pulverised carbon and partial results

Date	γ_{d-o} (kN/m^3)	σ'$_{c-o}$ (kPa)	Δσ$_v$ (kPa)	Nf
05/04/2005	5.2	200	80	11
06/04/2005	5.2	200	62	40
08/04/2005	5.2	200	80	15
11/04/2005	5.2	200	94	8
12/04/2005	5.2	200	105	4
13/04/2005	5.7	200	115	6
14/04/2005	5.7	200	100	12
15/04/2005	5.7	200	85	30
18/04/2005	5.7	200	93	16
16/06/2006	4.5	200	60	11
17/06/2006	4.5	200	50	17
21/06/2006	4.5	200	70	6
22/06/2006	4.5	200	80	8
23/06/2006	4.5	200	40	35
14/09/2005	4.5	100	30	12
13/09/2005	4.5	100	37	6
12/09/2005	4.5	100	46	3
15/09/2005	4.5	100	25	27
16/09/2005	5.2	100	45	6
18/09/2005	5.2	100	32	18
19/09/2005	5.2	100	38	9
21/09/2005	5.7	100	45	15
22/09/2005	5.7	100	56	9
26/09/2005	5.7	100	62	9
28/06/2005	4.5	50	15	17
29/06/2005	4.5	50	20	13
30/06/2005	4.5	50	24	5
05/07/2005	4.5	50	13	43
03/07/2005	5.2	50	28	9
08/07/2005	5.2	50	25	4
07/07/2005	5.2	50	20	11
11/07/2005	5.2	50	17	32
12/07/2005	5.2	50	25	7
13/07/2005	5.7	50	32	14
25/09/2006	5.7	50	40	2
26/09/2006	5.7	50	33	4
27/09/2006	5.7	50	30	12

Table 5. The cyclic strength SR_{15} in terms of γ_{d-o} and σ'_{c-o} of pulverized carbon. The void ratio after consolidation (e) and the state parameter (ψ) are also given.

γ_{d-o} (kN/m^3)	e_o	σ'$_{c-o}$ (kPa)	SR_{15}	R^2	e	ψ
4.5	2.60	50	0.17	0.9	2.47	0.00
4.5	2.60	100	0.15	0.97	2.45	0.25
4.5	2.60	200	0.14	0.84	2.4	0.47
5.2	2.12	50	0.2	0.88	2.07	-0.40
5.2	2.12	100	0.17	0.96	2.03	-0.17
5.2	2.12	200	0.17	0.96	1.95	0.02
5.7	1.84	50	0.3	0.78	1.82	-0.65
5.7	1.84	100	0.23	0.88	1.8	-0.40
5.7	1.84	200	0.21	0.98	1.75	-0.18

