INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

Panel discussion: Stress-strain behaviour of undisturbed clays in the laboratory

Débat de spécialistes: Comportement mécanique des argiles non remaniées au laboratoire

D.C.F. Lo Presti – *Department of Structural Engineering, Politecnico di Torino, Italy*

ABSTRACT: This Panel presentation concerns the stiffness-assessment of undisturbed clays in the laboratory and summarises the research activity undertaken at the writer's University over the last five years. The main purpose of this research was to define reliable test procedures in order to determine the stiffness of geomaterials to be used for settlement analysis under working loading conditions.

RESUME: Cette présentation concerne la détermination de la rigidité des argiles naturelles aux moyen d'essais de laboratoire et elle résume les dernières cinq années d'activité de recherce effectuées à l'Université où exerce l'auteur. Cette recerche voulait avant tout definir des techniques de reconsolidation satisfaisantes afin de déterminer de la rigidité des geomateriaux à utiliser pour calculer les déplacement des fondations.

1 INTRODUCTION

This panel presentation deals with three different topics: i) evaluation of sample disturbance by means of different methods, ii) assessment of reconsolidation techniques which minimise the sample disturbance effects, iii) influence of strain-rate on the stiffness of undisturbed clays.

As far as reconsolidation techniques are concerned, two different aspects are considered: i) the relevance of the K_o reconsolidation, ii) a comparison of wet vs. dry setting method.

Wet setting is the conventional method used to set a specimen prior to testing. Dry setting was originally developed at NGI (Berre 1982) and was more recently adopted at the University of Tokyo (Ampadu and Tatsuoka 1993) and the Politecnico di Torino (Lo Presti et al. 1995). The peculiar characteristic of this method is that the specimen is not allowed to adsorb water. The above indicated works describe the dry setting procedures in great details. Those adopted at the writer's Laboratory are briefly summarised in the following.

Dry filter papers and dry porous stones are used during specimen setting. The bottom drainage line is filled with water up to 1 cm below the porous stone. The top drainage line is left empty. After the specimen is set and the pressure cell is sealed, the system saturation is achieved in two steps:

- by flushing the deaired water with a head of 50 cm through the specimen for at least 24 hrs. During this stage the vertical and horizontal stresses are independently increased to prevent any axial and radial strains with a tolerance of $\pm 5\mu m$.
- by back pressurising to dissolve in the water any air bubbles which might still be trapped in the lines and in the space between the specimen and the membrane. This stage is terminated when the B parameter is equal to or larger than 0.95.

The above outlined procedure can be used only in the case of triaxial cells with the following characteristics: i) local gauges for the axial and radial strain measurements, ii) the Lucite cylinder of the pressure cell is external to the tie rods, so that the loading ram is rigidly connected to the top or base caps.

The results shown in this presentation concern two undisturbed Italian clays. Pisa clay samples were retrieved from depths of between 12 and 17 meters at the site of the Leaning Tower by means of a Laval sampler (La Rochelle et al. 1991). This is a soft, lightly overconsolidated ($OCR = 1.5$ to 2.0), quaternary, marine clay with low to medium PI. Detailed information on the Pisa clay deposit is given by Berardi et al. (1991), Costanzo et al. (1994) and Jamiolkowski et al. (1994).

The Augusta site is located on the east coast of Sicily. Augusta clay is a medium stiff, overconsolidated ($OCR = 2.0$ to 6.0), quaternary marine clay with low to medium PI. Samples were retrieved with a Shelby sampler of 86 mm diameter. Detailed information on the Augusta clay deposit is given by Maugeri et al. (1994).

2 EVALUATION OF SAMPLE DISTURBANCE

Three different methodologies have been considered to evaluate sample disturbance.

