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An Experimental Study to Assess the Shear Modulus Degradation by Fatigue of Mexico City Clay

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ABSTRACT

In this research, the degradation by fatigue of Mexico City clay is studied using a triaxial equipment where the cyclic stress amplitude was maintained constant during the experiment. The variables considered in the study were the following: state of the soil, effective mean confining stress, magnitude of cyclic stress and number of loading cycles. Undisturbed samples, anisotropically and isotropically consolidated, were subjected to cyclic loading for this purpose. When analyzing the cyclic stress-strain response with the number of cycles a threshold of permanent deformation in function of the cyclic deviator stress and axial strain was found. When the cyclic strain exceeds this distinctive value the rate of permanent (plastic) deformations accumulate faster. For practical applications of computing permanent deformations in Mexico city a simplified method is proposed. This method considers the above threshold and a hyperbolic model to represent the cyclic response in Mexico City clays.

INTRODUCTION

Soil fatigue develops mainly because of the cyclic shear distortions caused by seismic loading at microstructural level. The importance of this phenomenon depends on a number of factors such as type and state of soil, effective confining stress, magnitude of cyclic shear stress and number of times of cyclic stress applied. Degradation rates increases with the pore water pressure developed during cyclic loading. For soils where dynamic pore water pressures are negligible the fatigue phenomenon is of little importance except when the cyclic shear stress (plus the sustained, static, shear stress) is similar to the soil undrained shear strength. Such is the case of clays with high (larger than 150%) plasticity indices (Romo, 1995). This paper presents results of an experimental programme carried out to study the variables mentioned before that affect the degradation of the shear modulus. Cyclic triaxial tests were performed for this purpose using undisturbed clay samples of Mexico City obtained from the "Central de Abasto Oficinas (CAO)" site, having an average plasticity index of 250% (Romo et al., 1989; Taboada, 1989).

From a geotechnical point of view, Mexico City has been divided in three regions (see Fig. 1): Hill zone, lake zone and transition zone. The CAO site is located not far from downtown of Mexico City, within the central lacustrine deposit zone of the valley. At the site, the geological materials consists of quaternary soft clayey and silty soil over a partially cemented gravel and sandy alluvial stratum. The clayey deposit is 40-50 m thick and it contains not only important fractions of silt, but also some thin layers of fine sand and volcanic glass at various depths.

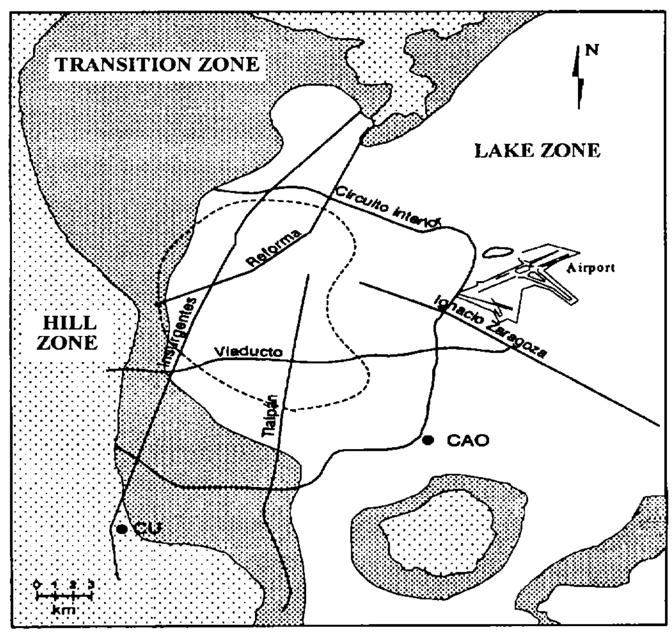

Fig. 1 Mexico City geotechnical zoning and location of CAO site

The water content w, varies from 50 to 200% for the top 10 meters; and reaches 350 to 450% between 10 m and 40 m. The samples were retrieved from depths between 32.6 and 33.1 m, having an average plasticity index of 250%.