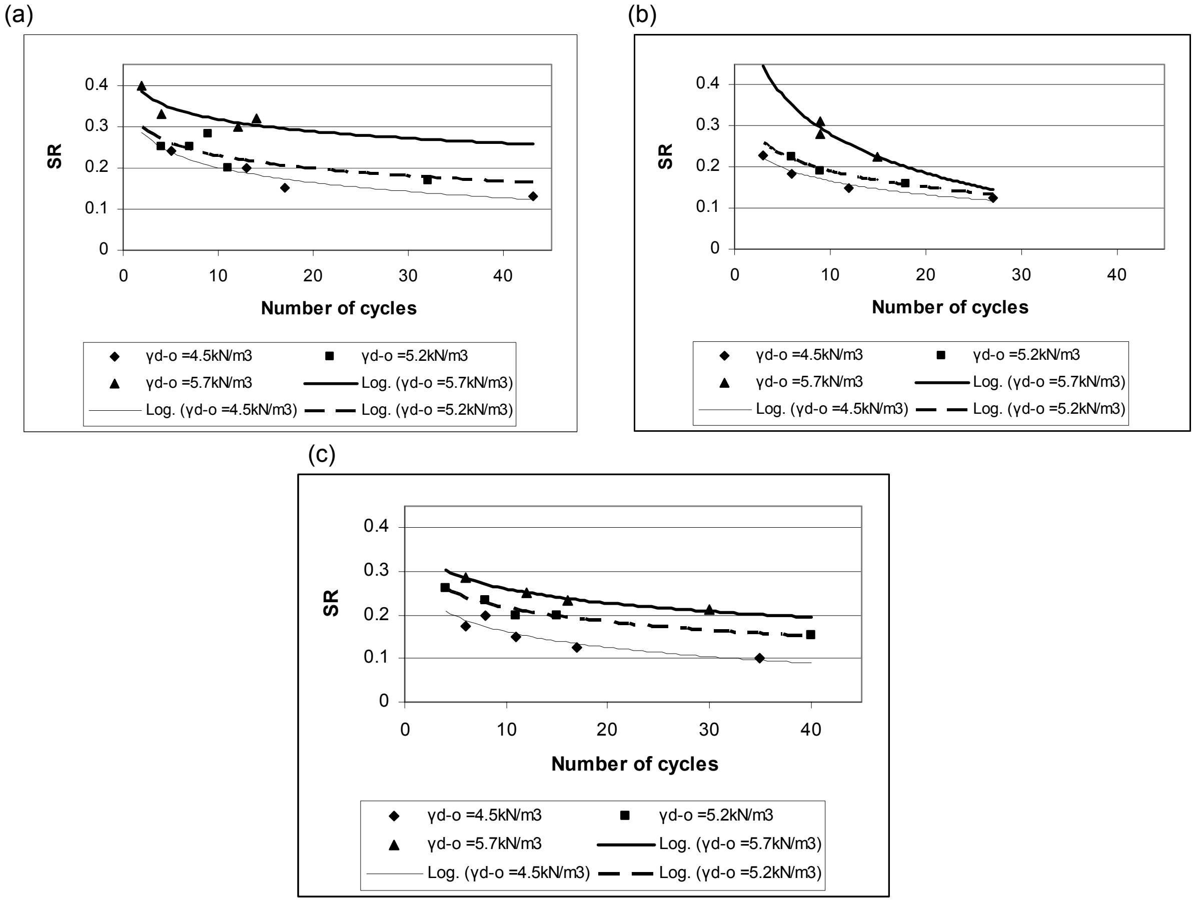

Figure 7. Liquefaction curves obtained for all cases of pulverized carbon considered (a) σ'_{c-o}=50kPa, (b) σ'_{c-o}=100kPa, (c) σ'_{c-o}= 200kPa

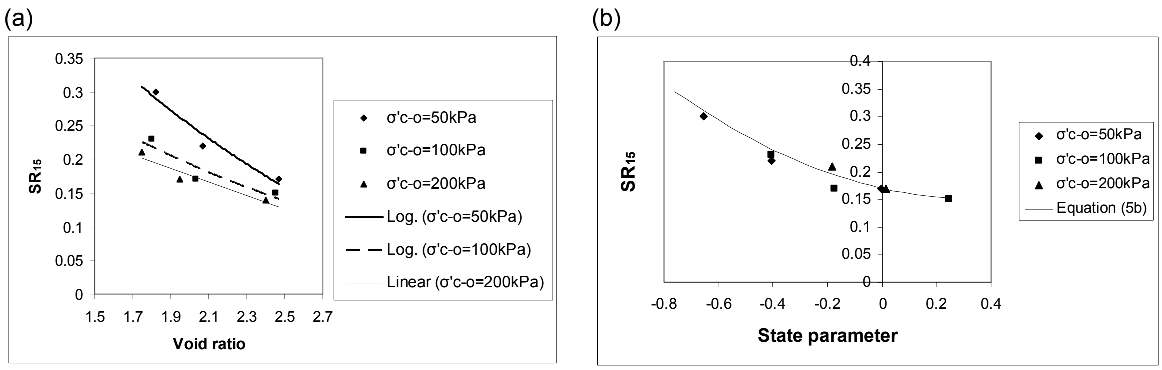

Figure 8. Cyclic strength of pulverized carbon in terms of (a) the consolidation stress and the void ratio and (b) the state parameter

