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CHARACTERISING THE NON LINEARITIES OF LACUSTRINE CLAYS IN THE GRENOBLE BASIN

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ABSTRACT - The thick layer of post-glacial lacustrine silt and clay in the Grenoble basin may be subject to dynamic amplifications during a seismic event. Since amplification is linked to the strain-dependent nonlinear behaviour of a soil, the objective of this study is to determine the nonlinear properties of the lacustrine layer. To this end, a cyclic triaxial testing programme on intact lacustrine core samples was carried out in order to establish modulus degradation and damping curves at confining pressures representing a depth of 40-50 metres. In addition, a series of soil classification tests were carried out on disturbed lacustrine samples at greater depths, confirming the homogeneous nature of the layer. Information on the non linear response should lead to a better prediction of the seismic response of the Grenoble basin during an earthquake.

1. Introduction

The Grenoble basin is located in the alpine valley which runs northeast-southwest between the cities of Grenoble and Chambery, known as the Grésivaudan valley. It is filled with hundreds of metres of soft sedimentary fill. In the event of a Grenoble design earthquake (magnitude 5,5), the seismic waves propagating upwards from the bedrock could undergo strong amplifications due to this very thick sedimentary layer. The amplification problem is considered more particularly in the lacustrine clay/silt layer, situated between approximately 40 and 400 m depth below the surface. The risk of dynamic amplification depends on the thickness of the layer, and also on the shear modulus, G and the damping coefficient, D, of the soil. As a function of the cyclic shear stress level, γ, in the medium to large strain domain, G tends to decrease for all soils and D tends to increase (therefore reducing dynamic amplification). This kind of behaviour is called non-linear behaviour, and thus the dynamic amplification problem is linked to the non-linear behaviour of a soil. Therefore, in order to undertake a study of the seismic response of this lacustrine layer, its non-linear behaviour must first be defined.

During cyclic loading on a typical soil, dissipative loops known as hysteresis loops are observed in shear stress-shear strain space. The two important characteristics of a hysteresis loop are its inclination and its breadth. G, the shear modulus, is determined by the average inclination of the loop, while D, the damping coefficient is determined by the calculation of two different areas associated with the hysteresis loop: the area of the loop itself, ΔW, representing the energy dissipated during one cycle of loading, and the triangular area known as the maximum elastic strain energy, W. Figure 1 shows the details of the hysteresis loop.

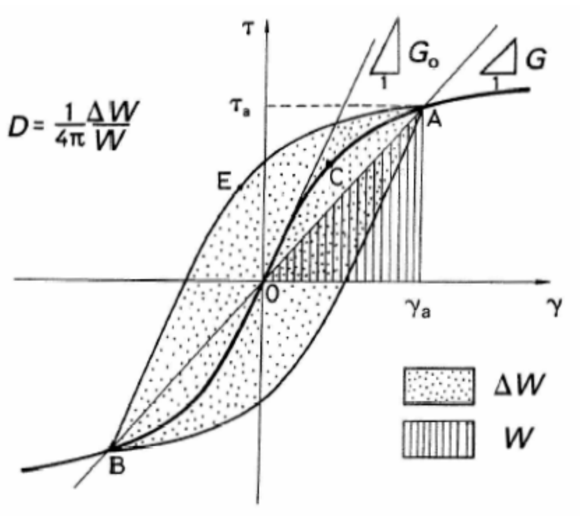

Figure 1. Typical hysteresis loop showing calculation of G and D

G decreases with shear strain amplitude, while the dissipated energy, and therefore D, increase.

The non linear behaviour of a soil undergoing cyclic loading can be displayed as the locus of points G or D as a function of shear strain amplitude, γ. G is typically normalised by the small strain shear modulus, G_{max}, so that there is no dependence on the confining pressure, σ'_0, or the relative density, D_r, for a particular type of soil. For plastic soils such as clays and silts, Vucetic and Dobry (1991) established such curves as a function of the plasticity index, PI. They showed that, the higher the PI, the larger the range of γ values for which G/G_{max} is equal to 1 and the more gradually the value of D increases. In other words, the higher the PI, the more “linear” the soil behaviour, and the greater the dynamic amplification will be.