EFFECT OF STATE OF THE SOIL ON SHEAR MODULUS

To study this effect on the shear modulus G, samples were subjected to the same magnitude of the mean effective confining pressure P'_0, applied under isotropic $K_o=1$ (CI) and anisotropic $K_o=0.5$ conditions, (CA). Where K_o is equal to the ratio between horizontal and vertical effective confinement stresses. Figure 2 shows the shear modulus normalized with the effective confining pressure at the end of the consolidation stage (G/P'_0) versus shear strain for N=15 cycles of loading. This normalization was done to eliminate the consolidation type effect (CI *versus* CA). This figure shows that for a given effective mean stress of consolidation P'o, where P'o is equal to $(\sigma_1' + \sigma_3')/2$, the shear modulus is not independent from the consolidation path. That is, samples anisotropically consolidated (CA) were more rigid than those samples isotropically consolidated (CI). On the other hand, the type of consolidation is considered negligible when the shear strain is greater than 1 %.

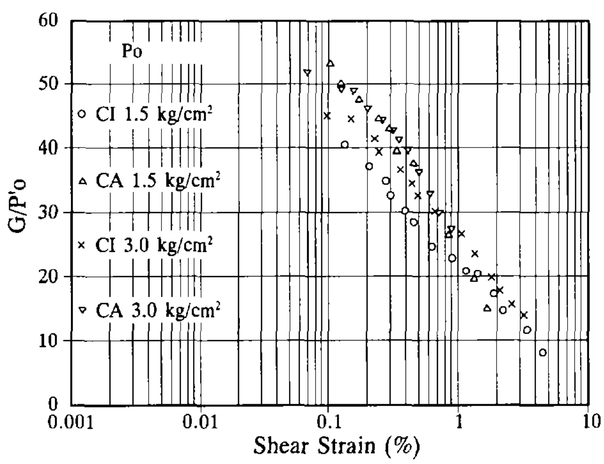

Fig. 2. Effect of the confining path (isotropic CI versus anisotropic CA)

Figure 2 shows that the consolidation stress becomes a key parameter to normalize the data due to the fact that the range of values are arranged in a narrower band. Hence, the figure can illustrate the following:

1. At shear strain lower than $\gamma \leq 0.1\%$, a constant stiffness zone is observed (see Fig. 3).
2. When shear strain is in the interval $0.1\% \leq \gamma \leq 1\%$, the shear modulus is suddenly degraded at approximately constant rate.
3. At large shear strains $\gamma > 1\%$ the type of consolidation is considered negligible on shear modulus , G.

EFFECT OF MEAN EFFECTIVE CONFINING PRESSURE ON SHEAR MODULUS

In fig. 3 it is presented the shear modulus versus shear strain for two values of the confining pressure $P'_0 = 1.5$ and 3 kg/cm^2. It is observed that as the confining pressure increases the shear modulus also increases. The same figure shows that when the confining pressure is applied under anisotropic conditions the shear modulus is higher than that obtained under isotropic conditions. Finally, the shear modulus is independent from the consolidation type when the shear strain is higher than 1%, as also shown by the results in Fig 2.

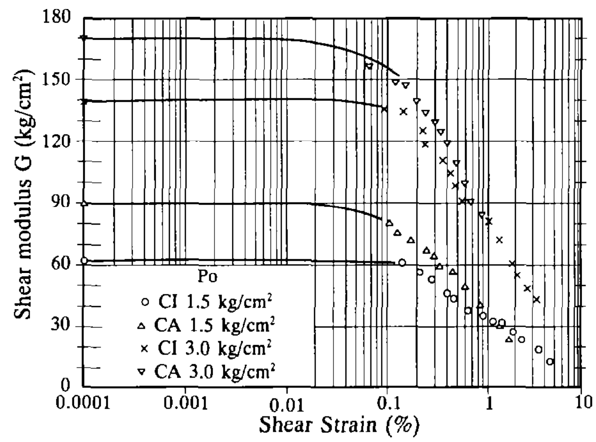

Fig. 3. Effect of confining pressure on shear modulus (isotropic CI and anisotropic CI).

