EFFECTS OF DEGREE OF CONSOLIDATION AND ANISOTROPIC CONSOLIDATION STRESSES ON SHEAR MODULUS AND DAMPING RATIO OF COHESIVE SOILS AT LOW STRAIN

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Abstract During consolidation process of saturated cohesive soil the soil stiffness increases. Increase of the effective stress due to dissipation of excess pore pressure causes additional stiffness of soil mass. This phenomenon has a very important effect on the behavior of saturated cohesive soils during dynamic loading. In the current investigation the changes in maximum shear modulus, G_{max} and damping ratio, D (as two important properties that affect on the dynamic behavior of saturated cohesive soils) at low shear strain are studied. Considering the properties of some typical soils which were used as a core of an embankment dam, a mixture of two different types of soil (SP + CL) was selected. A new resonant column system in shear mode was used in the shear strain range between 10^{-6} % to 10^{-3} % . Then the mentioned soil properties were found in two different degrees of consolidation and three different confining stresses. It is concluded that increase in the degree of consolidation causes increase in G_{max} and decrease in D. These changes completely depend on the consolidation stress. The results of this study can be used to determine the dynamic behavior of the core of embankment dams during dynamic loading based on its degree of consolidation in actual cases.

Key Words Shear Modulus, Damping Ratio, Cohesive Soils, Soil Dynamics

INTRODUCTION

Saturated cohesive soil deposits can be subjected to undrained cyclic loads such as explosion, traffic loading [1], earthquake [1-3] and pile driving. The dynamic response of soil under this kind of loading depends to a large extent on the cyclic stress-strain characteristics of the soil during shear. Extensive laboratory and field studies have already clarified many aspects of the influence of soil type, void ratio, initial state of effective stresses, over consolidation ratio (OCR), geologic age and other factors on these cyclic stress - strain characteristics (Dobry and Vucetic [4]; Hardin and Drnevich [5, 6]; Idriss et al. [7]; Novak and Kim [8]; Matsui et al. [9]; Lefebvre and Pfendler [10]). However, the variation of shear modulus and damping ratio due to dissipation of excess pore pressure is very important to evaluate the dynamic response of a soil mass used as the core of an embankment dam. The generated pore pressure due to construction, reservoir filling, previous dynamic loading or some other factors may get time to dissipate and increase the effective stresses increases again. This concept is very important related to seismic rehabilitation of earth dams (Marcuson III, Hadala and Ledbetter [11]). The volume reduction that tend to occur originally will then take place and more stiffness will result. The dynamic loading, time effects and the corresponding unloading and reloading of effective stresses may also change the clay structure and influence the behavior of the clay during later dynamic loading events. This paper concentrates on the following aspects:

a) The effect of undrained dynamic loading at low strain and following drainage on soil behavior during later dynamic loading,
b) The effect of degree of consolidation on shear modulus and damping ratio at low shear strain and
c) The effect of consolidation stress on shear modulus and damping ratio at low shear strain.

The study was based on resonant column laboratory tests on saturated normally remolded, anisotropic consolidated clay. In the tests the specimens were subjected to undrained dynamic loading, dissipation of pore pressure and subsequent consolidation past the initial effective stress. Then, later dynamic loading was applied to the specimen to comparing the results of the two series of dynamic loadings. The range of the applied shear strain was frame 10^{-6}% to 10^{-3}% . To make a good compatibility between test results and actual cases, the properties of some used cohesive soils as core in some embankment dams were found and based on these data the soil samples were selected as a mixture of two different coarse and fine soils (SP+CL) so that their properties were completely inside the range of actual cases. Finally the designed tests were run and the output data were collected.

PREVIOUS STUDY ON G_{max} AND D

The effect of different factors on G_{max} and D summarized by Dobry and Vucetic [4] are presented here in Table 1.

As can be seen from Table 1 the relationship between Gmax , D and degree of consolidation is not determined exactly. In this study an attempt is made to introduce a relation between Gmax , D and degree of consolidation.