2.1 Void ratio variation

Table 1 shows the void ratio changes of Pisa clay samples that have been K_o reconsolidated to the in situ geostatic stress. The K_o consolidation was automatically controlled by means of a PC and the specimens underwent the dry setting procedure. After the application of the consolidation stresses, the tests were paused until the creep rate fell to 0.05 %/day, following the recommendations given by Jardine et al. (1991). The first column indicates the void ratio after trimming; e_f is the void ratio after flushing, e_s is that after complete saturation by means of back-pressurisation and e_c is that at the end of consolidation. The global void ratio variation $(e_o - e_c)/e_o = \Delta e / e_o$ is reported in the last column. According to Lunne et al. (1997) when this parameter is less than 0.04. the sample can be considered to be very good or excellent. The results of Table 1 are exceptionally good in comparison to the indications given by Lunne et al. (1997). However it should be considered that the indications given by Lunne et al. (1997) concern clays with PI= 10 to 17 % which is much lower than PI of Pisa and Augusta clays. It should be also considered that, according to Higth (1993), the sample disturbance is more pronounced in low plasticity clays.

Tables 2a and 2b show the void ratio changes of Augusta clay samples that have been K_o reconsolidated to the in situ geostatic stress. In this case both wet and dry setting procedures were followed. The void ratio variation is also very limited for Augusta clay, in the case of dry setting. However, the specimens experience very large swelling strains after saturation, in the case of wet setting. According to Lunne et al. (1997) these samples should be classified as poor. The last column reports two different numbers: the first considers the void ratio variation with respect to e_o, the second with respect to e_s.

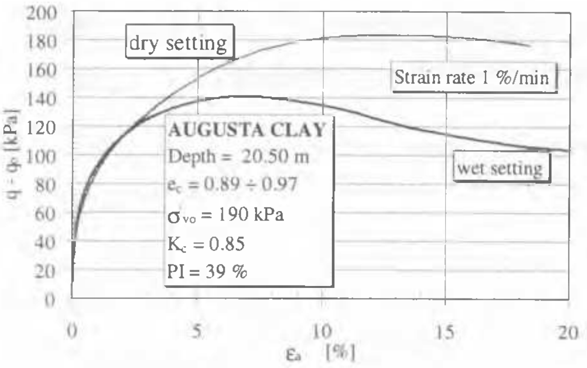

Figure 1 Influence of reconsolidation method on the stress-strain behaviour of Augusta clay.

Table 1 Void ratio of Pisa clay at different stages during reconsolidation (dry setting)

e_0	e_f	e_s	e_c	$\Delta e / e_0$
1.701	1.702	1.715	1.688	0.008
1.731	1.732	1.747	1.726	0.003
1.662	1.663	1.662	1.643	0.011

Table 2a Void ratio of Augusta clay at different stages during reconsolidation (dry setting)

e_0	e_f	e_s	e_c	$\Delta e / e_0$
0.954	0.955	0.954	0.936	0.019
0.896	0.898	0.899	0.884	0.014

Table 2b Void ratio of Augusta clay at different stages during reconsolidation (wet setting)

e_0	e_f	e_s	e_c	$\Delta e / e_0$
0.998		1.071	1.017	-0.02/0.05
1.022		1.093	0.974	0.05/0.11

Table 3a Residual stresses (PISA CLAY)

σ'_v [kPa]	σ'_h [kPa]	σ'_{vc} [kPa]	σ'_{hc} [kPa]	σ'_{ho} [kPa]
65	68	130	92	88
58	63	128	78	88

Table 3b Residual Stresses (AUGUSTA CLAY)

σ'_v [kPa]	σ'_h [kPa]	σ'_{vc} [kPa]	σ'_{hc} [kPa]	σ'_{ho} [kPa]
62	69	200	129	200
90	95	185	157	185

The impact of dry and wet setting on the large strain stiffness of Augusta clay is shown in Fig. 1 by comparing the stress-strain curves from CKoU Compression Loading Triaxial tests performed under similar conditions. The differences, at very small strains, cannot be appreciated in this Figure and will be discussed later.

2.2 Residual stresses

It is believed that the pressure required to prevent swelling represents the residual stress in a specimen. The first two columns of Tables 3a and 3b show the individual stress components (σ'_v,σ'_h) which were necessary to maintain the axial and radial strains equal to zero (with a tolerance of ±5μm) during the aturation process for the Pisa and Augusta clays respectively.