EFFECT OF MAGNITUDE OF CYCLIC STRESS AND NUMBER OF TIMES IT WAS APPLYED

In Fig.4 it is shown the effect of the cyclic stress amplitude and number of times of cyclic stress applied, N, in the shear modulus of an isotropically consolidated clay sample. It is observed that degradation increases with the number of cycles and the magnitude of the cyclic stress. Figure 5 presents the same information but for the case of a sample subjected to anisotripic consolidation ($K_o=0.5$). Comparing the results of Fig. 4 and 5 it is concluded again that the consolidation type also has an influence in the shear modulus degradation. Anisotropically consolidated clays are more susceptible to degradate by the action of cyclic load.

The diminish of the shear modulus of the clay can be estimated with the following expression (Idriss et al., 1978):

$$G_N = G_0 N^{-t} \tag{1}$$

where G_N is the shear modulus for the load cycle N, G_0 is the initial shear modulus and t, is the degradation parameter.

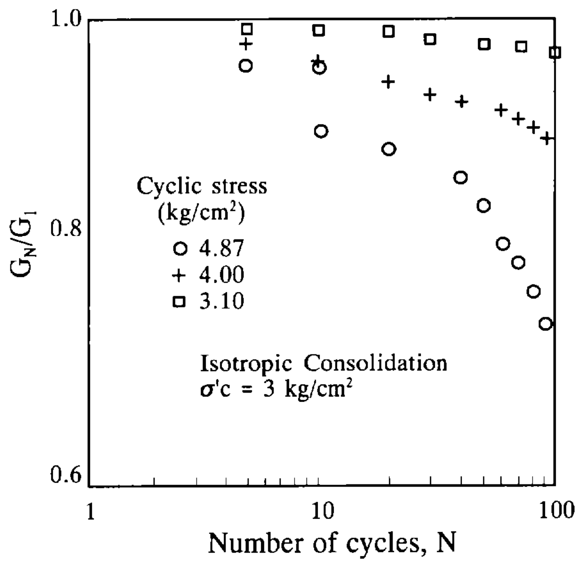

Fig. 4 Shear modulus degradation of samples isotropically consolidated.

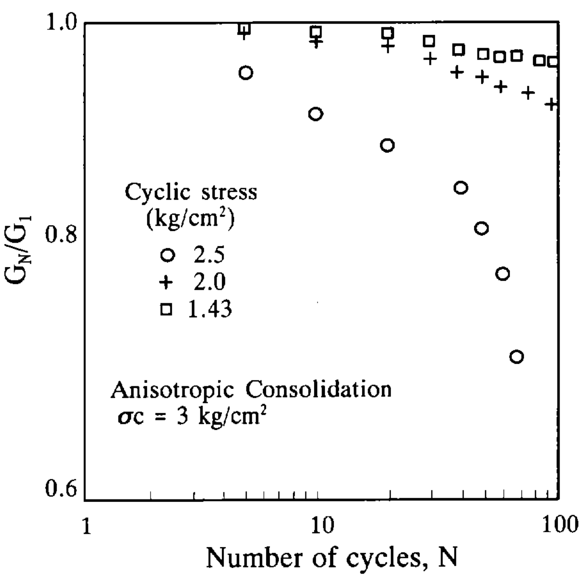

Fig. 5 Shear modulus degradation of samples anisotropically consolidated

There exists a bulk of experimental information showing that the degradation parameter depends on the magnitude of the cyclic strain, the stress path followed in the sample consolidation, over consolidation ratio and plasticity index (Romo, 1991; Dobry and Vucetic, 1987). For the normally consolidated clays studied, the parameter *t* varies with cyclic shear strain, according to the following approximate relations, as shown in Fig. 6.

Isotropic consolidation

$$t = 0.0122\ \varepsilon_c \tag{2}$$

Anisotripic consolidation

$$t = 0.0299\ \varepsilon_c \tag{3}$$

where ε_c is the cyclic axial deformation in a sample tested using a cyclic triaxial equipment. The corresponding shear strain can be found as $\gamma = 0.5(1+\nu)\varepsilon_c$, where ν is the Poisson ratio. For saturated clays it is considered that $\nu = 0.5$ (Bishop and Hight, 1977), therefore, $\gamma = 0.75\varepsilon_c$.