Hardin and Drnevich [6] developed the following empirical equation for G_{max}:

$$G_{max}=3230 \, \frac{(2.973-e)^2}{(1+e)} \, (OCR)^{k} s_0^{1/2} \; (kN/m^2) \qquad (1)$$

Where: Gmax= maximum shear modulus; e= void ratio; OCR= overconsolidation ratio; $s_0 = (1/3)(s_v + 2s_h)$ and k as a function of plasticity index: PI, such that for PI = 0, 20, 40, 60, 80 and 100, the values of k = 0.0, 0.18, 0.31, 0.41, 0.48, 0.50, respectively (for PI > 100, k = 0.5) and for normally consolidated soils Gmax does not depend on the plasticity index. Also from Equation 1 it is concluded that G_{max} increases with OCR and s and decreases with e. On the basis of these results it is found that a normally consolidated clay subjected to undrained dynamic loading undergoes a

TABLE 1. Effect of Increase of Various Factors on Gmax, G/Gmax and Damping Ratio of Normally and Moderately Over Consolidated Clays (Dobry and Vucetic [4]).

Increasing Factor (1)	G_{max} (2)	G/G_{max} (3)	l (4)
Confining Pressure, $SÅ_0$	Increases with $SÅ_0$	Stays Constant or Increases with $SÅ_0$	Stays Constant or Decreases with $SÅ_0$
Void Ratio, e	Decreases with e	Increases with e	Decreases with e
Geologic Age, t_g	Increases with t_g	May Increase with t_g	Decrease with t_g
Cementation, c	Increases with c	May Increase with c	May Decrease with c
Overconsolidation, OCR	Increases with OCR	Not Affected	Not Affected
Plasticity Index, PI	Increases with PI if: OCR>1; Stays about Constant if: OCR=1	Increase with PI	Decrease with PI
Cyclic Strain, g_c	-----	Decreases with g_c	Increases with g_c
Strain Rate, g (Frequency of Cyclic Loading)	Increases with g	G Increases with g; G/G_{max} Probably Not Affected if G and G_{max} are Measured at Same g	Stays Constant or May Increase with g
Number of Loading Cycles, N	Decreases after N Cycles of Large g_c but Recovers Later with Time	Decreases after N Cycles of Large g_c (G_{max} Measured before N Cycles)	Not Significant for Moderate g_c and N

reduction in effective stress and then approaches the projection of the CSL as illustrated in Figure 1.

Theoretically, during a dynamic loading a clay specimen which has reached point B may behave in a similar manner to the over consolidated clay produced by unloading from point D to point B. A clay at point B can therefore be regarded as apparently over consolidated. The undrained strength of an apparently overconsolidated clay may decrease depending on the OCR associated with the distance of point B from point A. In other words, the dynamic loading under undrained conditions is similar to unloading path in ordinary consolidation test because of the reduction in effective stress. In other words unloading in consolidation test corresponds to undrained dynamic loading because of the decrease in effective stress and loading in consolidation test corresponds to drainage after dynamic loading because of the increase in effective stress. However, effective stress increases with the degree of consolidation and causes increase in stiffness of soil specimen. Therefore, it is expected to increase Gmax and

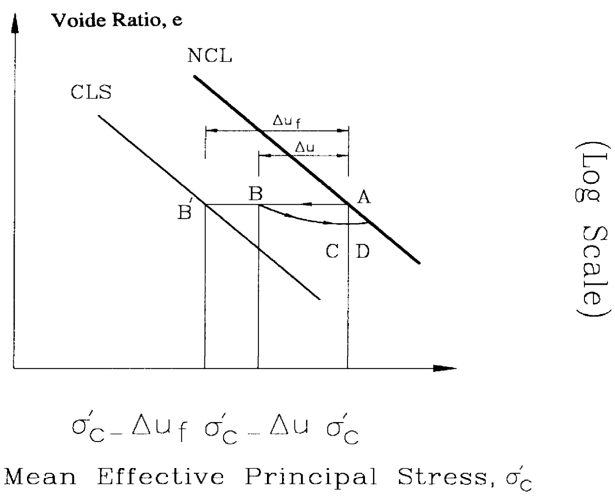

Figure 1. Void Ratio Versus Mean Principal Effective Stress Representation During Undrained Cyclic Loading Followed by Drainage.

to decrease D at low strain with the degree of consolidation.

EXPERIMENTAL PROGRAM

a- Test Materials: First of all, some properties of the cohesive materials which were used in the core of designed and constructed embankment dams were collected. The main part of these properties are tabulated in Table 2.

Then it was tried to make a suitable mixture of some coarse and fine material (CL+SP) and one of them was selected because of its practicality.

The properties of the selected mixture are shown in Table 3. Then, the soil samples were remolded and saturated. Table 5 presents the properties A the samples.

b- Test Equipment: The equipment used was Quasi-Static Torsional Shear/Resonant Column

TABLE 2. Soil Properties as Core of Embankment Dams (Actual Cases).