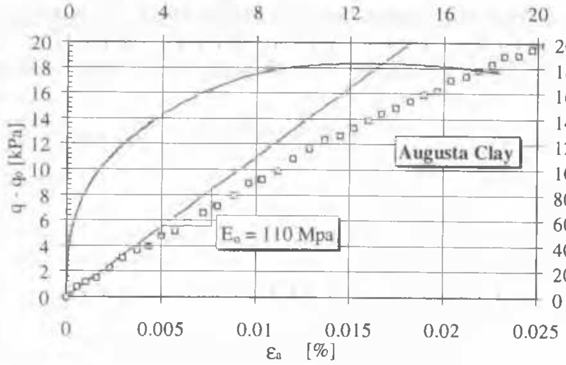

Figure 2 Stress-strain curve of Augusta clay

Table 4 Small strain stiffness of Augusta and Pisa clays

σ'_{vc} [kPa]	σ'_{hc} [kPa]	e_c [-]	G_0 [MPa]	E_0 [MPa]	**Soil Setting**
200	130	0.914	43	100	Augusta dry
185	156	0.884	43	110	Augusta dry
221	135	1.071	36	80	Augusta wet
187	158	0.974	30	75	Augusta wet
128	78	1.688	35	70	Pisa dry
128	100	1.726	28	80	Pisa dry
130	92	1.643	28	90	Pisa dry

The stresses at the end of the consolidation are also indicated together with the best estimate of the in situ horizontal stress (σ'_{ho}) in the last column. This estimate was inferred from other laboratory tests (K_0 odometer tests) in the case of Pisa clay (Berardi et al. 1991) and from in situ Dilatometer tests in the case of Augusta clay (Maugeri et al. 1994). The better quality of the Laval samples is clearly seen by comparing the residual stresses observed in the laboratory to σ'_{ho}. It is also worthwhile to stress that the PC-controlled K_0-consolidation gave K_0 values in the laboratory that were close to those estimated in situ only for the Pisa clay samples, whilst in the case of Augusta clay the K_0 was underestimated in the laboratory.

2.3 In situ vs. laboratory small strain shear modulus

The values of the small strain shear modulus (Go) and the small strain Young's Modulus (Eo) are shown in the case of Augusta clay in Table 4. Go was determined during the Triaxial test by means of seismic tests which were performed using a pair of bender elements. The reported value of Go is that which was determined at the end of consolidation. Eo was determined from the initial slope of the stress-strain curve at very small strains (Fig. 2). It is possible to see that both the Go and Eo values obtained from tests which underwent the wet setting procedures are smaller in comparison to those obtained in the case of dry setting conditions. The reference value of Go from in situ seismic tests is of about 80 Mpa, for the considered specimens. It is possible to see that Go is largely underestimated in the laboratory in both cases. The observed large discrepancy is probably due to the fact that the horizontal consolidation stress applied to the specimen in the laboratory is much lower than that which exists

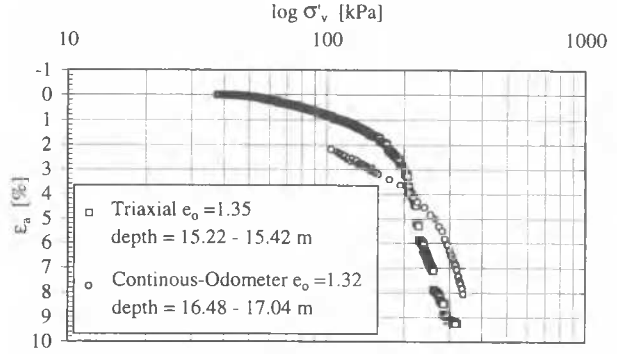

Figure 3 Comparison of compression curves of Pisa clay ($\varepsilon_h = 0$)

