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Papers

Well known aspects of soil behaviour so often neglected

S. Leroueil
Université Laval, Ste-Foy, Québec, Canada

Abstract: Beyond classical soil mechanics in which the behaviour of soils depends only on their stress history and density, and is considered either elastic or plastic, there are peculiar aspects that may have an important influence on their behaviour both in laboratory and in the field. Mostly in relation with soft clays, the influence of five of these aspects is considered in this paper: (a) non-linear stress-strain behaviour; (b) anisotropy; (c) microstructure; (d) fabric; and (e) viscosity. In each case, after generalities, some practical implications are examined.

INTRODUCTION

Fortunately, we are generally successful in designing earthworks and structures in soils and failures seldom occur. We are in particular successful with bearing capacity of foundations and stability of embankments and cuts for which our practice is based on a long experience and use a factor of safety. It is more difficult when we talk about deformations, especially when it concerns special projects such as underground constructions or offshore foundations for which we have less experience, or when details of soil behaviour are examined. This is mostly due to the fact that soil behaviour is complex and influenced by many factors. Hight & Higgins (1994) reviewed the factors that are influencing stress-strain behaviour of geomaterials. For a given soil, the most important seem to be: degree of disturbance, in particular sampling disturbance; current stress conditions, in particular OCR; q/p'; recent (small strain) stress history and aging; stress path, including stress axis rotation; strain rate and temperature. The aim of this paper is to examine some of these factors that are thought to be important for solving practical problems, but are often neglected in practice.

INFLUENCE OF NON-LINEAR STRESS-STRAIN BEHAVIOUR

Generalities

It has been recognized for long time that stress-strain behaviour of soils is non-linear, but it is only recently, with the development of devices allowing the accurate measurement of local strains (e.g. Jardine et al., 1984), that this behaviour started to be understood. Jardine et al. (1991), Jardine (1992) schematized this behaviour as shown in Fig. 1, it can be described as follows:

- There is the outer yield curve Y_3, associated with a change in fabric coincident with or slightly inside what this author often calls limit state curve. Soil experiencing stress paths that reach this curve undergo large plastic strains. Inside Y_3, strains are small to moderate.
- Within the inner sub-yield curve Y_1 surrounding the current effective stress condition (zone 1), soil behaviour is linear-elastic. This zone is characterized by the small strain shear modulus G_o. According to Jardine (1994), the strain at Y_1, ε_{Y1}, is in the order of 0.001 % for Bothkennar clay, but has a tendency to increase with plasticity index, to be in the order of 0.1% for the highly plastic Mexico City clay.

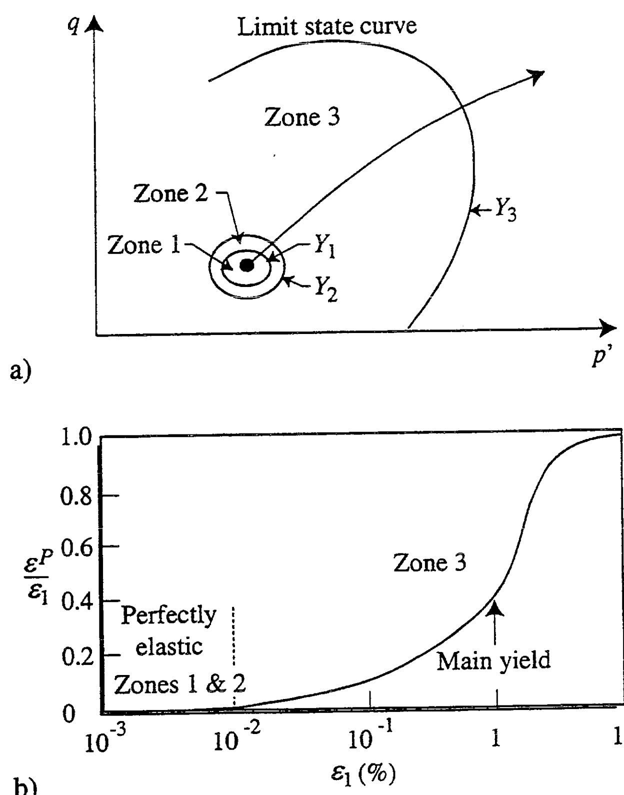


Fig. 1 - Scheme of multiple yield surfaces (a) and increase in $\varepsilon^p/\varepsilon_1$ ratio with strain (b) (modified after Jardine *et al.* (1991) and Jardine (1992))

- When a stress path crosses Y_1, but remains inside Y_2 (zone 2), the behaviour is non-linear elastic.
- Between Y_2 and Y_3 (zone 3), soil develops plastic strains. As indicated in Fig. 1b, the ratio of plastic over total strain progressively increases as the stress path

approaches Y_3. In this zone, shear modulus decreases as the stress path approaches the limit state curve.

- When Y_1 and Y_2 are crossed by the effective stress path, they are dragged with the effective stress point. However, an elastic behaviour is obtained only if there is a sharp change in stress path direction.

As an example, Fig. 2 shows the curve Y_3 as well as the curves Y_1 and Y_2 around in situ stresses for the Bothkennar clay. Fig. 3 shows a typical normalized shear stiffness-strain curve deduced from a triaxial test performed on a sample taken at a depth of 6 m.

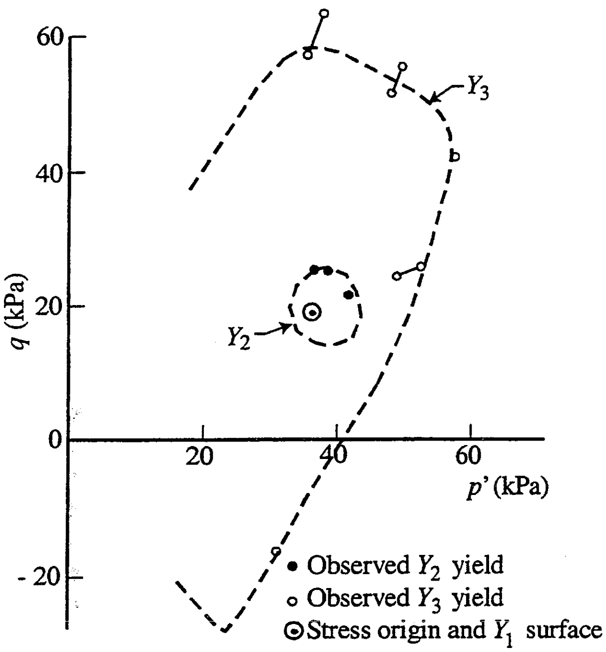


Fig. 2 - Yield surfaces identified for Bothkennar clay under in situ stresses (after Smith et al., 1992)

As indicated by the schematic Fig. 4 from Hight & Higgins (1994), the curves Y_1, Y_2 and Y_3 decrease in size as the principal stress axis rotates from the vertical direction to the horizontal one (α changing from 0 to 90° in Figure 4). Also, as indicated by this figure, the elastic zone could vanish for large values of the angle of stress axis rotation.

Practical implications

Consequences of the non-linear stress-strain behaviour of soils has been shown by several authors. We can refer to Jardine et al. (1995) who loaded a pad foundation to failure at Bothkennar. The pad was 2.2 m square, giving an equivalent diameter D of 2.48 m. Fig. 5 shows the ground surface settlement δ_r developed at a distance r from the centre-line, normalized with respect to the mean pad settlement δ_c, versus r/D. The full lines show the settlements for load factors L_f, corresponding to the applied pressure divided by the ultimate capacity, of 0.15, 0.5 and 1.0. It can be seen that the non-linear clay model, LPC2, simulates relatively well the observed behaviour; in particular, surface settlements diminish with radial distance far more rapidly than is expected from linear elastic calculations.

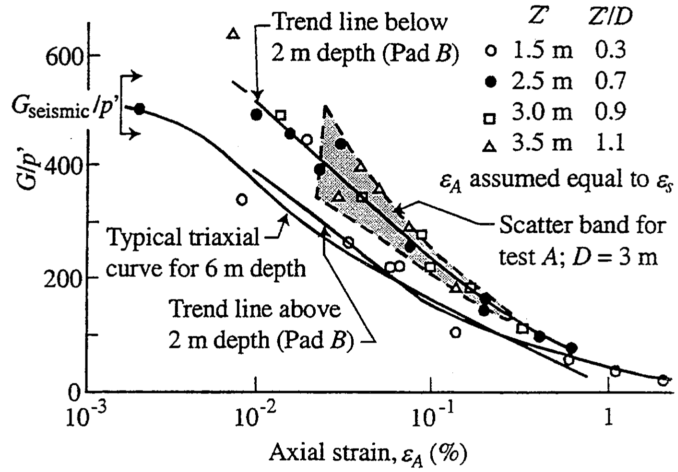


Fig. 3.- Normalized shear stiffness-strain plots derived from centre-line stress-strain plots from pad loading tests at Bothkennar (modified after Jardine *et al.*, 1995)

Fig. 3 presents some normalized shear stiffness-strain plots derived from centre-line stress-strain plots from the pad loading tests. It can be seen that the trend is very similar to the results deduced from laboratory tests and confirms the non-linear behaviour of Bothkennar clay.