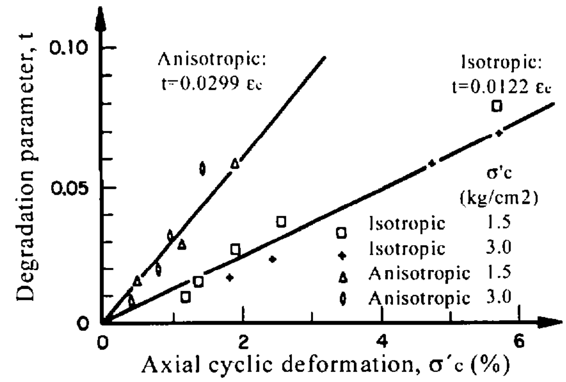

Fig. 6

Degradation parameter according to the stress path followed in the sample consolidation

The above results seem to indicate that anisotropically consolidated soil samples are less susceptible to the fatigue phenomenon (rate of degradation is lower). This is understandable since the shear stresses induced during the consolidation stage modify more significantly (than isotropic stresses) the microstructure leading to a somewhat more stable soil structure; furthermore, the reversal of cyclic shear stresses on the potential failure plane may be precluded by the initial static shear stress, thus decreasing the damaging effects on soil stiffness.

STRESS-STRAIN RESPONSE

As indicated in Fig. 7, when a soil specimen is subjected to cyclic loading it undergoes a transient cyclic deformation (cyclic deformation) and after a number of load application the sample accumulates deformations (permanent deformations). For a given soil, the former basically depends on the magnitude of the cyclic stress and the latter on the cyclic stress magnitude and loading duration.

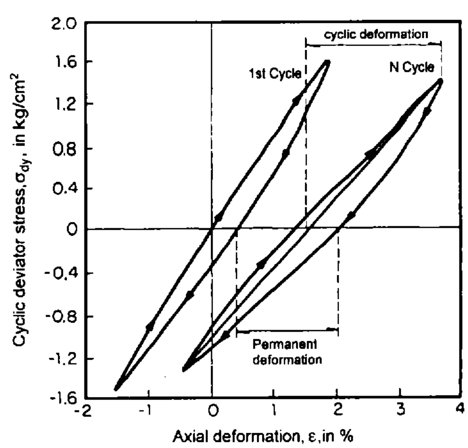

Fig. 7 Dynamic strain deformations (cyclic and permanent).

These two types of deformations are correlated because while the amplitude of the cyclic deformation increases, the permanent deformation accumulates faster. Figures 8 and 9 show the variation of the permanent deformation, ε_p, with the number of loading cycles, N, and the cyclic deformation, ε_c, for the studied clay. It is possible to observe that for an ε_c value, the deformation ε_p increases with N. These curves show a distinctive strain value where the rate of permanent deformations increases indicating the existence of a threshold of cyclic strains beyond which permanent (plastic) deformations accumulate faster. For samples anisotropically consolidated the threshold of cyclic deformation diminish considerable with respect to the corresponding samples isotropically consolidated. According to the results of Fig. 8, the critical axial strain is about 3% (the corresponding shear strain is γ =0.75x(3%)=2.25%) for the case of isotropic consolidation. And approximately γ =0.9% for anisotropic, as seen in Fig. 9.

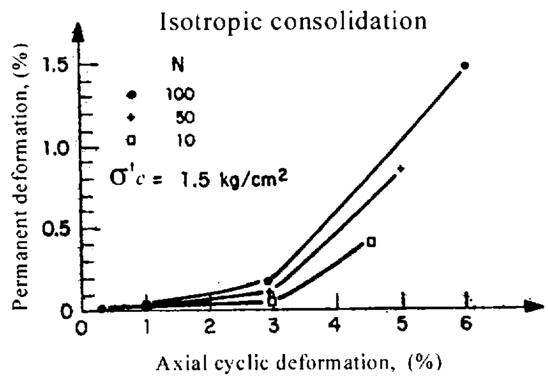

Fig. 8 Relation between cyclic and permanent deformation for samples isotropically consolidated.