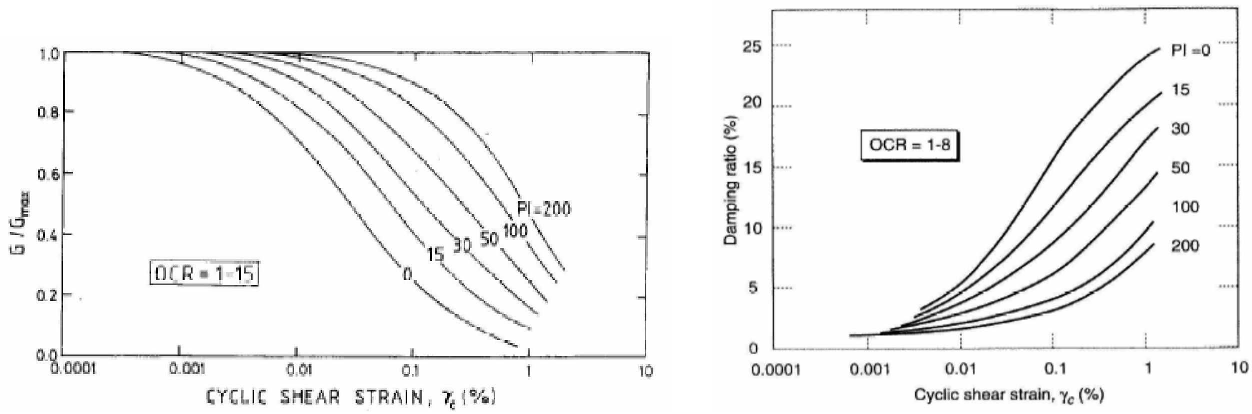

Figure 2a). Modulus degradation curves and b) Damping coefficient curves for plastic soils as a function of PI, after Vucetic and Dobry (1991).

Similar curves have been established for non-plastic, cohesionless soils such as sands and gravels (Seed et al. 1986, Iwasaki et al. 1978, Seed and Idriss 1970). In this case, the governing parameter for the nonlinear behaviour is the confining pressure, σ'_0: the higher the confining pressure, the more “linear” the soil behaviour.

The aim of this study is to establish similar nonlinearity curves for the lacustrine silt/clay layer in the Grenoble basin, by cyclic laboratory tests on intact samples as well as general soil classification tests (plasticity indices, mineralogy, grain size distribution) on lacustrine samples at different depths. The results will go towards a more precise determination of the likely dynamic amplification of the lacustrine layer: in the case of a very non linear behaviour, for example, the risk of dynamic amplification would be greatly reduced (D increases rapidly with γ).

2. Classification of the lacustrine clays at two sites: Crolles and Montbonnot

2.1 Test sites

Clay samples were taken from two sites in the Grenoble basin in order to perform a series of laboratory tests establishing the principal physical characteristics of the soil.

Site 1: Montbonnot

In 1999, a non cored borehole was drilled by the IPSN (The French Institute for Nuclear Safety and Protection) at the village of Montbonnot (Nicoud et al., 2002). The borehole penetrated 534 m of soft material before reaching the basement rock, and enabled identification of the approximately 450-m thick lacustrine sediment layer with fluviatile deposits closer to the surface. This identification was carried out mainly by geophysical testing methods, as well as visual assessment of the cuttings along the length of the borehole.

For this study, several cuttings (disturbed samples) corresponding to depths between 103 m and 430 m along the 564-m long borehole were taken for laboratory testing.

Site 2: Crolles

In 2005, a borehole with some undisturbed sampling was carried out in the industrial zone of the village Crolles by the geotechnical firm Fondasol. The borehole reached 48,5 m depth, and core samples were taken from 19 – 24 m (superficial clay layer, between 4 and 27 m depth) and from 44 – 48,5 m (beginning of the lacustrine layer). For this study, only testing of the lacustrine clay samples has been undertaken, namely on material from the top of the core sample 44 – 45,5m and from the top of the core sample 47 – 48,5 m.