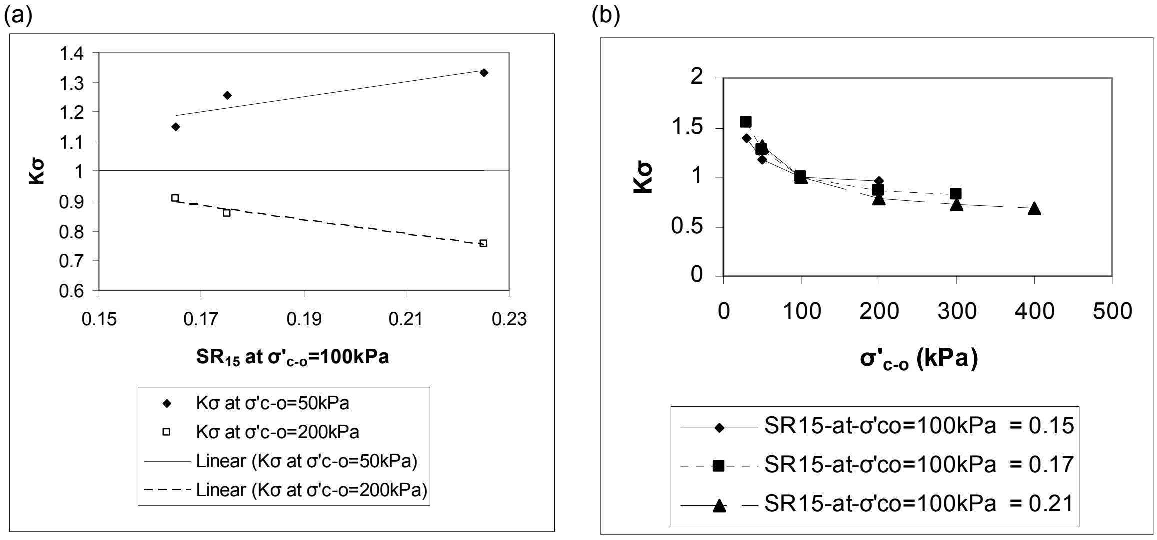

Figure 9. Kσ factors obtained (a) from figure 8a, (b) from figure 8b using equations (1) and (5)

DISCUSSION

The material tested has small cyclic strength and that may make it susceptible to liquefaction. On the other hand, the cyclic strength does not take small values (less than 0.13), even for loose samples.

As an example of liquefaction susceptibility, a layer of the organic material with properties similar to the material studied existing at Megalopolis, at a depth of 0 to 5m is examined. The water table is assumed at a depth of 1m. The dry density of the material is taken as 5.2kN/m^3. The water content is taken below the water table as 130% and above the water table as 50%.

According to the Greek Seismic code (OASP, 1999) provisions in Megalopolis the design earthquake has maximum acceleration equal to 0.24g where g is the acceleration of gravity and the Factor of Safety (FS) against liquefaction equals

$$FS = (SR_{15}\ \sigma'_v) / (0.65*0.24*\ \sigma_v) \qquad (6)$$

where σ'_v and σ_v are the effective and total vertical stresses prior to the application of the earthquake respectively.

At depths of 1, 2, 3, 4 and 5m, $\sigma'_v=\sigma_v=7.8$kPa, $\sigma'_v=0.49\sigma_v=9.8$kPa, $\sigma'_v=0.37\sigma_v=11.7$kPa, $\sigma'_v=0.31\sigma_v=13.7$kPa and $\sigma'_v=0.28\sigma_v=15.6$kPa respectively. According to table 5, at $\gamma_{d\text{-}o}$ =5.2kN/m^3 and σ'_v less than 50kPa, SR_{15} equals 0.2. According to equation (6), FS at depths of 1, 2, 3, 4 and 5m, equals 1.28, 0.63, 0.47, 0.40 and 0.36 respectively. It is inferred that the organic soil layer is liquefaction-susceptible.

Figure 8a illustrates that as the void ratio increases, the cyclic strength at similar values of $\sigma'_{c\text{-}o}$ decreases with a progressively smaller rate. This is consistent with measured response in sands, reported e.g. by Boulanger (2003).