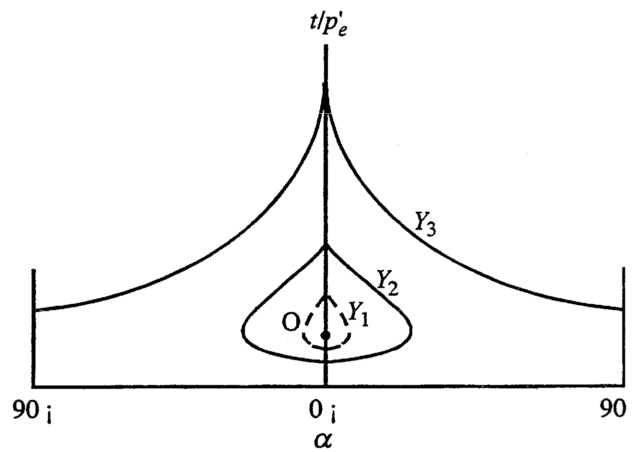


Fig. 4 - Framework for soil behaviour - Influence of the angle of stress axis rotation on the yield curves (After Hight and Higgins, 1994)

Hight & Higgins (1994) and Ochi et al. (1994) provide other evidences of the non-linear stress-strain behaviour of geomaterials other than soft clays in field conditions.

The practical use of small strain characteristics, and in particular of G_o, in soft clays is often questioned. The practical applications are however important: G_o is the starting point for the stress-strain curve that controls soil movements around foundations, excavations, tunnels, etc.; the small strain compressibility also controls, with hydraulic conductivity through the coefficient of consolidation/swelling, the rate at which consolidation/swelling processes occur in clay deposits when overconsolidated.

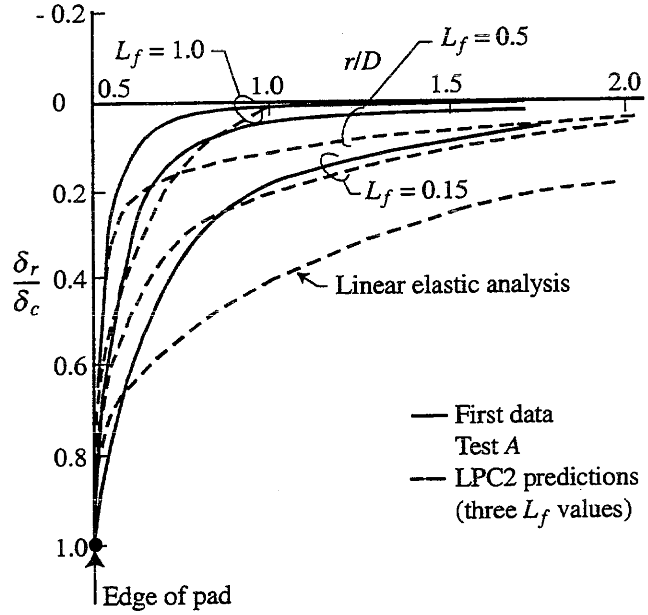


Fig. 5 - Settlements developed close to a rigid pad loaded on Bothkennar clay (after Jardine *et al.*, 1994)

This latter aspect is presently examined at Laval University and preliminary results are presented hereunder.

The Saint-Hilaire excavation is 8 m deep and is a square with four slopes of 18°, 27°, 34° and 45° to the horizontal. Pore pressure observations are reported by Lafleur et al. (1988). Laflamme and Leroueil (1999) simulated the pore pressure variations in the 18° and 27° slopes that have not been affected by failures. Fig. 6 shows a section of the excavation with the location of the piezometers and Fig. 7 presents the pore pressures measured during excavation and after, till full pore pressure equilibration, in three of these piezometers. For the simulation, a linear elastic model was used. First considered were hydraulic conductivity measured in laboratory and in situ with a self-boring permeameter and small strain shear modulus G_o measured in cross-hole tests or deduced from piezocone tests at large depths. However, to get the good fit shown in Fig. 7, the resulting coefficient of consolidation/swelling (c_{vs}) had to be multiplied by a factor of 2.3 in this case.

Tavenas & Leroueil (1980) evidenced the fact that there is significant consolidation of clay foundations during the early stages of construction of embankments, whereas the soil is still overconsolidated. This observation was reflecting a coefficient of consolidation c_{vs} higher than generally thought. Laflamme and Leroueil (2001) examined the construction pore pressures under the Saint-Alban test fills (Leroueil et al., 1978). The three sections B, C and D had the final geometry shown in Fig. 8 with a height of 3.4 m. Fig. 9 shows the $\Delta u/\Delta\gamma H$ profiles observed under the centre-line of the fills for an embankment load of 28 kPa (Leroueil et al., 1978).

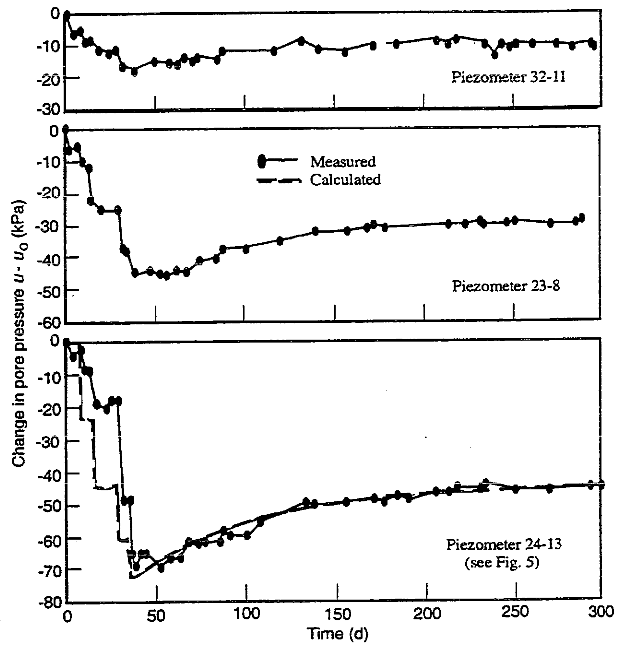


Fig. 7 - Measured and calculated changes in pore pressure with time at Saint-Hilaire (after Lafleur *et al.*, 1988; Laflamme & Leroueil, 1999)

For the simulation of the construction, Laflamme and Leroueil (2001) considered a linear elastic model, hydraulic conductivity profiles obtained from laboratory and in situ measurements, and small strain shear modulus obtained from cross-hole and SASW tests. However, to get the best fit shown with dashed lines in Fig 9, the resulting coefficient of consolidation/swelling had to be divided by a factor of about 1 to 1.5. It is worth noting that the results, not shown here, indicate that the clay stiffness would be higher for loads smaller than 28 kPa and smaller for loads larger than 28 kPa, which supports the fact that stiffness of soils decreases as strain increases.