2.2 Testing programme

3 principal types of test were carried out for the purposes of soil classification: Atterberg limit tests, grain size distribution tests and mineralogy analyses (the latter performed at the University of Science and Technology in Lille). For each set of tests, results for Montbonnot samples between 103 and 430 m and Crolles samples between 44 and 48 m were compared in order to confirm to some extent the vertical and lateral homogeneity of the lacustrine clay layer.

2.2.1 Atterberg limit testing

14 samples corresponding to depths between 103 and 282 m were taken from the Montbonnot cuttings for Atterberg limit testing, as well as two samples corresponding to 44 m and 47 m depth from the Crolles samples. The liquid limits (wl) were obtained by the English cone test, while the plastic limits (wp) were obtained by the rolling method. A correction has been made on the values of wl, (and thus the plasticity index values, PI) in order to obtain the equivalent values that would have been obtained by the Casagrande apparatus. This correction enables the samples to be classified on the British plasticity chart.

The results are presented in the table below. Results for the Montbonnot samples have been grouped into depth intervals for which the wl and PI values were relatively constant.

Table I. Liquid and plastic limits and plasticity indices for samples at Crolles and Montbonnot

Site	Depth (m)	wl (%)	wp (%)	PI (%)
Montbonnot	100 - 120	35,3	20,8	14,5
Montbonnot	120 - 160	33,7	21,3	12,4
Montbonnot	160 - 200	36,0	19,6	16,4
Montbonnot	240 - 282	30,9	17,8	13,1
Crolles	44	33,2	23,4	9,8
Crolles	47	42,0	27,4	14,6

It can be seen that the PI values are reasonably homogeneous and correspond to a soil of low to intermediate plasticity. The values of wl and PI were plotted on the British plasticity chart below.

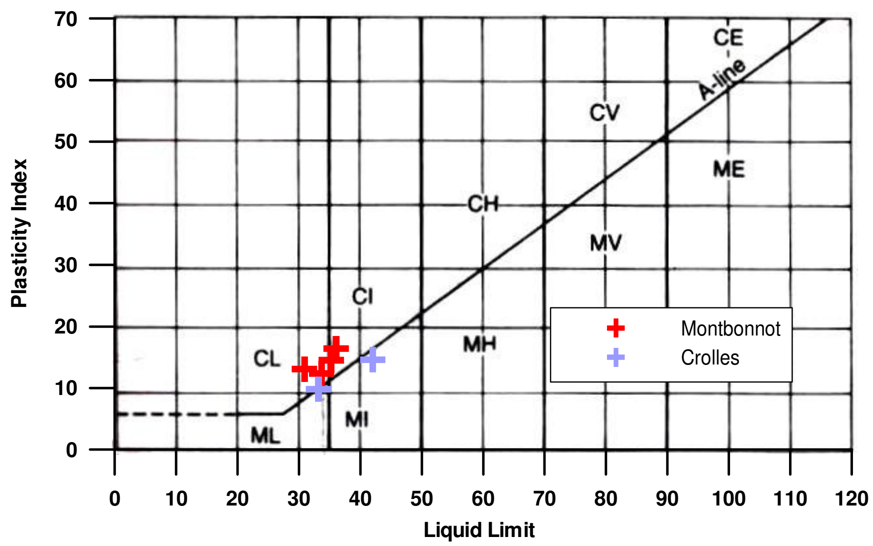

Figure 3. Position of Montbonnot and Crolles samples on the British plasticity chart.

According to the British system, the samples from Montbonnot are classified as low and intermediate plasticity clays, while the samples from Crolles are classified as low and intermediate plasticity silts. All values plot very close to the A-line, which is the line differentiating silts from clays.

Using the PI values, an initial estimation of the likely non linear behaviour of the lacustrine clay/silt layer can be made. Predicted degradation curves are situated with respect to the modulus degradation curves established by Vucetic and Dobry (1991) for saturated fine-grained soil deposits.

