Cyclic/dynamic simple shear tests: Recent developments
Essais de cisaillement cycliques/dynamiques: Développements récents

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SYNOPSIS: This paper deals with the influence of the way of loading a soil specimen on the dynamic simple shear tests results and with the significant role of factors such as the number of cycles or the previous dynamic history. The experimental investigation was carried out on a pyroclastic soil (constituted of low crushing strength grains) and using a computer-aided laboratory procedure.

1. INTRODUCTION

It is well known that the stress-strain behaviour of soils, subjected to cyclic as well as dynamic shear loads, may be described by means of shear modulus G and damping ratio D. Both these parameters strongly depend on shear strain γ.

The shear modulus has an initial tangent value (G_o), which remains constant up to an amplitude threshold strain (γ_{at}), that for most soils is in the order of 10^{-3}%. Within this range ($\gamma<\gamma_{at}$), soils exhibit a linear-elastic behaviour. The modulus G is independent on the shear strain γ and furthermore its value is not influenced by the previous and the current dynamic loading history. In the same strain range, the damping ratio D has a minimum constant value (D_o), about zero if viscous effects are not involved.

At strain levels greater than γ_{at}, the shear modulus becomes lower and lower, while the damping ratio increases. Soils exhibit a markedly non-linear behaviour. Moreover, beyond a higher threshold strain (γ_{ct}), soil structure irreversibly modifies: therefore soils start remembering previous (dynamic or cyclic) history and their response to dynamic excitation depends on the way of applying the loads.

The recent development of digital laboratory equipment suited for fully automatized simple shear tests allows remarkable refinements of the experimental procedures in order to study the constitutive laws of the soil at low-intermediate strain levels (Ni 1987; Papa et al. 1988a). At the same time, such a technique allows more critical analyses of the test interpretation criteria and, on the other hand, an insight to several peculiar features of the rheologic behaviour of soils.

At the Soil Mechanics Department of the University of Naples a research project on these themes is being carried out. In this paper only some results will be shown about:

a - the influence of the dynamic ways of loading on the results of the resonant column test in the non-linear range of the soil ($\gamma>\gamma_{at}$). The aim may be the development of more realistic models, suited to analyse the response of the soil during this kind of test.

b - the significant role of factors, often neglected, such as the number of cycles of loading or the effect of the previous dynamic history at high strains on G_o and D_o.

The results reported in this paper were obtained from torsional shear, resonant column and free vibration tests, performed by means of a unique device, originally designed at the University of Texas at Austin and widely described by Isenhower (1979).

This apparatus is supported by digital control and acquisition devices, fully interfaced to a computer. The features of the computer aided laboratory procedures are described by Papa et al. (1989).

2. TESTING MATERIAL AND PROCEDURES

The experimental investigation was carried out on a pyroclastic soil («pozzolana»), whose general features and static properties have been widely studied (Pellegrino 1967).

The tests were performed on 28 undisturbed samples. The tested soil is a sandy silt, formed by a fine matrix (volcanic ash) and few pumiceous and lithic inclusions, whose diameter is smaller than 5 mm. Although a small fine fraction is present (CF = 5÷10%), the soil does not show any plasticity.

The volcanic ash is mainly constituted of pumiceous particles, formed by the consolidation of fragments of magma rich in gas, that were thrown into the atmosphere and rapidly cooled. They have a vitreous texture which is more or less spongy and frothy. The grains are irregularly shaped and have an high porosity. Only a limited amount of internal pores are in communication with one another and with the atmosphere.

With reference to the tested samples, the average values of the index properties are:

Unit weight of particles	γ_s=24.2±0.1	kN/m^3
Dry unit weight	γ_d=10.8±1.0	kN/m^3
Void ratio (external)	e =1.22±0.04	
Natural water content	w =0.15±0.05	
Degree of saturation	S_r=0.32±0.12	

It can be noticed that either γ_s or γ_d assume unusually low values.

The grain structure affects the mechanical behaviour of the soil even at relatively moderate stress levels, because of the low crushing strength of the particles (Rippa & Vinale, 1982).