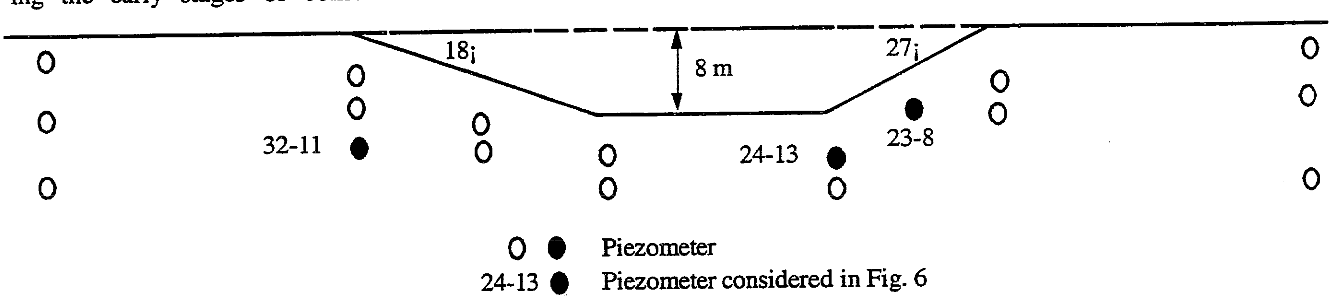


Fig. 6 - Test excavation at Saint-Hilaire (after Lafleur *et al.*, 1988)

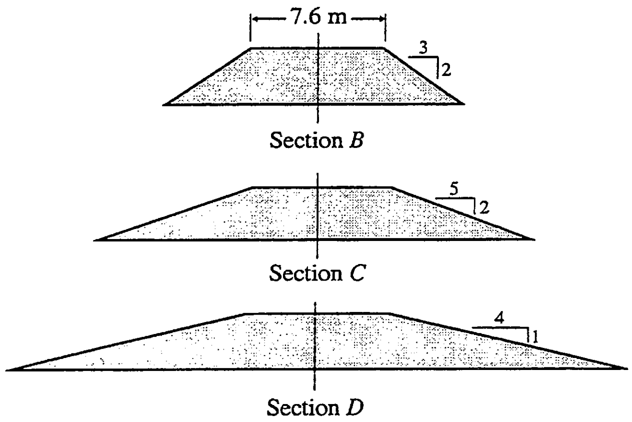


Fig. 8 - Saint-Alban test embankments (After *Tavenas et al.*, 1974)

The study presently performed by Laflamme and Leroueil involves two sites in addition to Saint-Hilaire and Saint-Alban. The results are preliminary but clearly indicate that: (a) the stiffness to consider in these small strain consolidation or swelling processes is close to the one given by measured G_o; and (b) field observations generally confirm the decrease in stiffness as strains increase.

INFLUENCE OF ANISOTROPY

Generalities

Natural clays were deposited under one-dimensional conditions, i.e. under an effective stress ratio $K_{o\ n.c.} = \sigma'_h/\sigma'_v$ approximately equal to $(1 - \sin\phi'_{n.c.})$, where $\phi'_{n.c.}$ is the friction angle of the normally consolidated soil. Even if it has not been evidenced for soft clays, this stress anisotropy probably results in an anisotropic distribution of contacts between particles or aggregates. What has been demonstrated is that it significantly influences the mechanical behaviour of clays. This is particularly true for the limit state curve (Y_3 in Figs. 1 and 2). Fig. 10 shows the limit state curve for the sensitive clay from Louiseville, Quebec. The influence of anisotropy is evident. Compiling such limit state curves obtained on natural clays from ten different countries, Diaz-Rodriguez et al. (1992) found the results summarized in Fig. 11. Fig. 11a shows extreme cases obtained on Winnipeg clay ($\phi'_{n.c.} = 17.5°$) and Mexico City clay ($\phi'_{n.c.} = 43°$). The shape is more or less elliptic and more or less centred on the $K_{o\ n.c.}$ ($= 1 - \sin\phi'_{n.c.}$) line in the $(\sigma'_1 - \sigma'_3)/2$ vs $(\sigma'_1 + \sigma'_3)/2$ diagram. As shown in Fig. 11b, the geometrical characteristics of the limit state curves $(\sigma'_1 - \sigma'_3)_{max}/2\ \sigma'_p$ and σ'_{pi}/σ'_p respectvely increases and decreases when $\phi'_{n.c.}$ increases. This indicates that the shape of the limit state curves of natural clays is essentially controlled by the friction angle of the normally consolidated soil. Studies performed in the last decade on other soft clays confirm the results presented in Fig. 11b.

Independently from anisotropy, the Mohr-Coulomb failure criterion, as well as those suggested by Matsuoka &

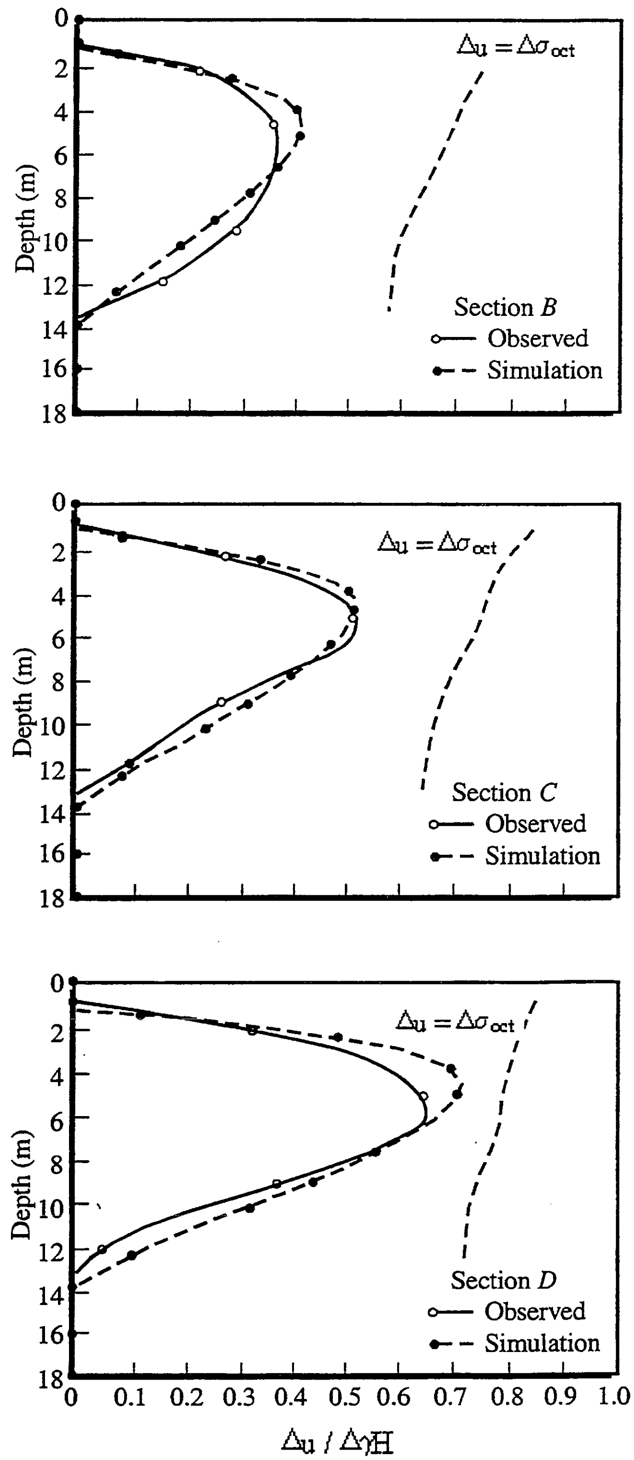


Fig. 9 - Observed and predicted pore pressures under the centre line of test fills B, C, D for an embankment load of 28 kPa at Saint-Alban (from Laflamme and Leroueil, 2001)

Nakai (1974) and Lade & Duncan (1975), imply for a given mean effective stress p' value a shear strength in triaxial compression (axial stress higher than the radial ones) larger than the shear strength observed in triaxial extension (axial stress smaller than the radial ones). This can be seen on Fig. 10 where the two dashed lines represent the strength envelopes in compression (positive q values) and extension (negative q values) according to Mohr-Coulomb and a friction angle of 32°.

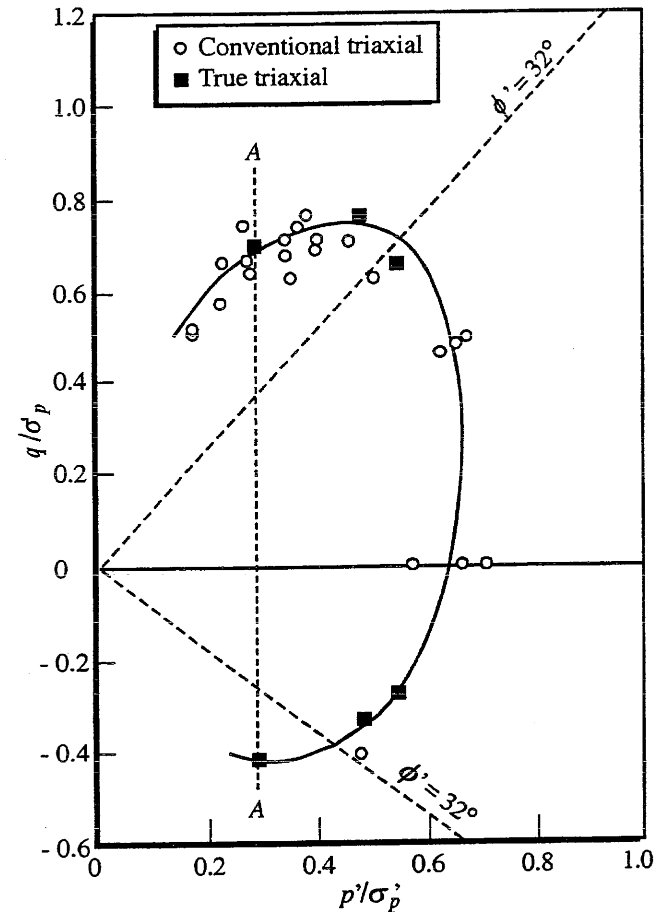


Fig. 10 - Limit state curve of Louiseville clay normalized with respect to the preconsolidation pressure

The behaviour previously described stands for axisymmetric stress conditions with radial stress generally applied along the depositional plane. However, most practical problems do not imply axisymmetric stress conditions but three principal stresses that can be different and rotated relatively to the vertical and horizontal directions. This 3-D behaviour can be examined in laboratory in two complementary ways: study of the effect of the third principal stress, which can be seen through the effect of $b = (\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3)$; and the study of the effect of stress axis rotation characterized by α, the inclination of the major principal stress relative to vertical. The effect of b is often examined with true triaxial apparatus (TTA); the effect of α can be examined with torsion shear hollow cylinder apparatus.