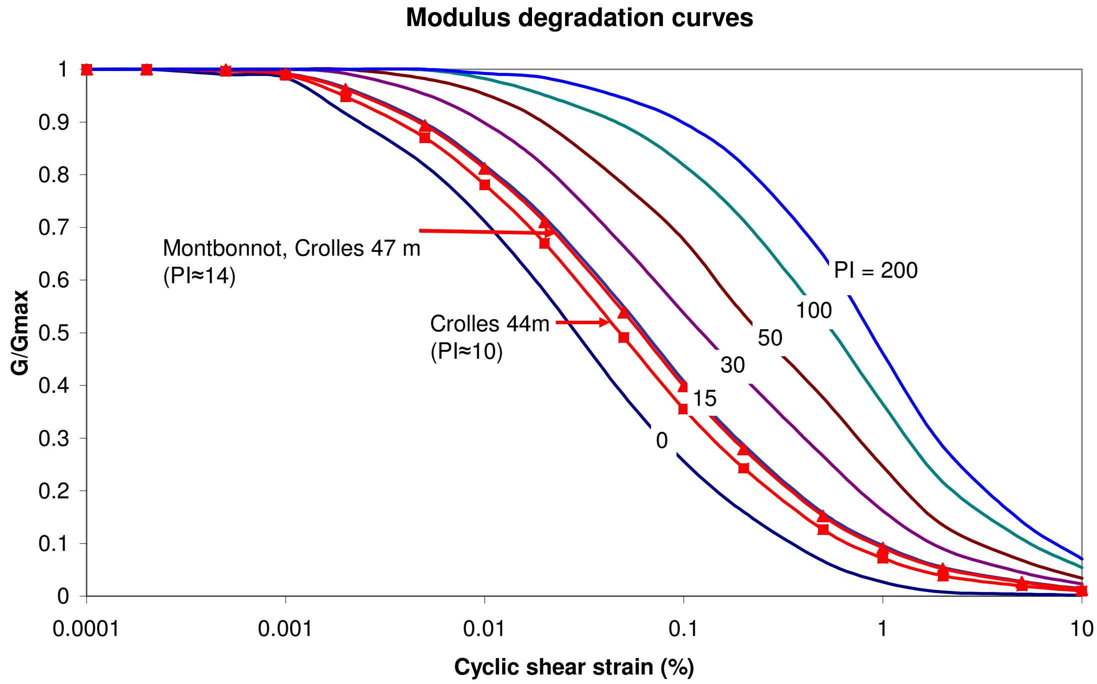

Figure 4. Predicted non linear behaviour of lacustrine silts/clays with respect to modulus degradation curves after Vucetic and Dobry (1991).

The PIs obtained from Atterberg limit tests thus predict quite strongly non linear behaviour of the lacustrine material, which means that the soil damping would increase significantly with cyclic shear strain level.

2.2.2 Grain size analysis

Grain size analysis by laser diffraction was carried out for 14 samples from Montbonnot (depths from 103 to 430 m) and 2 samples from Crolles (44 and 47 m). Grain size distribution curves were obtained for each sample, and the results were as follows:

Table II. Results of grain size analysis for Crolles and Montbonnot samples

Site	% Clay (d < 0,002 mm)	% Silt (0,002 < d < 0,06 mm)	% Sand (d > 0,06 mm)
Crolles (44, 47 m)	7,8	90,0	2,2
Montbonnot (103; 121-181; 430 m)	6,3	86,0	7,7
Montbonnot (109; 198-326 m)	3,6	71,4	25,0

At the outset of this study, the samples were considered to be principally clay with small fractions of silt and sand. However, the grain size analyses clearly show that the principal constituent of the lacustrine soil samples at both sites is silt, and that the clay fraction is constantly below 8%. There are passages in the Montbonnot profile where the sand fraction is more significant, but silt content does not drop below 65 %.

The Atterberg limit tests remain valid, as the samples are cohesive and fine grained. The grain size distribution is reasonably homogeneous for the entire depth of the Montbonnot profile, and corresponds well to the Crolles samples.