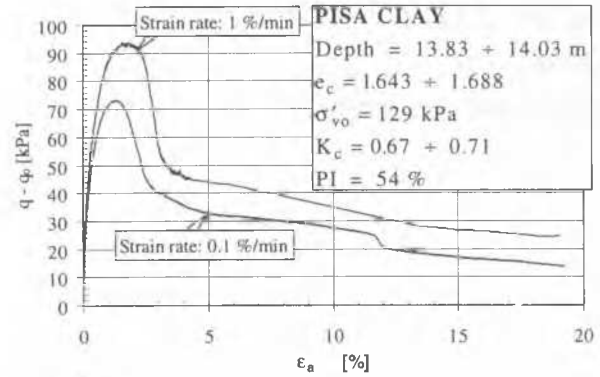

Figure 4 Stress-strain curves of K_o consolidated Pisa clay specimens

in situ. This last consideration is indirectly confirmed by the not so bad agreement between the reference Go in situ (40 MPa) and the small strain stiffness determined in the laboratory for the Pisa clay samples (Table 4). Moreover, in the case of Augusta clay it was found (Lo Presti et al 1998) that $G_0(\text{lab})/G_0(\text{field}) \cong 0.86 \pm 0.1$ when the specimens are reconsolidated in the laboratory with $K_c = 1$.

3. K_o RECONSOLIDATION

Specimens can be easily reconsolidated under K_o conditions by means of computer controlled triaxial apparatuses. Fig. 3 compares the compression curves of Pisa clay obtained from a continuos odometer test at constant pore pressure gradient and that obtained from a triaxial test performed at nil radial strain. In particular, the latter curve allows a better identification of the preconsolidation pressure and exhibits a highly non linear behaviour and a clear collapse of the structure when the preconsolidation pressure is exceeded. This is probably due to the absence of friction along the lateral surface of the specimen that had been subjected to K_o compression in the triaxial apparatus.

The impact of K_o consolidation on the stress-strain relationship of Pisa clay is clearly shown in Fig. 4. It is possible to see that, under K_o conditions, even a soft clay, like Pisa clay, exhibits brittle behaviour.

The results of Figs. 3 and 4 clearly show the importance of automatic reconsolidation procedures in the laboratory. Moreover, the unsuccess of these procedures for the Shelby tube samples of Augusta clay (Table 3b) indicates the importance of using only high quality samples.

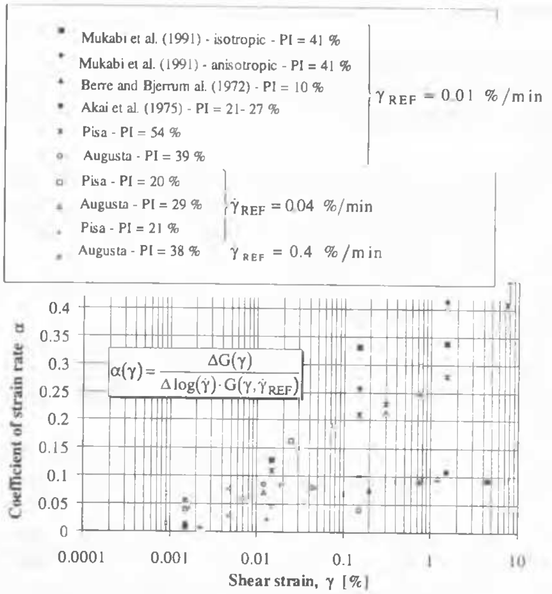

Figure 5 Coefficient of strain rate

4 RATE EFFECT

Data of Fig. 4 also provide information about the influence of the strain rate on the stiffness of Pisa clay at large strains. The rate effect at small strains cannot be seen in this Figure. However, at very small strains the rate effect on the stiffness of clays is less important. The increasing influence of strain rate on soil stiffness for increasing strain level can be appreciated by computing the so called coefficient of strain rate $\alpha(\gamma)$. This coefficient is defined, for a given strain level, as the increase in stiffness for one log cycle of strain rate normalised with respect to the stiffness obtained for that strain level and for a reference strain rate. The coefficient of strain rate expresses the rate effect on the soil stiffness as a function of the strain level.