Several authors used TTA to examine yielding and failure of soft clays in three-dimensional stress conditions, both in normally consolidated conditions (e.g. Kirkgard & Lade, 1993) and in the overconsolidated domain (Boudali, 1995; Callisto & Calabresi, 1998). Callisto & Calabresi (1998) tested Pisa clay after reconsolidation to in situ stresses. Their results are shown in Fig. 12. It can be seen that the failure envelope of the intact clay is rounded compared to Mohr-Coulomb criterion, which is in agreement with the failure criteria proposed by Matsuoka & Nakai (1974) and Lade & Duncan (1975). Boudali (1995) performed a similar study on the highly microstructured Louiseville clay. He, in particular, performed a series of tests at p' = cst. = 0.29 σ'_p (see Fig. 10). The results are shown in Fig. 13. It can be seen that: (a) due to microstructure, the failure surface is well above the Mohr-Coulomb failure envelope defined on the basis of the friction angle of the normally consolidated soil; and (b) in comparison with the Mohr-Coulomb criterion, the results show a strong anisotropy. Similar results were obtained on Ottawa clay (Wong and Mitchell, 1975).

In all cases (Figs. 10, 12 and 13), the failure envelopes differ from the Mohr-Coulomb criterion but always show, at the same p' value, a compression strength, S_{uc}, larger than the extension one, S_{ue}. The strength ratio is increased

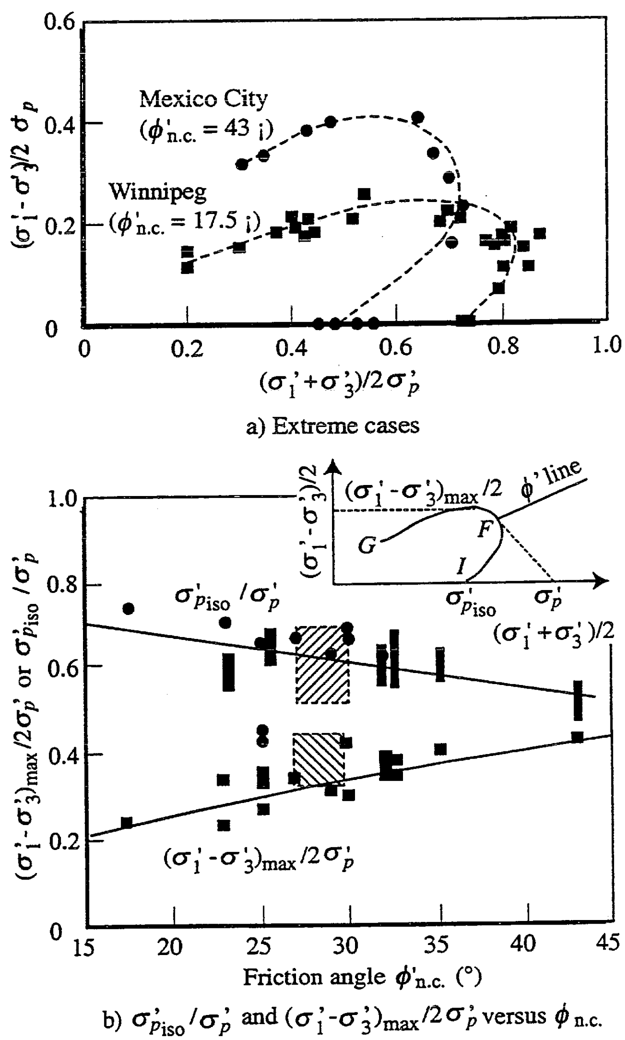


Fig. 11 - Summary on the limit state curves of natural clays (after Diaz-Rodriguez *et al.*, 1992)

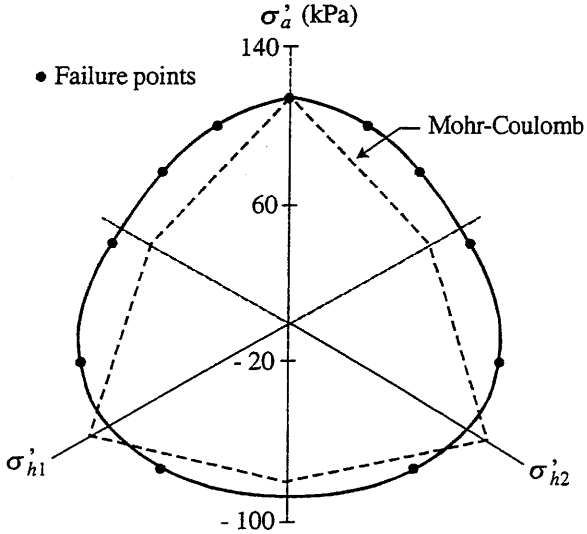


Fig. 12 - Failure envelope determined from tests in the TTA, Pisa clay (after Callisto and Calabresi, 1998)

by the anisotropy of natural clays, the resulting shape of their limit state curve, and the fact that p' at failure in compression is larger than p' at failure in extension. This is confirmed by empirical correlation, as shown in Fig. 14 from Kulhawy and Mayne (1990). The ratio in undrained shear strengths measured in extension and compression typically increases from 0.3 to 0.8 when the plasticity index increases from 0 to 60.

The behaviour previously described is also schematized on Fig 15. The undrained shear strength obtained in compression, S_{uc} at point B, is much larger than the undrained shear strength obtained in extension, S_{ue} at point D. It may be important to mention that, too often, numerical models based on constitutive equations that are not representative of the behaviour of natural clays are used in design. This is in particular the case for Modified Cam clay (MCC) models. As indicated in Fig. 15, MCC models correspond to a limit state curve centred on the p' axis that is very different from those of natural clays. It also indicates an undrained shear strength in extension, at point D', much larger than the real one, at point D.

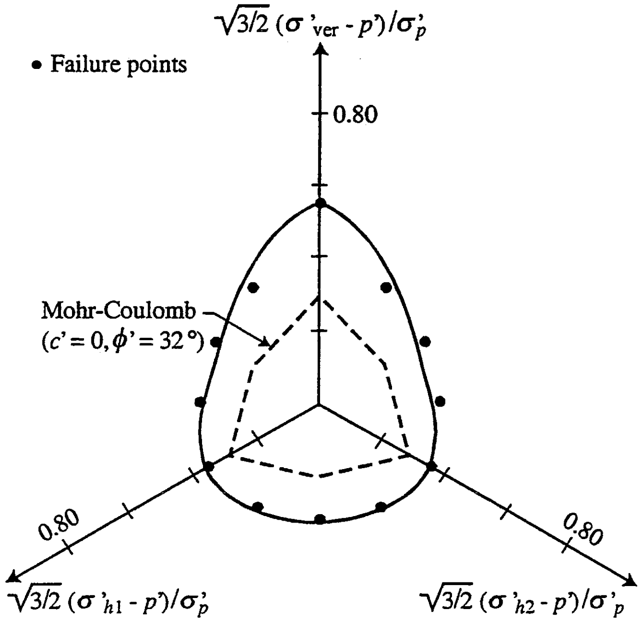


Fig. 13 - Failure envelope determined from tests in the TTA at $p'/\sigma'_p = 0.29$ (Section AA in Fig. 9), Louiseville clay (after Boudali, 1995)