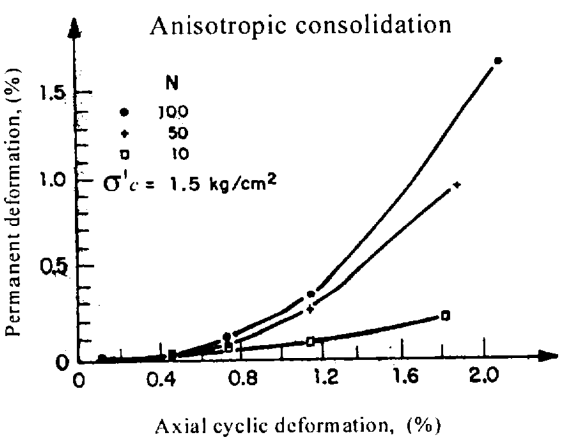

Fig. 9 Relation between cyclic and permanent deformation for samples anisotropically consolidated.

For isotropic consolidation this threshold strain (2.25%) is about one order of magnitude higher than the critical strain defined as boundary between linear and nonlinear Mexico's clay behavior (Romo, 1995). This difference between both thresholds seems to indicate that even though the clay behaves as a nonlinear material the plastic permanent deformations remain negligible until a cyclic shear deformation of the order of 2.25% is reached. This implies that in the case of highly plastic clays (like Mexico City's) permanent deformations will develop significantly only when the soil is close to failure under dynamic loading.

Such behavior is manifested also in terms of cyclic stress magnitude in Fig. 10 where the normalized total stress versus permanent strain is plotted. The results show that permanent deformations (axial deformation) accumulate only when the total shear stress (cyclic plus static) exceeds the strength of the clay. They also point out at the fact that the dynamic strength is higher than the undrained static strength (S_u). In the case of the saturated clays of Mexico City the dynamic strength may be about 60% higher than the static undrained strength.

The accumulated deformation after each loading cycle can be determined measuring the displacement of the actual hysteric cycle with respect to the previous cycle (see Fig. 7). Figure 10 shows permanent deformations caused by the application of 50 dynamic stress cycles of different amplitude. On the ordinates are plotted the values of the total deviatoric stress (cyclic, σ_{dy}, plus static or sustained, σ_d), normalized by twice the static undrained resistance, S_u. In this case the soil samples were consolidated anisotropically for two values of the octaedrical consolidation stress. Similar results were obtained for isotropic consolidation (Romo et al, 1989).

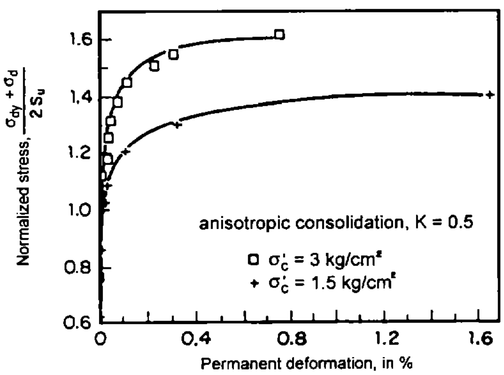

Fig. 10 Dynamic stress-plastic deformation relationship

HYPERBOLIC MODEL

Figure 10 shows that permanent deformations are important when a threshold of total stress (cyclic plus sustained) is exceeded and of the order of 20% higher than $2S_u$. Also it is observed that the total stress-permanent deformation follows a path that can be adjusted to a hyperbola. The following hyperbolic model was utilized to describe this cyclic response:

$$\sigma = \frac{\sigma_{dy} + \sigma_d}{2S_u} = \frac{\varepsilon_p}{a + b\varepsilon_p} \tag{4}$$

where a and b are soil parameters.

It is worth to point out that the inverse value of **a** is a measure of the dynamic Young modulus, E_o. Then, the shear modulus for low shear strains (10^{-4}%) can be estimated as $G_{max}=E_0/[2(1+\nu)]$, considering $\nu=0.5$, then $G_{max}=E_o/3$.

Typical values for Mexico City clay (I_p>250%) are $a=0.000317/2S_u$ and $b=0.724/2S_u$ for isotropic consolidation; and $a=0.00031/2S_u$ and $b=1.195/2S_u$ for anisotropic consolidation.