The experimental work was planned to verify the possible effectiveness of this factor also on the dynamic behaviour of this pyroclastic soil. As a matter of fact this occurrence was confirmed by the results of the investigation presented herein. The main evidences are:

a - the stress-strain relationship, which shows a shape quite different from those typical of materials having the same grain size distribution;

b - the clear modification of the soil properties with the number of cycles.

During the tests, each specimen was previously consolidated under a spherical pressure and then was subjected to cyclic and dynamic loads (i.e. torsional shear and resonant column tests) starting from a shear strain $\gamma \approx 10^{-3}$% up to about 1%.

The tests performed at $\gamma > \gamma_{at}$ were periodically alternated by low-amplitude resonant column tests, with the purpose of verifying the sensitivity of the modulus G_o to previous excursions at strain levels greater than γ_{at}.

Instead of the usual procedures, the tests were performed adopting some peculiar dynamic loading sequences. For instance, the resonant column tests were carried out using different frequency variation rates («sweep velocities»), while the torsional shear tests were planned to analyse in detail the evolution of the material properties during a given sequence of loading cycles.

More details about the testing procedures are given in the following pages, together with the discussion of the results.

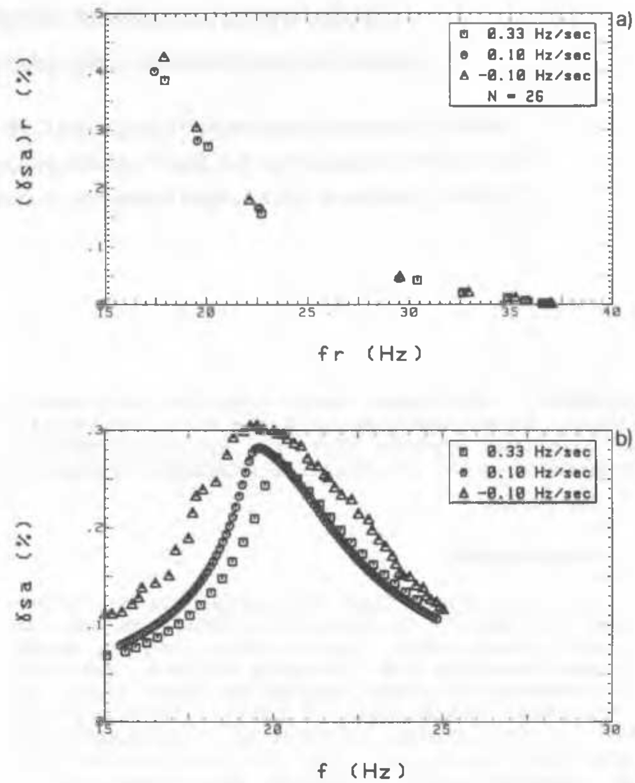

Figure 1. Results of a series of resonant tests (a) and typical frequency response curves (b).

3. NON-LINEARITY EFFECTS IN RESONANT COLUMN TESTS

Beyond the amplitude threshold strain γ_{at}, because of the non-linear ($\gamma > \gamma_{at}$) and the non-elastic ($\gamma > \gamma_{ct}$) behaviour of soils, the way of applying dynamic excitation during resonant column tests plays a significant role (Ni 1987, Papa et al. 1988b-c).

Both theoretical and experimental results point out that, for a given excitation amplitude, the dynamic responses (amplification curves) of the specimen may be quite different, according to the laws of load variation with time. In particular, different values of resonant frequency (f_r), half-power bandwidth and peak strain amplitude may be obtained depending on the frequency sweep velocity.

3.1 Test results

For the reasons pointed out above, three different test procedures were planned. In each case the frequency was varied over the same range (10 Hz width): the first two tests were both performed with upgrade sweeps, but with different gradients (0.33 and 0.1 Hz/s); while during the third test a downgrade sweep was executed setting a velocity of -0.1 Hz/s.

After each set of resonant tests a free vibration test was performed in order to evaluate the damping ratio.