Dam "Core"	Sonbol Roud	Berenjestaanak	Sibooyeh	Pishin	Alimaalaat
Relative Compaction "Standard Proc."	(85 ~ 95)	(95 ~ 98)		(97 ~ 100)	(94 ~ 99)
g_{dmax} (Lab) (KN/m^3)	(18 ~ 18.4)	21		(17.6 ~ 19.1)	(17.8 ~ 18.1)
Wopt. (%)	(15.2 ~ 17)	10		(13.2 ~ 18.0)	(16.3 ~ 17.3)
g_{dmax} (site) (KN/m3)	(15.7 ~ 17.4)	(20 ~ 20.7)	(12.8 ~ 14.5)	(17.7 ~ 19.4)	(17 ~ 18)
Wopt. (%)	(14.4 ~ 23.7)	(9.5 ~ 14.1)	(8.6 ~ 20.3)	(12.1 ~ 20.7)	(8.1 ~ 14.8)
L. L.	(48 ~ 63)	(40 ~ 50)	(30 ~ 40)	(29 ~ 49)	
P. L.	(15 ~ 26)	(18 ~ 20)	(11 ~ 21)	(15 ~ 25)	
P. I.	(20 ~ 39)	(22 ~ 30)	19	(11 ~ 24)	

TABLE 3. Soil Properties.

LL	PL	PI	g_{dmax} (KN/m^3) Stand.Proc.	$W_{opt.}$ (%)	G_s	k (cm/sec)	C_{uu} (KN/m^2)	C_{cd} (KN/m^2)	fÜ$_{uu}$	fÜ$_{cd}$
32	15.7	16.3	19.6	11.6	2.7	5.2x10^{-8}	60	10	0	23

TABLE 4. Consolidation Data.

Load (KPa) Consolidation	0	27	54	108	216	432	864	1728
C_v(mm^2/min)		4.188	4.227	4.678	3.328	3.995	6.077	12.121
Time$_{100\%}$ (min)		27	24	31	30	24	18	19

TABLE 5. Initial Condition of Soil Samples.

Soil Property	W %	V_{sat} (m^3)	g_{sat} (KN/m^3)	g_{dry} (KN/m^3)	e	S %
I	18.9	8.6x10^{-4}	21.2	17.8	0.51	100
II	18.9	8.6x10^{-4}	21.1	17.7	0.51	100
III	19.2	8.6x10^{-4}	21.2	17.9	0.52	100