Fig. 5 summarises the values of the coefficient of strain rate obtained at the speaker's laboratory and those inferred from other published data. It is possible to see that the coefficient increases with the plasticity index and, for a given soil, with the shear strain level.

CONCLUSIONS

The void ratio variations due to reconsolidation processes are good indicators of sample disturbance. Residual stresses and the ratio $G_0(\text{lab})/G_0(\text{field})$ are also good indicators. However a judgement on sample quality can be very strongly affected by the reconsolidation procedures followed in the laboratory.

Dry setting is very important, especially when reconsolidating overconsolidated clays with high swelling potential. Swelling strains developed during wet setting can drastically reduce the quality of a sample.

K_o reconsolidation is also extremely important when determining the stress-strain behaviour of geomaterials and can be easily achieved by means of computer controlled triaxial apparatuses. However, the use of automatic procedures is successfull only in the case of high quality samples.

The influence of strain rate on clay stiffness is relevant especially at larger strains. The coefficient of strain rate, which expresses such a dependence, increases with PI and, for a given soil, with the shear strain.

REFERENCES

Ampadu, S.K. and Tatsuoka, F. 1993. Effect of Setting Method on the Behaviour of Clays in Triaxial Compressiom from Saturation to Undrained Shear *Soils and Foundations:* 33(2): 14-34.

Berardi, G., Caroti, L., Giunta, G., Jamiolkowski, M. and Lancellotta, R. 1991. Mechanical Properties of Upper Pisa Clay. *Proceeding of the X ECSMFE Firenze 1991, Associazione Geotecnica Italiana:* 1: 7-10

Berre T. 1982. Triaxial Testing at the Norvegian Geotechnical Institute. *Geotechnical Testing Journal,* 5(1/2): 3-17.

Costanzo D., Jamiolkowski M., Lancellotta R. and Pepe M.C. 1994. Leaning Tower of Pisa - Description of the Behaviour. *Invited Lecture - Proceedings Settlement '94,* Texas A&M University

Higth D.W., 1993. A Review of Sampling Effects in Clays and Sands. *Offshore Site Investigation and Foundation Behaviour,* 28: 115-146.

Jamiolkowski M., Lancellotta R. and Lo Presti D.C.F. 1994. Remarks on the Stiffness at Small Strains of Six Italian Clays. *Theme lecture Session la, Proceedings IS Hokkaido,* 2: 817-836.

Jardine, R. J., D.M. Potts, St. John, H. D. and Hight, D. W. 1991. Some Practical Applications of a Non Linear Ground Model. Proceedings of the X ECSMFE, *Firenze 1991, Associazione Geotecnica Italiana, 1: 223-228.*

La Rochelle, P., Sarrailh, J., Tavenas, F. and Leroueil, S. 1981, Causes of Sampling Disturbance and Design of a New Sampler for Sensitive Soils. *Canadian Geotechnical Journal,* 18: 52-66.

Lo Presti D.C.F., Pallara O. and Puci I. 1995. A Modified Commercial Triaxial Testing System for Small Strain Measurements: Preliminary Results on Pisa Clay *Geotechnical Testing Journal,* 18(1): 15-31.

Lo Presti D.C.F., Maugeri M., Pallara O. and Cavallaro A. 1998 Shear modulus and damping of a stiff marine clay from in-situ and laboratory tests. *ISC 98,* Atlanta, Georgia, submitted.

Lunne T., Berre T. and Strandvik S. 1997. Sample Disturbance in soft low plastic Norvengian clay. *Recent Developments in Soil and Pavement Mechanics,* Almeida (ed.) Balkema Rotterdam, 81-102.

Maugeri M., Castelli F. and Motta E. 1994. Pile Foundation Performance of an Earthquake Damaged Building. *Proc. of the Italian-French Symposium on Strengthening and Repair of Structures in Seismic Areas,* Nice, France