2.2.3 Mineralogy analysis

To complete the soil classification programme, an analysis by x-ray diffraction was performed on the clay fraction (d < 0,002 mm) of three lacustrine samples (Crolles 47 m; Montbonnot 282 and 430 m). The principal clay minerals present were as follows:

Table III. Clay minerals present in the clay fraction of 3 lacustrine samples

Sample	% Swelling minerals	% Illite	% Chlorite
Crolles 47 m		66	34
Montbonnot 282 m	10	56	34
Montbonnot 430 m	3	56	41

The clay mineral contents for the three samples are very similar: principally illite with a significant fraction of chlorite. By referring to correlations made by Holtz and Kovacs (1981), the validity of the points plotted for each sample on the British plasticity chart in figure 3 is confirmed: they plot within the typical range of values for illite clay minerals.

3. Cyclic triaxial testing programme: Crolles samples

3.1 Description of the tests

Using the sample cored between 47 and 48,5 m at Crolles, two separate cyclic triaxial tests were carried out with the aim of establishing the modulus degradation and material damping curves for the lacustrine silt. The tests were performed at laboratoire 3S, using a cyclic triaxial press.

To prepare a cylindrical sample, a part of the core sample (including PVC casing) was cut off. It was observed that significant sample disturbance had occurred during coring and therefore, in order to trim the sample to its required dimensions for triaxial testing, it had to be consolidated isotropically at a confining pressure, σ'_0, of 200 kPa. Once the sample was trimmed to the appropriate dimensions (height 140 mm and diameter 70 mm), it was placed in the test cell, saturated and then consolidated isotropically to $\sigma'_0 = \sigma'_3 = 450$ kPa, corresponding to the in situ confining pressure of the material.

The sample then underwent several series of undrained, strain-controlled cycles, each series having a different maximum imposed strain value, ε1. The value for the smallest stable strain series was determined as a function of the minimum velocity of the press. The objective was to have 6 strain series as follows:

- Series 1: $\varepsilon 1 = \pm 5 \cdot 10^{-5}$
- Series 2: $\varepsilon 1 = \pm 1 \cdot 10^{-4}$
- Series 3: $\varepsilon 1 = \pm 5 \cdot 10^{-4}$
- Series 4: $\varepsilon 1 = \pm 1 \cdot 10^{-3}$
- Series 5: $\varepsilon 1 = \pm 5 \cdot 10^{-3}$
- Series 6: $\varepsilon 1 = \pm 1 \cdot 10^{-2}$

For each strain series, the sample underwent a certain number of strain “half cycles”; in other words, a vertical displacement was imposed on the top of the sample until the positive limiting strain value was reached for the particular series (half cycle number one), at which point the sample was unloaded and reloaded in the opposite direction until reaching the negative limiting strain value (half cycle number two). 6 to 10 half cycles were performed for each strain series. Strain, deviator stress (q) and excess pore water pressure (Δu) were measured continuously during the cyclic undrained triaxial tests.

The hysteresis loops for each strain series were plotted (plots of the major principal stress, σ1 = q + σ3, as a function of ε1) and the secant Young's modulus, E, was determined for each series as the average inclination of the hysteresis loop. G, the corresponding shear modulus, was then determined using the elastic relationship:

$$G = \frac{E}{2(1+\nu)} \tag{1}$$

Where v is Poisson's ratio, equal to 0,5 in the undrained case. The distortion, or cyclic shear strain, γ, was related to the axial shear strain ε1 by (undrained case):

$$\gamma = \frac{\varepsilon_1 - \varepsilon_3}{2} = \frac{3*\varepsilon_1}{4} \tag{2}$$

Finally, the material damping coefficient, D, was calculated for each strain series by calculating the area of the hysteresis loop and dividing it by 4π multiplied by the maximum strain energy, as shown in Figure 1.

Plots of G/Gmax and D (%) as a function of γ (%) were thus established for the lacustrine silt at Crolles. At this stage, G_{max}, the elastic small-strain shear modulus, has not been determined experimentally, and was estimated by extrapolation of the G-γ curve to the linear threshold value of γ, below which G is considered to be the small-strain shear modulus G_{max}.