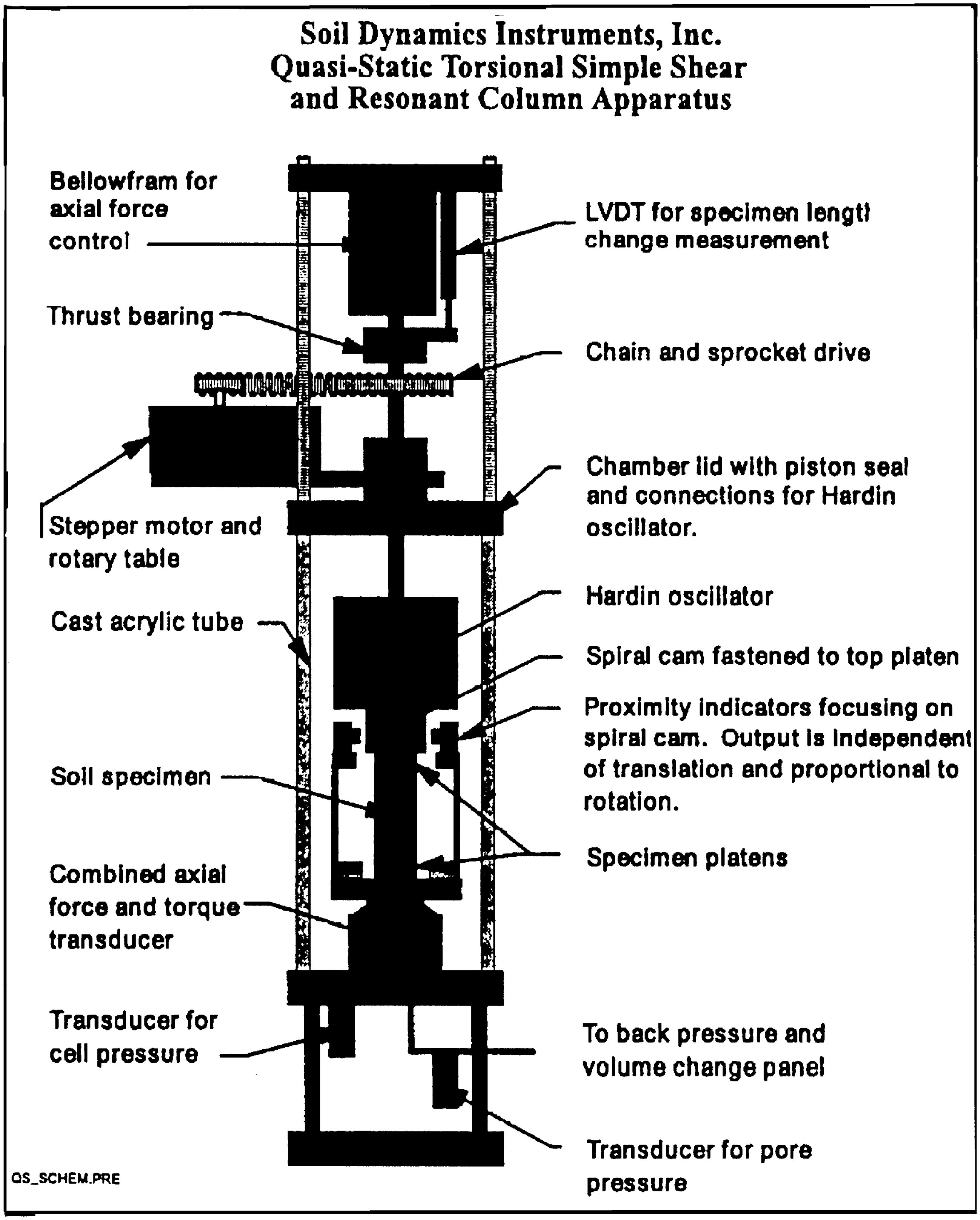

Figure 2. Quasi-static torsional simple shear/resonant column apparatus.

which was made by Prof.V.P. Drnevich. It is a combination of a new resonant column system and cyclic torsional shear (Figure 2).

The resonant column system was used in this study to evaluate G_{max} and D at low strain. This system was able to apply a shear strain range between 10^{-6} % to 10^{-3} % . Hardin oscillator was the source of shear waves and the signals could be registered by the oscilloscope. The resonant condition could be observed on the screen of the oscilloscope. Using the output of the system and with the aid of computer program written for this experiment, by the author the G_{max} and D were calculated.

c- Running the Test: Two series of tests corresponding to two degrees of consolidation were performed. The approximate range of shear strain was from 10^{-6} % to 10^{-3}% . At any stage of the test (any input voltage) the input frequency was changed until resonance condition and the outputs were registered on the basis of which G_{max} and D were calculated.

After setting the sample in the chamber the drainage valves were opened and the anisotropic condition of loads were applied to the specimen and the first series of dynamic loading under undrained condition was applied after one hour. Again the drainage valves were opened and consolidation was continued until 24 hours, when the second series of dynamic loading under undrained condition was applied. It should be noticed that based on the following simple calculation the test after one hour corresponded to 20% degree of consolidation and the test after 24 hours corresponded to 90% degree of consolidation .

$$C_{v(av.)} = 5.51 \text{ mm}^2/\text{min}$$

$$H_{dr} = \frac{\text{Sample Height}}{2} = \frac{19.5}{2} = 9.75\text{cm} = 97.5\text{mm}$$

$$1)\ t_1 = 1 \text{ hour} = 60 \text{ min} \Rightarrow T_v = \frac{c_v t}{H^2_{dr}}$$

$$= \frac{5.51*60}{(97.5)^2} = 0.035 \Rightarrow U = 20\ \%$$

$$2)\ t_2 = 24 \text{ hours} = 1440 \text{ min} \Rightarrow T_v = \frac{c_v t}{H^2_{dr}}$$

$$= \frac{5.51*60*24}{(97.5)^2} = 0.835 \Rightarrow U=90\%$$

Therefore, samples were tested at two different degrees of consolidation. To simulate the actual case as the core of embankment dam the samples were consolidated anisotropically, Table 6.