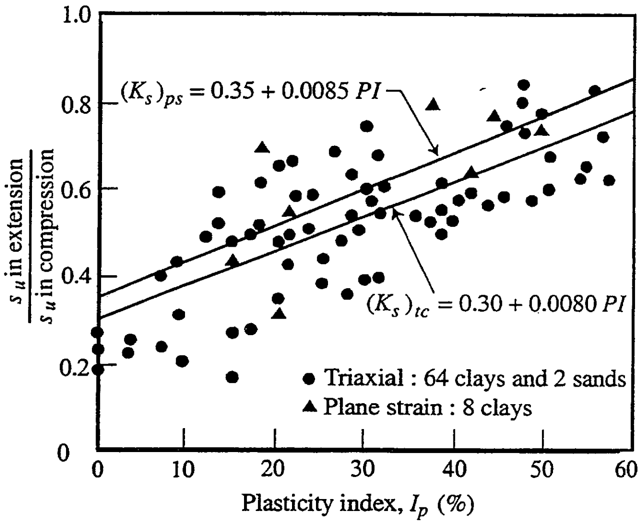


Fig. 14 - Undrained strength ratios in extension and com-Pression versus plasticity index (modified after Kulhawy and Mayne, 1990)

It may be finally mentioned that anisotropy of natural clays is also reflected at small strains. Jamiolkowski et al. (1994) found G_{hh}/G_{vh} ratios under isotropic stress conditions of 1.4 and about 1.55 for the natural Pisa & Panigalia clays respectively. Also under isotropic stress conditions, Nash et al. (1999) found ratios of 1.5-1.9 over the range of void ratio tested for the one-dimensionally reconstituted Gault clay.

Practical implications

Even if strength anisotropy is recognized in some methods of stability analysis (Bjerrum, 1973; Ladd & Foott, 1974), it is not always taken into account, in particular in numerical modelling. The consequences of the differences in models can, however, be extremely important. An example is provided by Whittle & Hashash (1994) who compared results obtained with the MIT-03 model, that fits relatively well the behaviour of natural soft clays (Whittle & Kavvadas, 1994), and MCC model. For an excavation in Boston Blue clay, the MIT-03 model predicts failure when the excavation reaches a depth of about 22.5 m whereas MCC model indicates that a 40 m deep excavation is feasible.

An implication of the shape of the limit state curve of natural soft clays is described by Mc Rostie et al. (1972). They report on a tied-back sheet pile system in Champlain Sea clay that showed important vertical movements and horizontal movements away from the excavation. Laboratory tests and detailed analysis indicated that yielding was due to increase in horizontal stresses to values in excess of their initial value before excavation, but much smaller than the preconsolidation pressure of the clay. It was then attributed to the shape of the limit state curve of the clay.

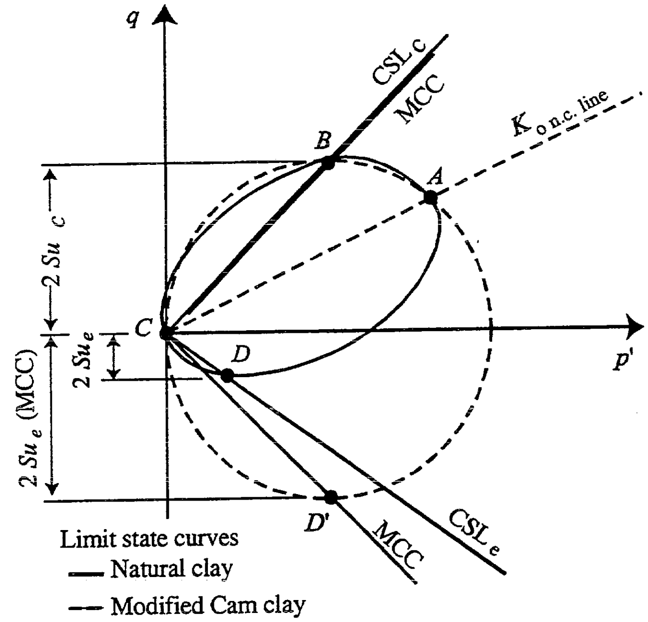


Fig. 15 - Limit state curves for natural clay and as defined by Modified Cam Clay (MCC) model

INFLUENCE OF MICROSTRUCTURE

Generalities

If 20 years ago, microstructured clays, with bonds between particles or aggregates, were considered exceptional, it has since been demonstrated that the opposite stands (Burland, 1990; Leroueil & Vaughan, 1990). Most natural clays and, probably, other soils are microstructured and, in comparison with the same soil non-microstructured or reconstituted, the intact soil has a larger strength and a higher stiffness in the pre-yield range. If bonds are broken, there is destructuration. The effects of destructuration have been observed by many researchers, in particular by Leroueil et al. (1979). They can be summarized as follows: (a) a decrease in stiffness of the soil inside the limit state curve (Y_3 in Figs. 1, 2 and 10); (b) a decrease of the peak shear strength and of the preconsolidation pressure, as well as a shrinkage of the entire limit state curve; and (c) a decrease of the compression index.

Microstructure also influences the small strain shear modulus. For reconstituted and thus non-microstructured clays, Viggiani & Atkinson (1995) showed that G_o could be defined as a function of the applied effective stress and the overconsolidation ratio. From the same set of data, Leroueil (1998) showed that the equation proposed by Viggiani & Atkinson (1995) could also be written as follows:

$$G_o = G_o\,(\text{at } \sigma'_{Y3})\ OCR^{-0.5} \qquad (1)$$

Where σ'_{Y3} is the effective stress on the Y_3 yield curve (Fig. 1), corresponding to σ'_p in one-dimensional compression.

Rampello et al. (1994) demonstrated that G_o could equally be expressed as a function of the applied effective stress and the current void ratio. Moreover, Jamiolkowski et al. (1994) showed that the influence of OCR is negligible when the void ratio is taken into account. In fact, applied stress, void ratio and OCR being interrelated, G_o can be described as a function of 2 of these 3 parameters. Figure 16 shows G_o/G_o (at σ'_p) vs $(\sigma'_v/\sigma'_p)^{0.5}$ for non-microstructured geomaterials with the dashed line. Experimental results (see Leroueil, 1998, for references) show that, for given void ratio and applied stresses, the small strain shear modulus is larger for the microstructured geomaterial than for the same material, non-microstructured. Leroueil (1998) summarized this behaviour with Fig. 16. Shibuya (2000) came to similar conclusions.

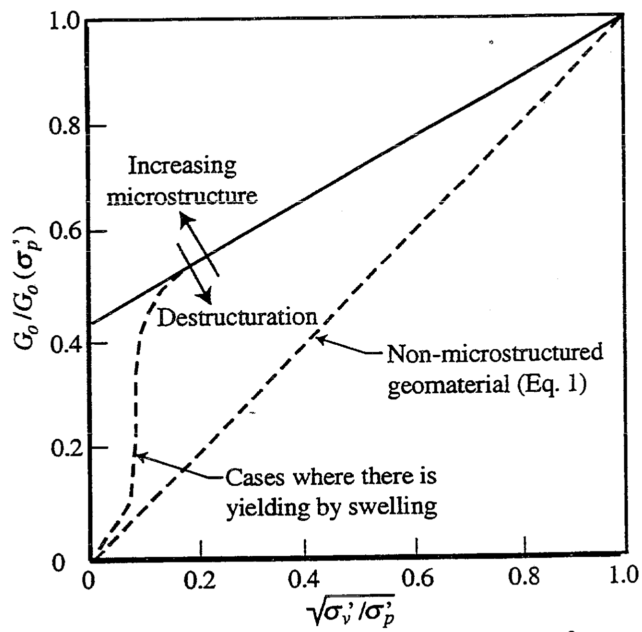


Fig. 16 - Schematic figure showing the influence of Microstructure on *Go* (after Leroueil, 1998)

A consequence of the influence of microstructure on G_o is that destructuration tends to lower the normalized G_o curve towards the non-microstructured one (see Fig. 16). Lohani et al. (1999) came to a similar model for describing the effect of sampling disturbance on G_o.

To obtain realistic soil parameters from laboratory tests, the microstructure of the soil existing in in situ conditions has thus to be preserved at all stages of soil handling: sampling, extrusion, storage, trimming and early stages of testing. Preserving microstructure means avoiding strains that could break inter-particle or inter-aggregate bonds. According to the work made on the behaviour of clayey soils at small strains, irrecoverable strains start developing on the Y_2 surface, Fig. 1). However, major destructuration is induced when a stress path reaches the limit state curve or Y_3, either by compression, shearing or swelling (Leroueil and Vaughan, 1990).