3.2 Test results

For the two cyclic undrained triaxial tests performed, adjustments had to be made in order to determine the real strain amplitude for each strain series, as the amplitudes did not correspond to those imposed at the beginning of the tests. The table below summarises, for the two tests performed, the amplitude of each strain series with respect to the amplitude imposed.

Table IV. Summary of strain amplitudes for each strain series for the two cyclic non drained triaxial tests (Crolles lacustrine silt samples).

Test	**1 (December 2005)**					**2 (February 2006)**				
	Series 1	**Series 2**	**Series 3**	**Series 4**	**Series 5**	**Series 1**	**Series 2**	**Series 3**	**Series 4**	**Series 5**
ε1 imposed	5E-5	1E-4	5E-4	1E-3	5E-3	5E-5	1E-4	5E-4	1E-3	5E-3
ε1 real	1,3E-4	1,6E-4	5,8E-4	1,1E-3	5,1E-3	4,9E-5	1,1E-4	5,0E-4	1,1E-3	4,2E-3

The modulus degradation curves for both tests are shown in the Figure 5 below. They are plotted with the reference curve for a cohesive, fine-grained saturated soil having a PI of 15% (Vucetic and Dobry, 1991).

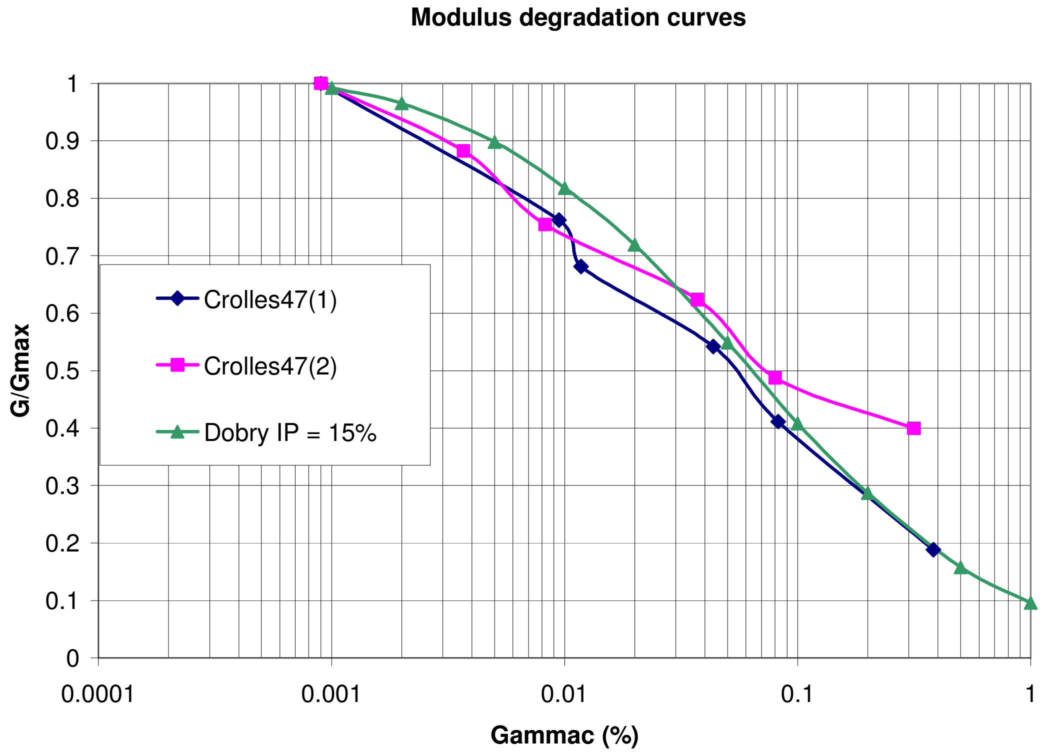

Figure 5. Experimental modulus degradation curves for the Crolles lacustrine silt samples, with respect to curve PI = 15% established by Vucetic and Dobry (1991).