TEST RESULTS

Using resonant column system, two important parameters, shear modulus, G and damping ratio, D were obtained which could be used to define the soil behavior during the dynamic loading. Four categories of curves are presented as test results:

a- Shear modulus versus shear strain at the same consolidation stress but different degrees of consolidation, Figures 3 to 5.

b- Damping ratio versus shear strain at the same consolidation stress but different degrees of consolidation (Figures 6 to 8).

c- Shear modulus versus shear strain at the same degree of consolidation but different consolidation stresses, (Figures 9 and 10).

d- Damping ratio versus shear strain at the same degree of consolidation but different consolidation stresses (Figures 11 and 12).

TABLE 6. Anisotropic Effective Stresses

Effective Stresses Sample No.	S_1 (KN/m^2)	S_3 (KN/m^2)	K = $\frac{S_3}{S_1}$
I	45	36	0.8
II	135	108	0.8
III	180	144	0.8

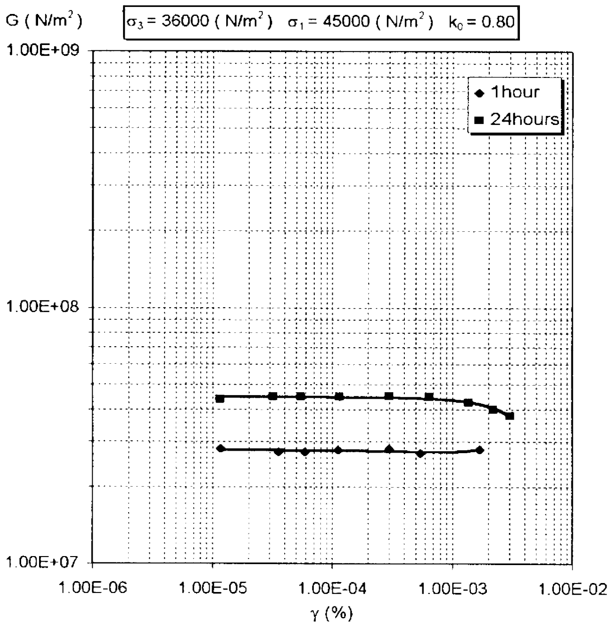

Figure 3. Shear Modulus Versus Shear Strain at S_3 = 36000 N/m^2.

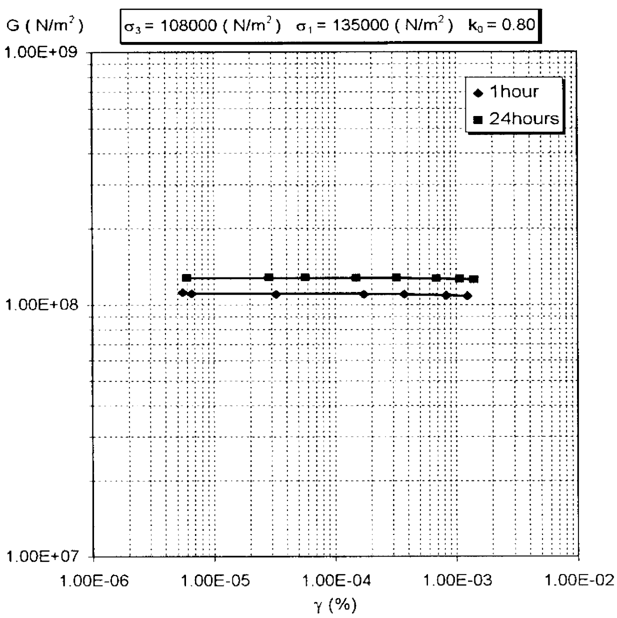

Figure 4. Shear Modulus Versus Shear Strain at S_3 = 108000 N/m^2.

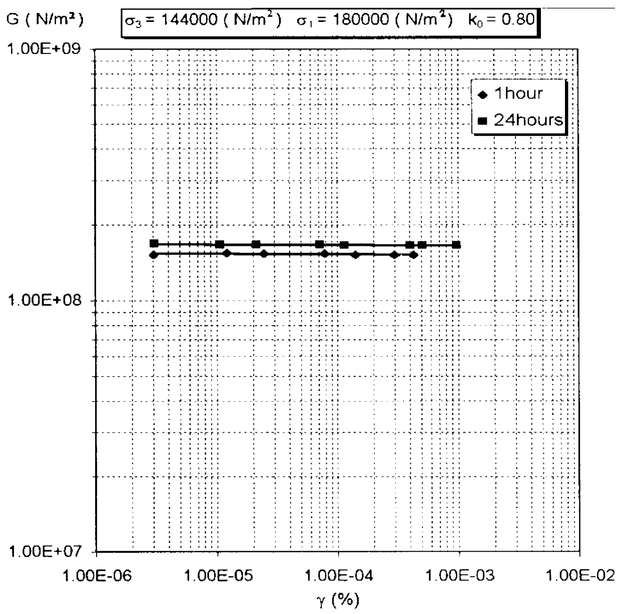

Figure 5. Shear Modulus Versus Shear Strain at S_3 = 144000 N/m^2.