Typical results of a complete sequence of resonant tests are shown in Figure 1a, where strain peak amplitudes $(\gamma_{sa})_r$ are plotted against resonant frequency values f_r.

It can be noticed that in a first strain range the data points are practically coincident, no matter which procedure was followed, while as γ increases ($\gamma > \gamma_{at}$) they are different depending on the sweep rate. In particular, the downgrade sweep produces the lowest f_r and the highest $(\gamma_{sa})_r$, while the highest f_r and the lowest $(\gamma_{sa})_r$ values belong to the upgrade faster sweep.

This occurrence can be better evaluated comparing the shapes of the three frequency response curves shown in Figure 1b, as a sample set of data: the differences among them, particularly marked in the ascending branches, will be widely discussed in the following pages.

The results of the complete sequence of resonant tests are also shown in terms of G and D in Figures 2a and 2b. In the latter figure the D values determined by free vibration tests are also reported.

The figure 2a underlines that all the experimental data indicate a unique G(γ) trend, apart from the differences among the sweeps. Therefore a first conclusion can be drawn: the G(γ) relationship may be determined by means of the whole set of data.

On the other hand, figure 2b shows that the same results produce quite different D values, depending on the sweep direction and velocity. In particular, the data might be divided into two different bands: the upper one including downgrade sweep results, that are very close to the decay data; the lower band containing the data related to both upgrade sweeps.

These results are in agreement with the theoretical modelling of the resonant column

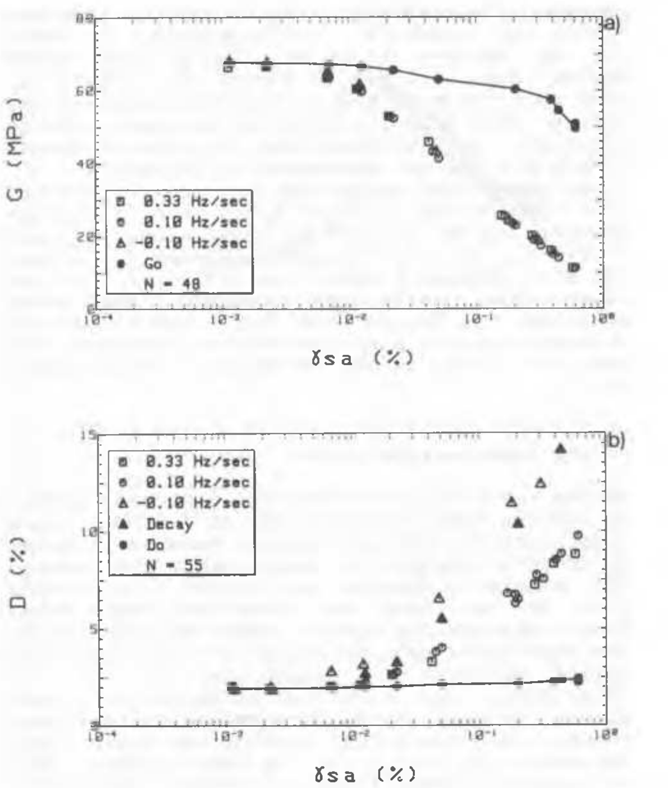

Figure 2. Shear modulus G and damping ratio D from a sequence of dynamic tests.

tests that will be discussed in the following paragraphs.

3.2 Theoretical background

The variability of the soil responses during resonant tests within the high-amplitude strain range can be theoretically justified by means of the analysis of the non-linear vibration of the soil specimen.

In the classical interpretation criteria of the dynamic motion of the specimen (Richart, Hall and Woods, 1970; Isenhower, 1979), on account of the analytical difficulties, the shear modulus G and the equivalent viscous damping coefficient μ (or its critical ratio D) are forced to be constant with the strain level γ. This linearity hypothesis is rather inadequate to model the soil response at high and intermediate strain levels.