Looking first at experimental curve 1, it can be seen that in general the curve fits well with the nonlinearity curve corresponding to PI = 15%, (note that the PI obtained for the sample was 14,6 %) indicating a strong non linear behaviour of the lacustrine silt. The first two points on the graph, corresponding to the first two strain series in the cyclic test, seem to underestimate G with respect to Vucetic and Dobry; this is probably due to the instabilities in the press data acquisition system at these lower strain values. Experimental curve 2 shows much the same trend as curve 1, apart from the last, erroneous point which seems to overestimate G with respect to Vucetic and Dobry.

Figure 4 shows the experimental damping curves obtained with respect to the damping curves established by Vucetic and Dobry (PI from 0 to 30%).

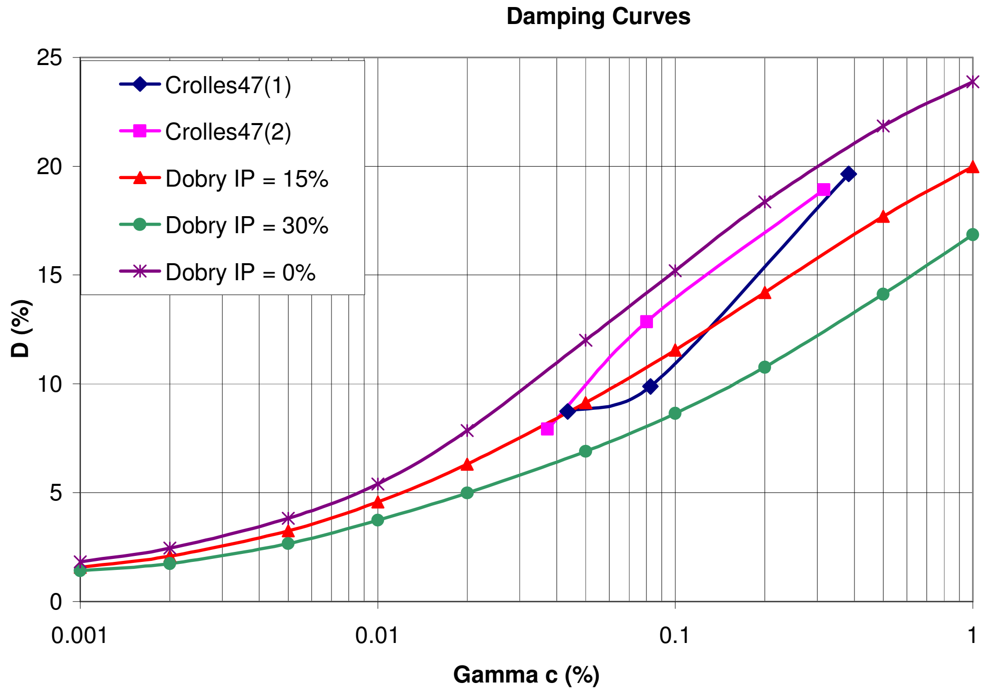

Figure 6. Experimental damping curves for the Crolles lacustrine silt samples, with respect to curves established by Vucetic and Dobry (1991).

The curves for test 1 and test 2 plot within the plasticity range 0 – 15%, and thus the damping ratio increases rapidly with shear strain, indicating (as predicted) a strong non linear behaviour of the lacustrine silt. Due to instabilities at low values of γ, it was not possible to calculate D for the first two strain series.

The values of D obtained experimentally for Crolles agree reasonably well with those obtained for some clayey silts and silty clays having similar PIs (Kim and Novak, 1981).

In general, the experimental curves reflect significant non linear behaviour of the Crolles lacustrine silt, and they also support the hypothesis of non linearity made on the basis of the plasticity indices determined by the Atterberg limit tests. Further cyclic triaxial testing should be carried out, however, in order to obtain G and D values for a wider range of cyclic shear strains. In addition, Bender element test results giving G_{max} for the Crolles silt (at σ'_0 = 450 kPa) should provide more precise modulus degradation curves.