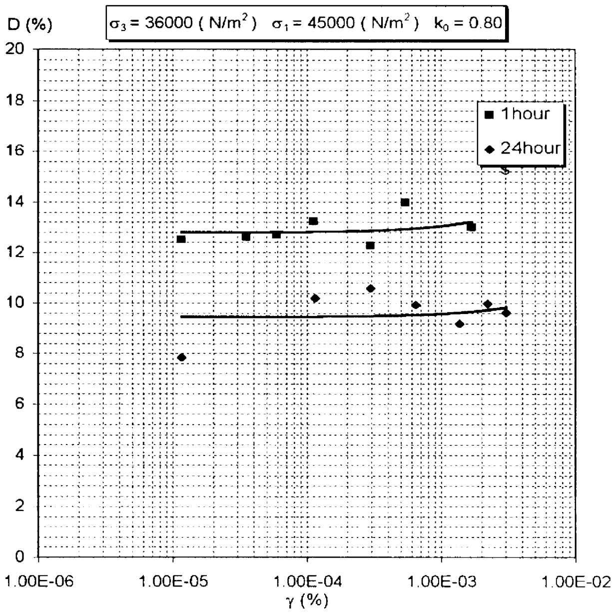

Figure 6. Damping Ratio Versus Shear Strain at S_3 = 36000 N/m^2.

DISCUSSION

a- The effect of the degree of consolidation onshear modulus:

As can be seen from Figures 3 to 5, shear modulus increases with the degree of consolidation as there is an increment in shear modulus in each curve. However, the relative difference between shear modulus at two degrees of consolidation can be calculated as

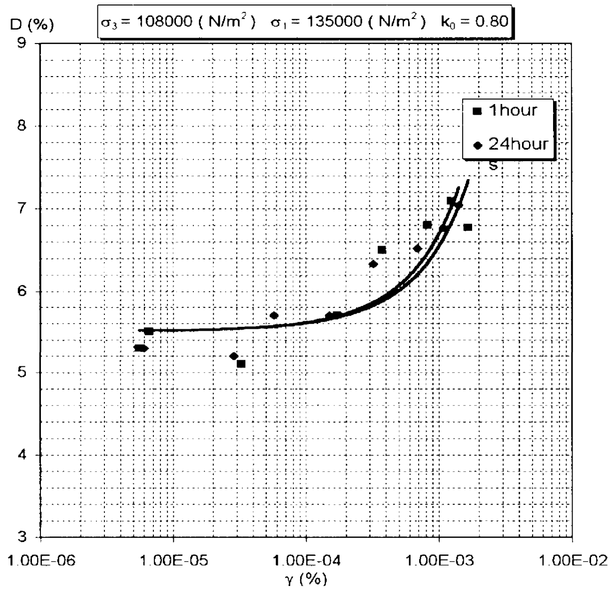

Figure 7. Damping Ratio Versus Shear Strain at S_3 = 108000 N/m^2.

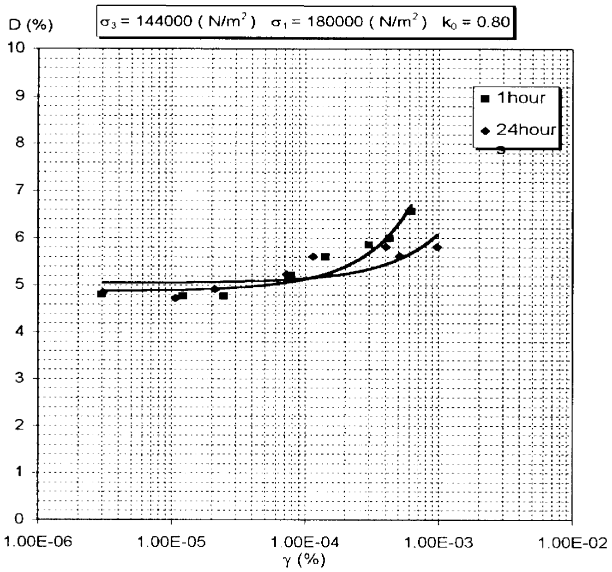

Figure 8. Damping Ratio Versus Shear Strain at S_3 = 144000 N/m^2.