For sake of simplicity, it is possible to refer to an alternative formulation of the usual dynamic model (infinite degrees of freedom), which looks very similar to the classical «simple oscillator» solution, writing the equilibrium equation in the following way (Papa et al., 1988c):

$$I_t \frac{\delta^2\gamma}{\delta t^2} + \frac{\mu J}{l}\frac{\delta\gamma}{\delta t} + \frac{G J}{l}\gamma = \frac{\bar{r}}{l} M_o(t) \qquad (1)$$

In the formula above the specimen is considered homogeneous, and the shear strain distribution in the specimen's body is assumed to be uniform, so that the dynamic motion may be expressed using only the function $\gamma(t)$ (reference shear strain). In addition (see Figure 3):

I_t = equivalent mass polar moment of inertia of the dynamic system (specimen + top cap + drive plate);

J,l = cross-section polar moment of inertia and height of the cylindric specimen;

$\bar{r}$ = specimen average equivalent radius (varying between 0.67 R and 0.80 R, where R is the external radius);

$M_o(t)$ = torsional excitation amplitude.

The formulation (1) allows to introduce more realistic constitutive laws in the model to face the problem in an appropriate way. Hence, it is theoretically possible to take into account the non linearity of the soil in the solution, assuming for G and μ whatever laws of variation:

$$G = G(\gamma) \qquad \mu = \mu(\gamma) \qquad (2)$$

Furthermore, the «classical» methods used to process the equation of motion cannot even consider the effective law of variation of the torsional excitation in the time. As a matter of fact, it is always assumed that the dynamic response of the specimen, subjected to a continuous frequency sweep, can be compared with the amplification curve obtained from a series of forced vibrations, each one having constant frequency and amplitude, and indefinite duration.

Nevertheless, as discussed above, the experimental practice remarks a certain sensitivity of the dynamic response to the sweep variation rates.

To take into account the effective laws of variation of the excitation with time, a more reliable analysis technique of the resonant tests might be available introducing in the second member of the equation (1) an expression like:

$$M(t) = M \operatorname{sen}[2\pi f(t)t + \phi(t)] \qquad (3)$$

where f(t) and $\phi(t)$ reproduce the actual loading function.

In § 3.3 and § 3.4 two different ways of solving the equation (1) in the non-linear field will be discussed.

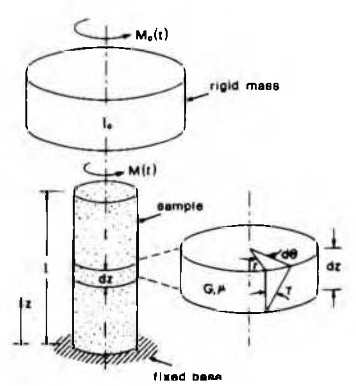

Figure 3. The dynamic model.

3.3 The solution with the Duffing model

One chance to solve equation (1) in an explicit form is related to the assumption of simplified relationships for the constitutive laws (2); this is possible employing the so-called «Duffing model» (Thomson, 1981; Ni, 1987), according to which it can be written:

$$G(\gamma) = G_o(1 + \alpha\,\gamma^2) \qquad \mu(\gamma) = \frac{2\,G_o\,D}{\Omega_n} = \text{constant} \tag{4}$$

where G_o is the low-amplitude shear modulus, α is a non-linearity factor (negative for "strain-softening" materials), Ω_n is the low-amplitude natural pulsation.

By substituting the expressions:

$$\frac{G_o\,J}{I_t\,l} = \Omega_n^2 \qquad \frac{\mu\,J}{I_t\,l} = 2D\Omega_n \tag{5}$$

and using the functions (4), eq. (1) can be rearranged as follows:

$$\frac{\delta^2\gamma}{\delta t^2} + 2D\Omega_n\frac{\delta\gamma}{\delta t} + \Omega_n^2(\gamma+\alpha\gamma^3) = \frac{\bar{r}}{I_t l}M_o(t) \tag{6}$$

and solved in the case of sinusoidal excitation with constant frequency f and amplitude M_o.