The relationship between the cyclic strength and the consolidation stress is usually expressed by the Kσ factor defined as

$$K\sigma = SR_{15} / SR_{15\text{-at-}\sigma'c\text{-}o=100kPa} \qquad (7)$$

Figure 9a gives the Kσ factors obtained from figure 8a in terms of the density. In this figure, the soil density is expressed indirectly in terms of the cyclic strength SR_{15} at $\sigma'_{c\text{-}o}$=100kPa.

From figures 8a and 9a it can be observed that (a) as the confining stress increases, the cyclic strength at similar void ratio decreases, (b) this effect is more pronounced for the denser samples, (c) at σ'_{c-o}=50kPa the Kσ factor varies from 1.05 to 1.40 depending on soil density and at σ'_{c-o}=200kPa the Kσ factor varies from 0.90 to 0.86 depending on soil density. All the above are in agreement with the Boulonger (2003) relations giving Kσ for sands.

The strong correlation of the pairs (SR_{15}, ψ) of equation (5b) illustrates that the cyclic strength can be analysed in terms of the state parameter for tests results performed with the same sample preparation method at different void ratios and confining stresses. Furthermore, the relationship of figure 8b allows the generation of Kσ versus σ'_{c-o} relations in terms of soil density, using equations (7) and (1). Figure 9b gives such relations. In this figure, the soil density is expressed indirectly in terms of the cyclic strength at σ'_{c-o}=100kPa.

FUTURE STUDIES

Test results presented in the paper belong to a continuing study of the mechanical properties of carbon-sand mixtures. In the near future the liquefaction susceptibility, or equivalently the cyclic strength, of the mixtures will be studied in terms of their carbon content, density and consolidation stress.

CONCLUSIONS

The paper investigates the cyclic strength of an organic soil. The “organic” material was carbon made from carbonized wood fully pulverized. Oedometer tests illustrated the correlation between the compressibility and initial density. The critical state line was obtained from triaxial undrained tests. Cyclic undrained triaxial tests were performed at three different densities and three different consolidation stresses σ'_{c-o}. As the void ratio increases, the cyclic strength at similar σ'_{c-o} decreases with a progressively smaller rate. For similar void ratio, the cyclic strength decreases as σ'_{c-o} increases. As the void ratio increases, the effect of σ'_{c-o} on the cyclic strength is less pronounced.

The relationship between the state parameter the cyclic strength has strong correlation coefficient. Consistently with the relationships between the cyclic strength, the void ratio and σ'_{c-o} described above, the cyclic strength decreases as the state parameter increases at a progressively smaller rate.

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APPENDIX A. SAMPLE PREPARATION METHODS USED

For the oedometer device

The soil is oven dried so that its moisture reaches approximately 0%. A specimen is prepared in the oedemeter ring of diameter 15cm and height 6cm in the following way: Knowing the volume and the desirable dry density we estimate the weight of the quantity to be placed. Soil is placed inside the ring in layers compacted with the use of a round metallic rammer by applying constant energy blows. The number of layers and blows can vary according to the desired compaction. Layers vary from 3 to 5. If the material cannot be compacted, water is added. Next follows saturation by filling of the socket around the ring with water. The water level was maintained constant during the test.

For the triaxial apparatus
The soil is oven dried so that its moisture reaches approximately 0%. A specimen is prepared in a cylindrical mould with open edges. The mold is placed on the pedestal of the compression frame and vacuum is applied at the base. Vacuum intensity is increased as the soil level inside the mold is raised. Placement is in 5 to 9 layers, by the application of constant energy blows with a rammer. If soil can not be compacted water is added.

Next follows saturation of the specimen. With closed drainage valves the lateral pressure and the back pressure are increased in successive steps to 500kPa. The B parameter, defined as the ratio of the pore pressure increase by the increment of the lateral stress, was measured. It was verified that it was equal to about one. All specimens were then consolidated by increasing the lateral pressure at 550, 600 or 700kPa.

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