follows:

at S_3 = 36 KN/m^2 :

$$DG\ \%_{(av.)} = \frac{G_{(24hours)} - G_{(1hours)}}{G_{(24hours)}} = 27.6\ \%$$

at S_3 = 108 KN/m^2 :

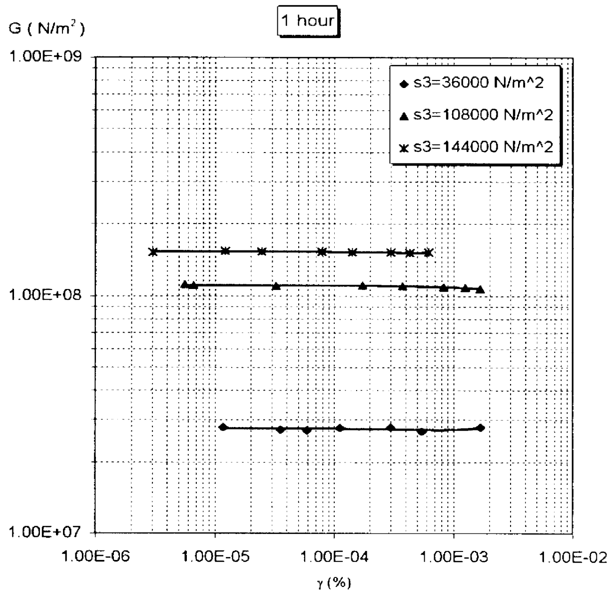

Figure 9. Shear Modulus Versus Shear Strain at Degree of Consolidation = 20 %.

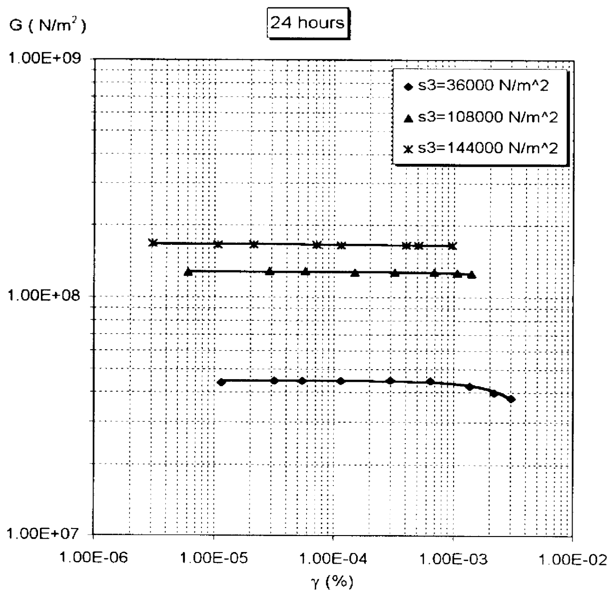

Figure 10. Shear Modulus Versus Shear Strain at Degree of Consolidation = 90 %.

$$DG\ \%_{(av.)} = \frac{G_{(24hours)} - G_{(1hours)}}{G_{(24hours)}} = 12.7\ \%$$

at S_3 = 144 KN/m^2 :

$$DG\ \%_{(av.)} = \frac{G_{(24hours)} - G_{(1hours)}}{G_{(24hours)}} = 8.4\ \%$$

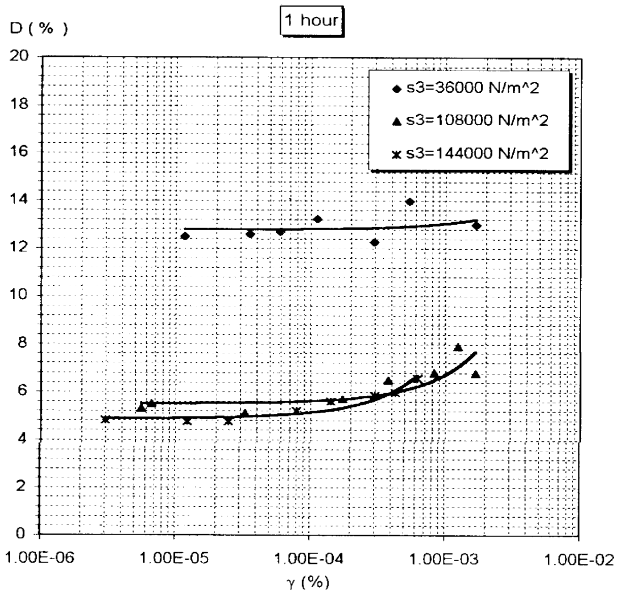

Figure 11. Damping Ratio Versus Shear Strain at Degree of Consolidation = 20 %.