As f variates, it is possible to evaluate the frequency response curve (strain maximum amplitude versus f), which consists of two different branches, described by the relationship:

$$\frac{f^2}{f_n^2} = 1+\frac{3}{4}\alpha\gamma^2 - 2D^2 \pm \sqrt{\frac{T_o^2}{\Omega_n^4\,\gamma^2} - 4D^2\left(1+\frac{3}{4}\alpha\gamma^2 - D^2\right)} \tag{7}$$

where T_o is given by:

$$T_o = \frac{M_o\,\bar{r}}{I_t\,l} \tag{8}$$

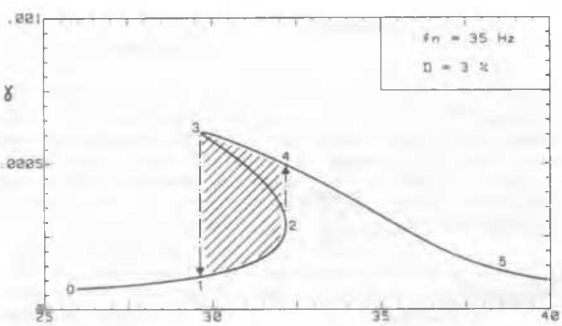

Figure 4. Example of amplification curve with the Duffing model.

Plotting the frequency response curve obtained in this way (Figure 4), it is possible to identify in the f-γ plane an "instability zone" (shaded area 1-2-3-4 in figure 4), that is, where each value of the excitation frequency may produce more than one value of the peak strain amplitude; at the same time the curve segments 0-1 and 4-5 can be considered as "stable".

This means that performing an "upgrade sweep" (i.e. increasing f with time) the frequency response curve will be followed along a path 0-1-2-4-5, while a "downgrade sweep" (f decreases) will produce a path like 5-4-3-1-0. In the former case, while the frequency approaches resonance, the strain level may suddenly increase, stepping from the lower stable branch to the upper one: this is the so-called «jump phenomenon».

3.4 The numerical solution

Besides the qualitative remarks discussed above, the Duffing model actually has no further chance of application with real soils, because it makes use of an unlikely G(γ) relationship and, moreover, assumes a damping coefficient μ invariable with γ. On the other hand, with the introduction of more appropriate models (removing the restrictive hypotheses upon which eq. (7) is based) the explicit solution must be excluded.

Therefore, the numerical integration is compulsory: in this way it is possible to take into account also the actual loading functions (see expression (3) in § 3.2). A good working tool may be one of the classical methods used in the vibration mechanics field (Runge-Kutta, Adams-Bashford-Moulton,...).

However, the numerical solution of the equation (1) demands the previous knowledge of the G(γ) and μ(γ) (or D(γ)) functions, which are not yet known during the execution and interpretation of the tests. Hence, this way of dealing with the non-linear model in order to determine the constitutive laws of the materials should require an iterative procedure, with considerable computational efforts. Therefore this strategy should be intended only as a prospect.

As a preliminary approach, taking the G(γ) and D(γ) laws obtained from the traditional ("linear", therefore approximate) test analysis criteria, it is possible to try numerically to reproduce the experimental procedure to verify at least its overall reliability.

This check was done, as an exemplification, referring to the set of three resonant tests reported in Figure 5a, introducing in the equation of motion (1) the characteristic laws of the hyperbolic modified model (Hardin and Drnevich, 1972).

The analysis was performed using the Runge-Kutta method, trying to simulate in a numerical way all the operative conditions of the actual tests: namely, the same sweep velocities and directions were brought in the loading function (3), and the outcoming response function γ(t) was reduced into the frequency domain by means of a RMS integration over time intervals corresponding to the sampling rate of the digital acquisition instruments.

In Figure 5b the resulting frequency response curves are plotted: their trends show a fair agreement with the experimental data. This attests the consistency between the observed phenomenology and the proposed physical modelling.