Sampling disturbance

One of the important factors controlling quality of soil specimens is sampling. The causes of sampling disturbance can be described as follows (Tavenas & Leroueil, 1987): (a) disturbance of the soil prior to sampling as a result of poor boring operations or of direct pushing of a piston sampler; (b) mechanical distorsion during the penetration of the sampling tube into the soil; (c) mechanical distorsion and suction effects during the retrieval of the sampling tube; and (d) release of total and effective in situ stresses.

Let us consider the last point in conditions of perfect sampling. In natural soft soils, the in situ effective stress condition is within the limit state curve, at a point such as A (Fig. 17). In ideal soil sampling conditions, a release of total in situ stresses takes place in undrained conditions, resulting in the elimination of shear stresses. The corresponding stress path is such as A-A_1, at essentially constant mean effective stress. If the stress path remains within the limit state surface and the strains remain essentially elastic and small, there is no significant disturbance. However, in the case of lightly overconsolidated clays (Point B on Fig. 17), the stress path associated with perfect sampling would be such as B-B_1 and, due to the shape of the limit state curve of natural clays (Diaz-Rodriguez et al., 1992), might touch the limit state curve before reaching isotropic conditions. In such a case, the clay could be destructured. This indicates that it could be impossible to get undisturbed nearly normally consolidated clays, say for OCRs < 1.15-1.25.

To study the effect of mechanical disturbance, several samplers (Shelby, NGI54, ELE100, Japanese, Laval, Sherbrooke (see Tanaka, 2000, for details on their characteristics)) have been recently compared on different clay sites

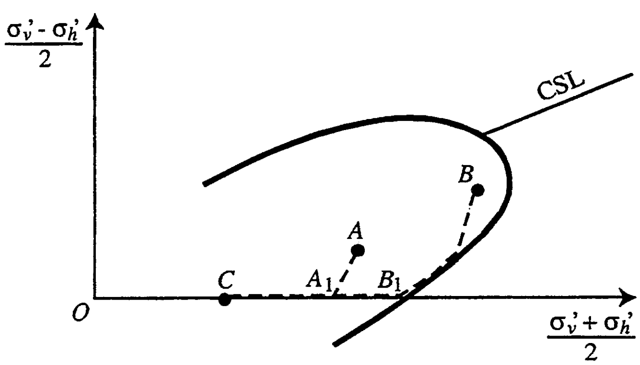


Fig. 17 - Perfect sampling (after Leroueil, 1977))

from Japan, Norway, Quebec and the United Kingdom (Lacasse et al., 1985; Hight et al., 1992; Lunne & Lacasse, 1997; Tanaka et al., 1998; Tanaka, 2000). Also, numerical and analytical works have been performed, in particular by Baligh (1985) and Clayton et al. (1998).

Baligh (1985) showed that, during penetration of the sampling tube into the soil, the soil to be sampled is subjected to an initial compression phase ahead of the sampler, an extension phase in the vicinity of the cutting edge, and a second compression phase inside the tube. Clayton et al. (1998) examined the influence of details of design features of the sampler on strains generated during sampling. Their main findings are summarized in Fig. 18 and hereunder: (a) The maximum axial strain in compression increases with the area ratio AR (Fig. 18b) and the outside cutting edge angle (Fig. 18c); (b) whereas an increase of the inside clearance ratio ICR decreases the maximum axial strain in compression, it significantly increases the maximum axial strain in extension. As indicated in Fig. 18c for a cutting edge angle of 9.9°, it increases from 0.3% to 1.15% when ICR is changed from 0.5% to 1.5%. For similar reasons, La Rochelle at al. (1981) suggested to eliminate inside clearance for the sampling of soft clays.

Such considerations have led to the development of high quality samplers such as the Sherbrooke one (Lefebvre & Poulin, 1979) which does not use any tube and provides samples with a diameter of 300 mm, and the 200 mm in diameter Laval sampler (La Rochelle et al., 1981). In this latter case, a sampling tube is used with an overcoring system to eliminate the effect of suction when the sampler is pulled out of the ground.

The studies made on different sites with different samplers and geometries generally confirm the results obtained by Baligh (1985) and Clayton et al. (1998). From a study made in Ariake clay, Japan, Tanaka (2000) reports average normalized undrained shear strengths obtained from unconfined compression tests equal to 1.03 for the Sherbrooke sampler, 0.98 for the Laval sampler, 0.96 for the Japanese sampler, 0.86 for the NGI54 sampler, 0.65 for the Shelby tube sampler and 0.61 for the ELE100 sampler. The poor performance of the three last samplers seems to be associated to the facts that no piston was used with the Shelby tube, the area ratio is quite high for the NGI54 sampler (54%, see Fig. 18b) and that the ELE100 sampler had a cutting edge angle of 30°, compared to 6° for the Laval and Japanese samplers (see Fig. 18c), and also a length of only 500 mm in that study. The other comparative studies generally confirm the previously described tendency.

In fact, sampling quality depends on how strains induced by sampling compare with those necessary to break bonds between particles or aggregates, which in turn depends on the soil considered. As indicated by Hight (ref. by Clayton et al., 1998), the Y_2 surface at which irrecoverable strains starts developing will be exceeded during tube sampling but that, depending upon soil characteristics and sampler design, main yield (Y_3) may or may not be exceeded. For example, Lacasse et al. (1985), comparing the quality of Norwegian clay samples taken with the Sherbrooke sampler and with the NGI95 sampler concluded that the difference is not significant for plastic clays but

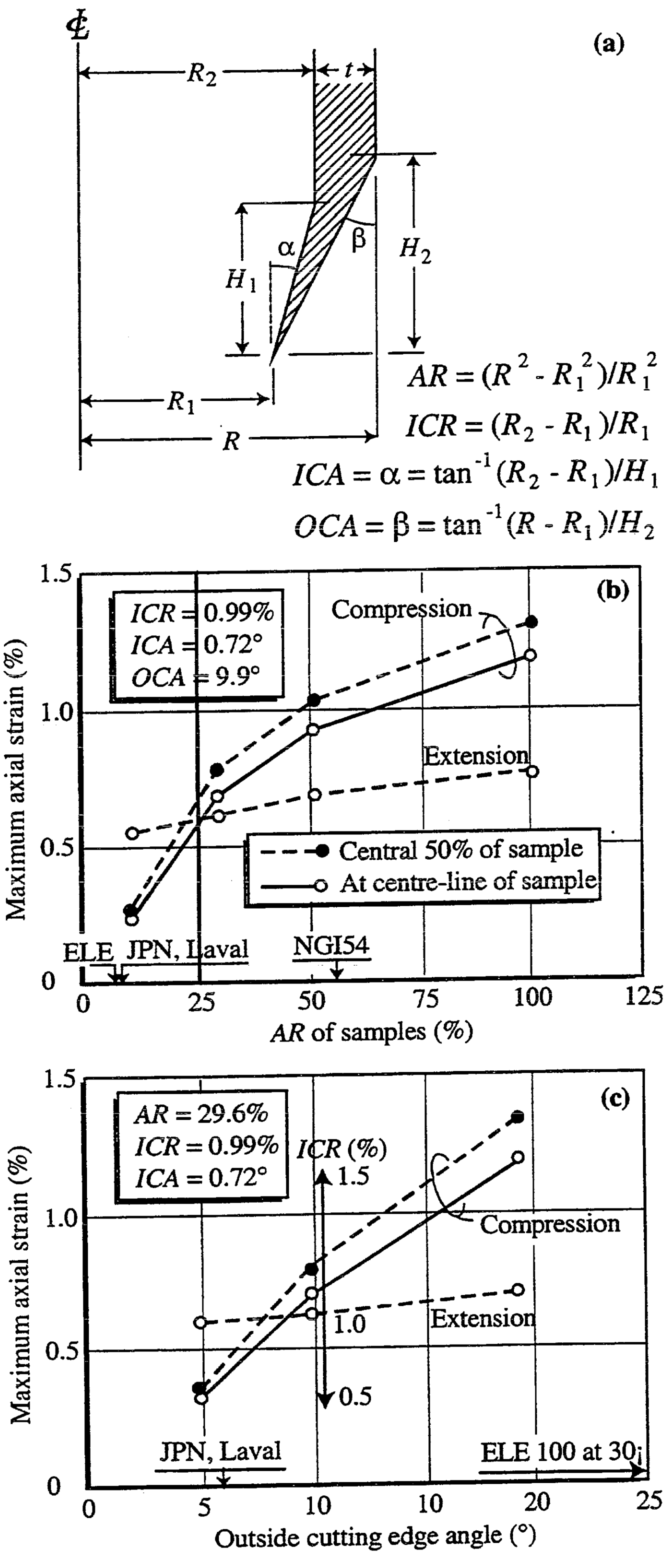


Fig. 18 - Influence of sampler geometry on the strains generated during tube penetration : (a) Geometry of a tube sampler; (b) Influence of area ratio; (c) Influence of outside cutting edge angle (after Clayton *et al.*, 1998)

found a clear superiority of the Sherbrooke sampler in low plasticity clays. The design of a sampler thus aims at finding a combination of parameters that minimizes induced strains while keeping the tube rigid enough. At the present time, the best quality samplers for clays seem to be the Sherbrooke, Laval and Japanese samplers. The area ratio and the cutting edge angle of the Japanese and Laval samplers are indicated in Figs. 18b and c.