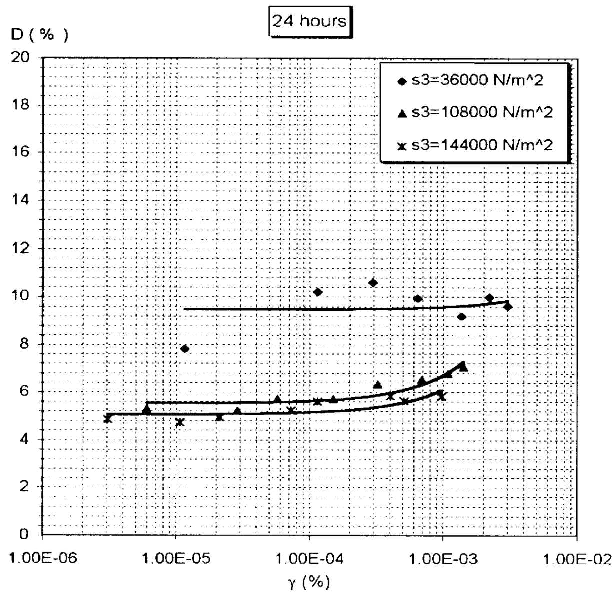

Figure 12. Damping Ratio Versus Shear Strain at Degree of Consolidation = 90 %.

It can be observed that in all the Figures the maximum shear modulus corresponded to 24 hours (degree of consolidation = 90 percent) is greater than the maximum shear modulus corresponding to 1 hour (degree of consolidation = 20 percent).

b- The effect of the degree of consolidation on damping ratio:

As can be seen from Figures 6 to 8, damping ratio decreases with the degree of consolidation as there is a decrement in damping ratio in each curve. However, the relative difference between damping ratio at two degrees of consolidation can be calculated as follows:

at s_3 = 36 KN/m^2 :

$$DD\ \%_{(av.)} = \frac{D_{(24hours)} - D_{(1hours)}}{D_{(24hours)}} = 22.5\ \%$$

at s_3 = 108 KN/m^2 :

$$DD\ \%_{(av.)} = \frac{D_{(24hours)} - D_{(1hours)}}{D_{(24hours)}} = 1.6\ \%$$

at s_3 = 144 KN/m^2 :

$$DD\ \%_{(av.)} = \frac{D_{(24hours)} - D_{(1hours)}}{D_{(24hours)}} = 1.5\ \%$$

It can be concluded that in all the Figures, the damping ratio corresponding to 1 hour (degree of consolidation = 20 percent) is greater than the value of damping ratio corresponding to 24 hours (degree of consolidation = 90 percent).

It should be noted that in both cases a and b, the average values of shear modulus and damping ratio are used and G and D are two indices for qualitative presentation of changes in G and D. That is, they are not quantitative parameters.

c- The effect of consolidation stresses on shear modulus.

As can be observed from Figures 9 and 10, shear modulus increases with consolidation stress as there is an increment in shear modulus in each curve. However, the effect of consolidation stresses on shear modulus decreases with increase in the degree of consolidation.

d- The effect of consolidation stresses on damping ratio.

It can be seen from Figures 11 and 12 that damping ratio decreases with consolidation stress as a decrement is observed in damping ratio in each curve. However, the effect of consolidation stresses on shear modulus decreases with increase in the degree of consolidation.

CONCLUSION

1. Undrained dynamic loading and drainage may make normally consolidated clay more resistant to later undrained dynamic loading.
2. Degree of Consolidation has a very important effect on the relationship between shear stress-shear strain in cohesive soils due to its effects on shear modulus and damping ratio.
3. During the life of a cohesive soil mass such as the core of an embankment dam, the degree of consolidation increases and thus shear modulus and damping ratio will not be constant. Also, the response of a soil mass to a dynamic loading completely depends on its shear modulus and damping ratio. Therefore, the response of the soil to even two equal dynamic loadings (e.g., two earthquakes) will not be the same.
4. G_{max} and D are the two parameters which completely depend on the degree of consolidation. Therefore, in formulas such as Equation 1, it is necessary to consider the effect of the degree of consolidation.

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