Due to a lack of undisturbed core samples, cyclic triaxial tests on lacustrine silt at greater depths, such as at Montbonnot, are currently not possible. Based on the results from section 2, where the samples from Crolles and Montbonnot were shown to be of almost identical plasticity, grain size distribution and mineralogy, one approach is to extrapolate the hypothesis of strong non linearity to all depths in the lacustrine layer from 40 to 400 or 500 m depth. In this case, there would be little risk of dynamic amplification in the layer during an earthquake when the shear strains became large enough ($\gamma > 9*10^{-6}$) for non linear behaviour to appear. However, the effect of confining pressures of the order of 4000 kPa (corresponding to approximately 400 m depth) on the non linear behaviour of fine grained, low plasticity cohesive soils is not well documented in current literature. Whether plasticity index remains the governing parameter for non linear behaviour of fine grained soils at all confining pressures is as yet uncertain.

Authors such as Seed, Idriss et al (1970, 1986), Hardin and Drnevich (1972) and Iwasaki et al (1978) have shown the trend of decreasing non linear behaviour of cohesionless soils such as sands and gravels (PI = 0) with increasing confining pressure.

However, there does not appear to be any data for confining pressures greater than 400 kPa in currently available literature.

Thus it is difficult to predict the non linear behaviour of the lacustrine silt at greater depths in the Grenoble basin. If there is no effect of confining pressure on the behaviour of the silt, the lacustrine layer can be supposed to behave non-linearly under dynamic loading, thereby reducing the risk of large shaking amplifications during an earthquake. However, since it is a material of low plasticity, and thus approaching a more sand-like behaviour, the effect of the confining pressure may be to reduce the non linear behaviour of the silt and thus increase the risk of amplifications in the lacustrine layer.

4. Conclusions

In order to determine the likely nonlinear behaviour of the lacustrine layer in the Grenoble basin, samples from two sites in the Grenoble basin underwent a series of tests to determine their plasticity indices, grain size distributions and mineral contents. The samples consisted of intact core samples taken from between 40 and 48 m during the destructive boring at Crolles (2005), and disturbed samples – “cuttings” – between 100 and 430 m from the destructive deep boring at Montbonnot (1999). The plasticity indices obtained were similar at all depths, with values between 10 and 20%, indicating relatively low plasticity. The grain size distribution tests indicated that the lacustrine material is in fact silt with a trace of clay and variable sand content (maximum 30% sand), depending on the depth. Finally, the mineralogy analyses showed that the amount and type of clay minerals present was almost identical at different sites and different depths in the lacustrine layer. The homogeneity of the lacustrine layer was thus confirmed. Based on the nonlinearity curves for cohesive, fine-grained soils established by Vucetic and Dobry (1991), a hypothetical nonlinearity curve can be obtained for the lacustrine silt as a function of its plasticity index.

Two undrained cyclic triaxial tests were carried out on intact samples from Crolles, after an isotropic consolidation corresponding to the vertical confining pressure in situ. The shear modulus, G, and damping coefficient, D, corresponding to different levels of cyclic shear strain, γ, were obtained from these tests, and the corresponding modulus degradation and damping curves plotted. While more testing is required, and for a wider range of strain levels, it was shown that the experimental curves obtained fit reasonably well with the Vucetic and Dobry curves corresponding to the same plasticity index.

This study thus predicts significant nonlinear behaviour in the lacustrine silt layer, especially at lower depths. If, given the homogeneous nature of the layer, this nonlinearity is extended to the entire thickness of the layer, the risk of dynamic amplification during a seismic event will be minimal. However, the effect of very high confining pressures on the non linear behaviour of the low plasticity lacustrine silt is currently unknown. A programme of cyclic triaxial testing under confining pressures of the order of 4000 kPa is being planned.

Finally, numerical simulations of the seismic response of a typical Grenoble basin vertical profile will be carried out. The nonlinear behaviour of the lacustrine layer will be taken into account in the simulations, using information obtained from this investigation.

5. Acknowledgements

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