Several approaches have been proposed for assessing sample quality. Measurement of the residual effective stress in the soil after sampling is one. Hight et al. (1992) showed that it can be an interesting criterion on a given site. On the other hand, Tanaka (2000) reports residual stresses much smaller than the mean effective stress in situ (often in the order of $p'_o/6$) in apparently high quality samples, which indicates that this parameter cannot be used as a general indicator for assessing sample quality. A better criterion seems to be the volumetric strain ε_v resulting from reconsolidation of a soil specimen to its in situ effective stresses (Andresen & Kolstad, 1979) or the relative change in void ratio $\Delta e/e_o$, as suggested in Table 1 from Lunne et al. (1997).

Table 1 Proposed criteria to evaluate sample disturbance (Lunne *et al.*, 1997)

Overconsolidation ratio	$\Delta e/e_o$			
	Very good to excellent*	Good to fair*	Poor*	Very poor*
1 ~ 2	<0.04	0.04~0.07	0.07~0.14	>0.14
2 ~ 4	<0.03	0.03~0.05	0.05~0.10	>0.10

*The description refers to use of samples for measurement of mechanical properties

Another criterion that can be used on a regional basis is the strain at yielding, either the strain at failure in triaxial tests or the strain at the passage of the preconsolidation pressure in oedometer tests.

The small strain shear modulus measured in laboratory can also be affected by destructuration and sampling disturbance of the clay (Fig. 16) and could also possibly be used as a quality criterion. There would, however, be a need for a reference. Considering 6 Italian clays, Jamiolkowski et al. (1994) found that, after reconsolidation under in situ effective stresses and with specimens allowed to experience one cycle of secondary compression, G_o measured in laboratory was slightly smaller than the field values. This was despite from the fact that the void ratio after reconsolidation in laboratory was slightly smaller than e_o in the field.

Practical implications in in situ conditions

Microstructure also has to be preserved during field works. A good illustration is provided by Phien-Wej & Chavalitjiraphan (1991) who described a test excavation in Bang Bo district, 80 km southeast of Bangkok. The test excavation had 4 slopes and a depth of 4 m: the non-treated 1:4 slope was expected to be the most critical and to fail in short-term conditions; the slopes treated with soil-cement piles and sand compaction piles were not expected to fail; the 1:6 non-treated slope was expected to fail in long-term conditions. However, the 1:4 non-treated slope turned out to be the most stable one; its failure could only be obtained after steepening the slope to 1:3. On the other hand, the three other slopes failed before completion of excavation to full depth. Phien-Wej & Chavalitjiraphan (1991) attributed this behaviour to disturbance of the soil by heavy equipment used prior to excavation and during soil improvement, and to the fact that the non-treated 1:6 slope was initially used as an access path to the test area. This hypothesis was confirmed by pore pressures in excess of the hydrostatic pressure observed at the start of excavation in all slope areas, except the 1:4 slope.

Another example of the effect of disturbance is shown by Shirlaw et al. (1999) who, in relation with tunnel projects, assessed the use of compensation grouting in Singapore marine clay. From the observations presented in the paper, heave at the surface was in the order of 20 mm at the end of the grouting period (Trial 4); excess pore pressures had been generated and progressively dissipated over a period of about 100 days. At that time, 9 of the 30 monitoring points showed an absolute settlement or no heave at all. It is thought that the clay had been partly destructured by grouting and reconsolidated under essentially the same overburden stress at a void ratio smaller than the initial one. Application of dead loads, use of heavy equipments and generation of vibrations or deformations have thus to be considered carefully if preservation of microstructure of soft clay is important.

Destructuration of clayey soils can also be obtained by swelling associated to a reduction of effective stress in soils containing swelling minerals (Leroueil & Vaughan, 1990) and by freezing (Graham & Au, 1985; Leroueil et al., 1991). Lunne et al. (2001) also show how clays taken at large depths under water can be destructured by gas exsolution.

So, in conclusion, most natural clays are microstructured and maximum effort must be put to preserve this microstructure at all stages of a project.

INFLUENCE OF FABRIC

The arrangement of particles or aggregates in a soil also influences its mechanical behaviour. This is true for all soils but appears particularly important for sandy materials. Figure 19 shows simple shear test results obtained on loose fine uniform Syncrude sand. The specimens were reconstituted by moist tamping (MT), air pluviation (AP) and water pluviation (WP), which produced different fabrics, and were tested at the same void ratio. The moist tamped specimen is highly contractive. Air pluviated specimen is also strain softening but to much smaller degree. In contrast, water pluviated specimen behaves in a strain hardening (dilative) manner. At shear strain of 15%, the shear stress mobilized on the WP specimen is 10 times higher then the shear stress mobilized on the MT specimen. Vaid et al. (1999) observed similar behaviour when performing undrained triaxial compression and extension tests on Fraser River sand.

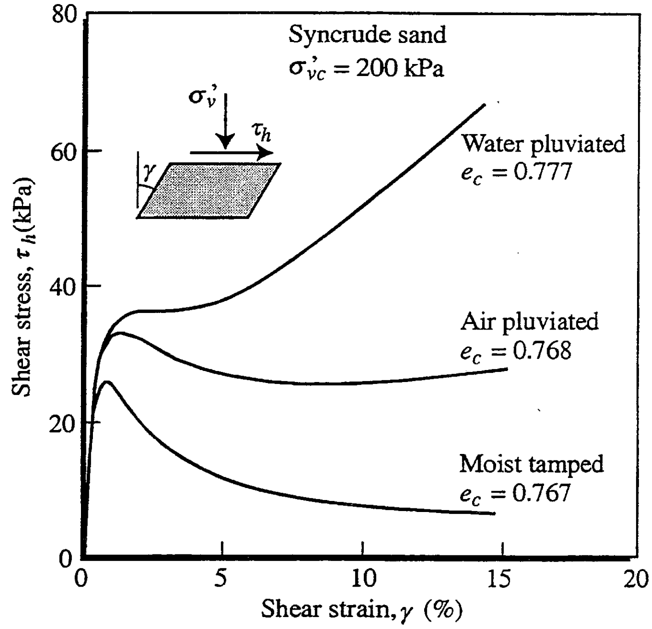


Fig. 19. - Effect of specimen reconstitution method on undrained simple shear response (after Vaid *et al.*, 1995)

Hoeg et al. (2000) compared the undrained behaviour of natural silt and a silty sand tailings in both undisturbed and reconstituted conditions. The natural silt was from a 10,000-year-old fluvial deposit and mine tailings were deposited hydraulically 5 years prior to the investigation. About half of the 26 undrained compression triaxial tests presented were performed on reconstituted specimens, generally prepared by moist tamping. In all cases, the undisturbed specimens showed dilative and ductile behaviour, whereas in all but a few cases, the reconstituted specimens showed contractive behaviour. A typical comparison is shown in Fig. 20 for silt. Even if the relative density was smaller (e=0,81, Dr=66%) than that of the reconstituted specimens (e=0,73, Dr=73%), the undisturbed specimen shows a dilative behaviour with continuous strength increase, whereas the MT specimen shows a low peak and brittle behaviour.

Differences in behaviour are associated to differences in fabric between specimens prepared with different methods or between undisturbed and reconstituted soil specimens. It is in particular thought that moist tamping generates aggregates of particles with relatively large voids in between, thus a potentially collapsible fabric. Vaid and Sivathayalan (2001) consider that WP more closely replicates the fabric of water deposited in-situ sands. Indeed, for two sands, they observed very similar undrained soil response on undisturbed and WP specimens. However, WP could not reproduce microstructure (bonding between particles) that may exist in a cohesionless deposit. So, as for clays, high quality sampling of these materials is encouraged.