Equation (4) can be rearranged in the following form

$$\varepsilon_p = \frac{a\sigma}{1-b\sigma} \tag{5}$$

That results more convenient to evaluate the permanent deformations induced by earthquakes on a foundation. To achieve this, it is sufficient to calculate the stresses in the soil impose by the foundation (sustained stress) and then with a seismic analysis evaluate the dynamic stresses. With this information plus the resistance of the soil S_u, obtained in undrained consolidated triaxial tests and equation (5), estimations of permanent deformations induced by earthquakes are possible.

PORE WATER PRESSURE RESPONSE

Results of cyclic triaxial tests on clay samples (I_P>250%) consistently show that the dynamic pore water pressure developed is very small even near sample failure. The maximum pore water pressures recorded during cyclic loading (and after a 48-hour rest period) did not exceed 0.30(P'$_0$). These results are reasonable considering the fact that plastic deformations do develop only until large (2%) cyclic shear strains are induced in the clay sample, which means that microstructural permanent distortions are not significant within a very broad range of cyclic shear strains. Since these permanent distortions are the main cause of pore water pressure generation then it is understandable why Mexico City clays develop very low pore water pressures under dynamic loading. These results seem to indicate that the amount of dynamic pore water pressure that a given clay may develop depends on its plasticity index (and presumably I_r). This is an aspect that should be pursued thoroughly in view of the results reported for other clays (I_P<60%) where large pore water pressures accumulate during cyclic loading. I_r is the relative consistency, ($I_r = w_L - w_n/I_P$; w_L = liquid limit, w_n = natural water content).

CONCLUSIONS

Form the experimental work carried out to asses the shear modulus degradation by fatigue of Mexico city clay the following conclusions can be drawn regarding the main parameters that are important in this phenomena, namely; state of the soil, effective confining stress, magnitude of cyclic shear stress and number of times of cyclic stress applied.

1. State of the soil: for a given effective mean stress of consolidation P'o, the shear modulus is not independent from the consolidation path. That is, samples anisotropically consolidated were more rigid than those samples isotropically consolidated. On the other hand, the type of consolidation is considered negligible when the shear strain is greater than 1 %.
2. Effective mean confining pressure: as the confining pressure increases the shear modulus also increases.
3. Magnitude of cyclic shear stress and number of times of cyclic stress applied: the results seem to indicate that anisotropically consolidated soil samples are less susceptible to the fatigue phenomenon (rate of degradation is lower). This is understandable since the shear stresses induced during the consolidation stage modify more significantly (than isotropic stresses) the microstructure leading to a somewhat more stable soil structure; furthermore, the reversal of cyclic shear stresses on the potential failure plane may be precluded by the initial static shear stress, thus decreasing the damping effects on soil stiffness.
4. Regarding permanent deformation it was found a distinctive strain value where the rate of permanent deformations increases indicating the existence of a threshold of cyclic strains beyond which permanent (plastic) deformations accumulate faster. For samples anisotropically consolidated the threshold of cyclic deformation diminish considerably with respect to the corresponding samples isotropically consolidated. For isotropic consolidation this threshold strain is about one order of magnitude higher than the critical strain defined as boundary between linear and nonlinear Mexico's clay behavior. This difference between both thresholds seems to indicate that even though the clay behaves as a nonlinear material the plastic permanent deformations remain negligible until a cyclic shear deformation of the order of 2.25% is reached. This implies that in the case of highly plastic clays (like Mexico City's) permanent deformations will develop significantly only when the soil is close to failure under dynamic loading.

Finally a method to estimate permanent deformations induced on a foundation is presented based in the observed stress-strain cyclic response. To achieve this, it is suffices to calculate the stresses in the soil imposed by the foundation (sustained stress) and then with a seismic analysis evaluate the dynamic stresses. With this information plus the resistance of the soil S_u, obtained from undrained consolidated triaxial tests and using equation (5), estimations of permanent deformations induced by earthquakes are possible. It must be said that above results are not conclusive and more research is going on to clarify some aspects of shear modulus degradation and permanent deformations of Mexico City clays.

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