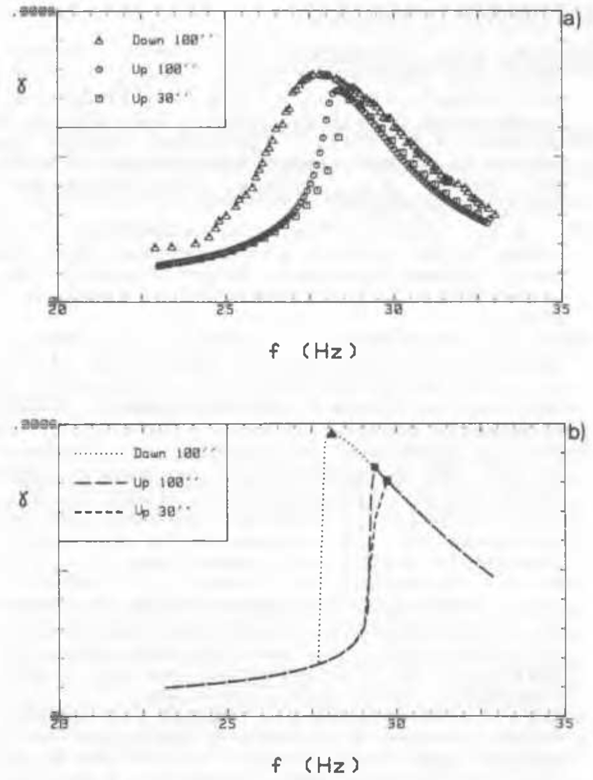

Figure 5. Experimental (a) and simulated (b) frequency response curves.

Both the experimental and the numerical results show that the resonant frequency f_r decreases with the upgrade sweep velocities (● slow, ■ fast), and the downgrade sweep (▲) produces the lowest f_r value. The peak amplitude and the half-power bandwidth increase in the same fashion: hence, using ordinary interpretation techniques, in upgrade sweeps higher G values and lower D values can be expected in respect to those obtained from downgrade sweeps.

Nevertheless, from a quantitative point of view, the simulated curves fail to overlap the experimental ones: this occurrence clearly highlights the limits of the standard criteria of interpretation of resonant column tests in the non-linear field.

4. EFFECTS OF THE NUMBER OF CYCLES OF LOADING IN TORSIONAL SHEAR TESTS

Like the resonant tests, the low-frequency cyclic torsional tests were conducted with the purpose of monitoring the influence of the terms of variation of the cyclic loading in the time.

For a better understanding of this matter, testing sequences implying only torsional shear tests were followed (Papa et al., 1988b), with careful attention paid to the effects of the loading parameters (i.e. number of cycles, frequency, and so on).

The experimental procedures allowed to point out some peculiar features of the rheologic behaviour of the soil, essentially due to the amount of the cycles of loading.

The results clearly show that, within the so-called non-elastic range ($\gamma > \gamma_{ct}$), the parameter N_{cycles} considerably affects both the shear modulus G and the hysteretic damping ratio λ.

At each strain level beyond γ_{ct}, a given excitation amplitude always produces a simultaneous reduction of both G and λ with the increase of N_{cycles}.

A sample set of data showing this effect is given in Figure 6 a-b. For each strain step, the shear modulus G(i) associated to the i-th cycle is normalized by the first cycle value G(1) and then plotted against the shear strain (Figure 6a) or the number of cycles (log scale, Figure 6b). The figures clearly show how the softening of the material (attenuation of G) with N_{cycles} is more and more evident with the increasing of the strain level.

Such a kind of effect becomes much more significant in terms of hysteretic damping ratio λ. This is clearly shown by the experimental data plotted in Figure 7, representing a typical set of results from a series of torsional shear tests. The extent of the decrease of the damping ratio with N_{cycles} is evident: the outcoming relative reductions of λ can raise up to 20 % of the initial value.

A further evidence of irreversible changes in the soil structure at these strain levels is given by the values of G_0 and D_0 obtained from the low-amplitude resonant tests performed after each high-amplitude strain step (see § 2). Looking for instance at Figure 2a, the considerable reduction of G_0 with the increase of γ seems to confirm the fact that the material is sensitive to the previous dynamic loading history.

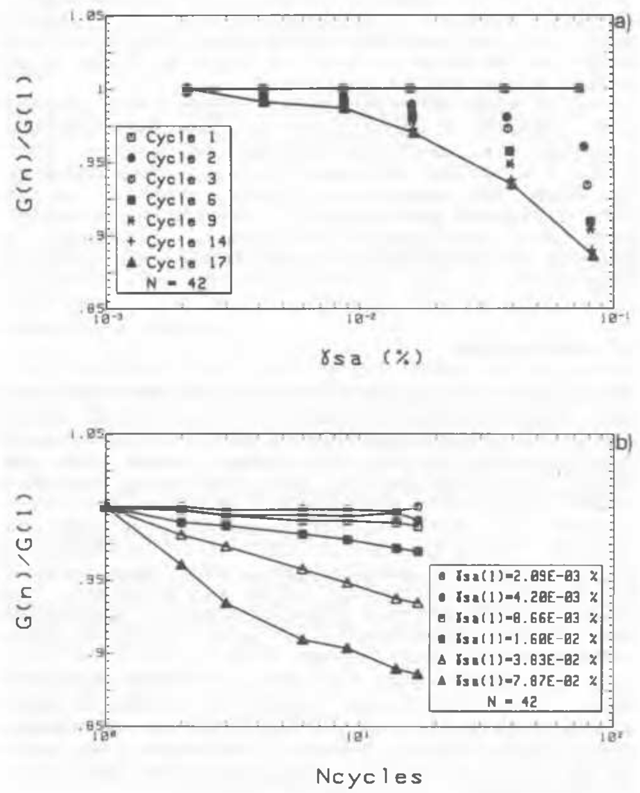

Figure 6. Effect of the number of cycles on G.

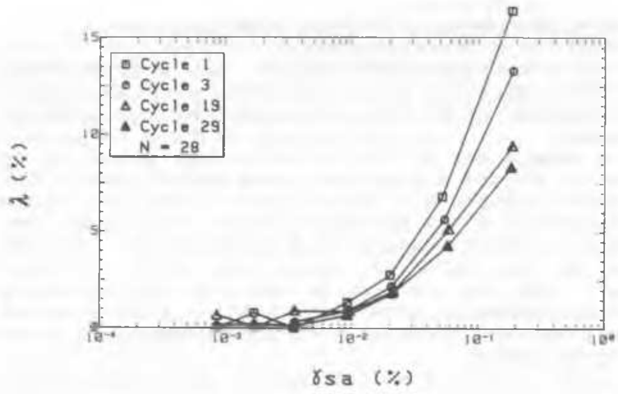

Figure 7. Effect of the number of cycles on λ.

An explanation of such a kind of behaviour may be sought looking at the low crushing strength of the pumiceous particles (see § 2).

The grading curves obtained before and after each test have seldom, and to a poor extent, pointed out any valuable change in the diameter of the grains: hence the behaviour described above does not seem to be due to the splitting of the particles, with consequent reduction of their size.

However, the occurrence of a decrease of G and λ with N_{cycles} might still be associated to the pyroclastic nature of this soil, supposing a shallow rupture phenomenon around the contact areas of the pumiceous particles in such a way that no valuable alterations in the grain size distribution can be generated.

Such a kind of hypothesis, after all, might also explain a peculiarity of the experimental results, i.e. the simultaneous reduction of both G and λ with the increase of N_{cycles}. This generally does not happen for other granular soils with different petrography, for which in analogous test conditions a decrease in stiffness is related to an opposite trend in damping properties.

5. CONCLUSIONS

The matters discussed in this paper have pointed out a remarkable influence of the ways of loading the specimen on simple shear dynamic test results within the non-linear range of the soils. This observation (feasible using computer aided laboratory procedures) requires the development of more realistic models, suited to analyse the response of the soil, especially during resonant column tests. For this purpose a complete, but simple, model was proposed. The numerical simulations presented herein encourage the future development of more sophisticated interpretation procedures.

On the other hand, it was shown that factors, often neglected (i.e. number of cycles or previous dynamic history at high strains), assume a significant role in the soil behaviour, at least in relation to the studied pyroclastic soil. As a consequence, it might be necessary to modify the usual testing procedures to highlight the influence of such parameters.

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