EFFECTS OF STRAIN RATE

Generalities

Leroueil & Marques (1996) reviewed the main effects of strain rate (and temperature) on soil behaviour. Their main conclusions can be summarized as follows:

- The strength envelope of the normally consolidated soil or critical state line is not significantly influenced by strain rate. On the other hand, the strength envelope of the soil in its overconsolidated range may be influenced by strain rate.
- The preconsolidation pressure and the entire compression curve of clayey soils are strain rate dependent. In fact, the entire limit state surface is strain rate dependent.
- It results from these viscous effects that the undrained shear strength depends on strain rate. Compiling data obtained from 26 different overconsolidated and normally consolidated clays, Kulhawy and Mayne (1990) found a typical decrease of undrained shear strength per logarithm cycle of strain rate of 10% (Fig. 21). Lacasse (1994) indicates that the effect can be more important, in particular at high strain rates and at high OCRs.
- The effect of strain rate on G_o is relatively small, less than 6% per tenfold increase in strain rate for clays. Lo Presti et al. (1997) show, however, that the effect increases with the shear strain level. Tatsuoka et al. (1998) indicate that a strain rate effect is observed at a small strain of 0.001% in most geomaterials, but is generally smaller than for clays.

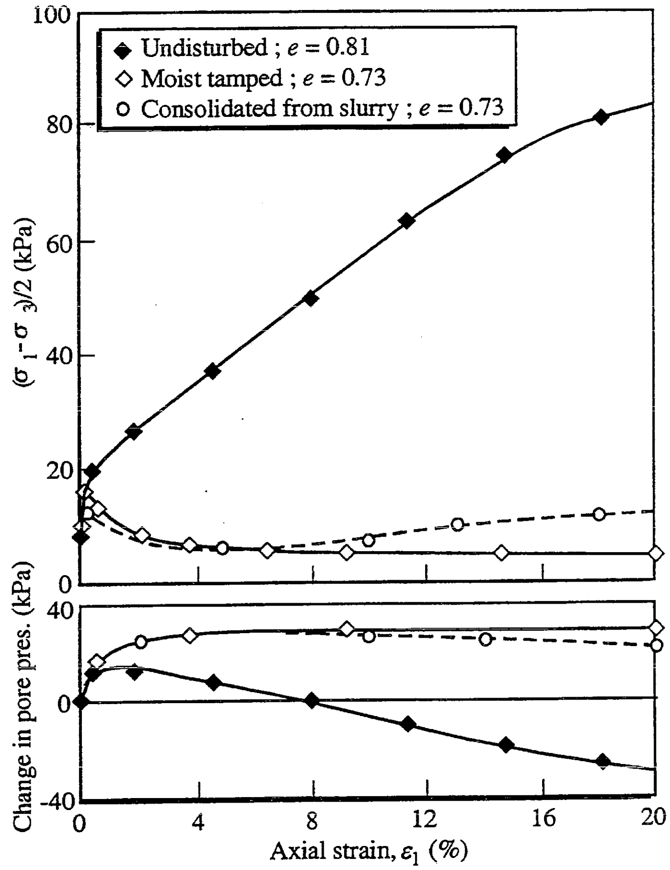


Fig. 20. - Comparison of typical stress-strain curves and pore pressure development for undisturbed, moist tamped and slurried specimens of natural silt (after Hoeg *et al.*, 2000)

Practical implication

The effect of strain rate is not limited to laboratory tests. It has also been observed with in situ tests (Leroueil &

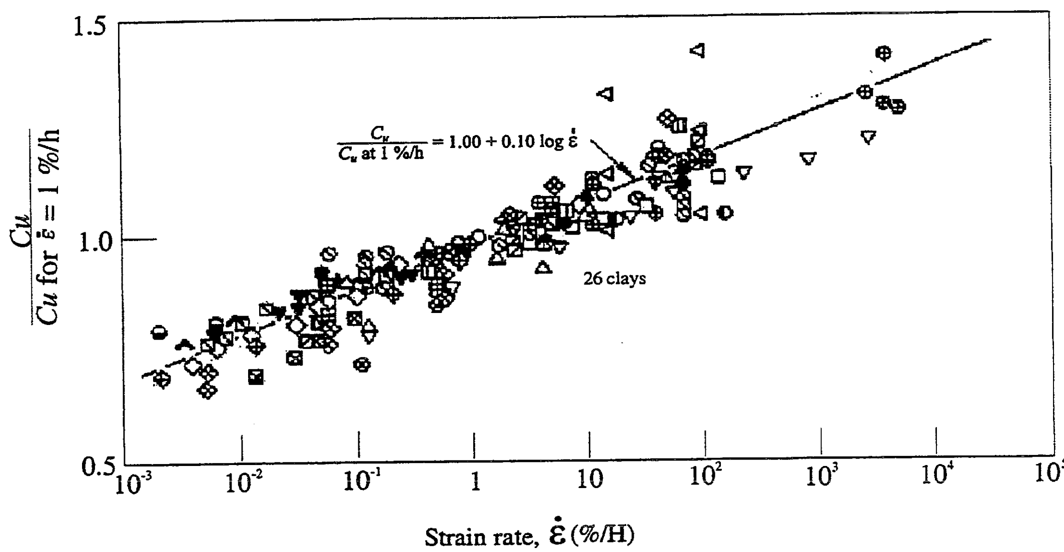
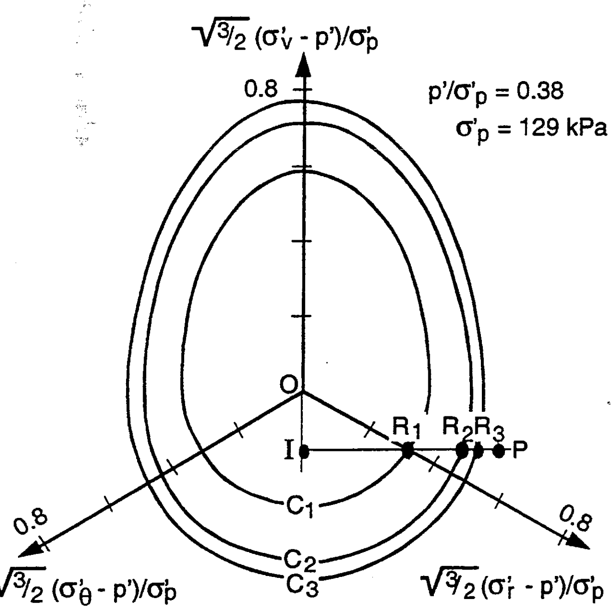


Fig. 21 - Influence of strain rate on the undrained shear strength measured in triaxial compression (from Kulhawy and Mayne, 1990)

Marques, 1996); it may, in particular, be worth mentioning here the interpretation of the self-boring pressuremeter test in clays made by Hamouche (1995).

As many other researchers, Hamouche (1995) obtained on Eastern Canada clays undrained shear strengths deduced from self-boring pressuremeter tests larger than those obtained through more conventional tests such as the field vane test. To examine this point more closely, a detailed study was performed on Louiseville clay. This study included self-boring pressuremeter tests at different depths and a series of tests in a "true triaxial" cell to define the 3-D failure surface (see Fig. 13). With the assumption that the soil is linear-elastic and undrained during the early stages of the pressuremeter test and that strains are constrained to a horizontal plane, it can be shown that the stress path then followed is at σ'_v = cst = σ'_{vo} in a p' = cst plane, with the mean effective stress p' equal to the mean effective stress in the soil prior to testing, p'_o. Based on these assumptions, the stress path followed during the pressuremeter test performed at Louiseville at a depth of 5.8 m is shown in Fig. 22. The stress path goes from the initial stress conditions I to yield conditions P at which point the undrained shear strength deduced from the pressuremeter test results is reached. Also shown on the figure is the limit state curve C_1 deduced from "true triaxial" tests (Fig. 13) for a mean effective stress equal to p'_o. It can be seen that P is quite different from stress conditions R_1 on the laboratory limit



C_1 : Laboratory limit state ($\dot{\varepsilon} = 3 \times 10^{-7}$ s^{-1} ; $T = 20$ °C)

C_2 : Limit state curve corrected for strain rate ($\dot{\varepsilon} = 10^{-4}$ s^{-1} ; $T = 20$ °C)

C_3 : Limit state curve at strain rate and temperature ($\dot{\varepsilon} = 10^{-4}$ s^{-1} ; $T = 10$ °C) corresponding to in situ test conditions

Fig. 22 - Yielding of Louiseville clay (5.8 m), as deduced from in situ pressuremeter test and from laboratory tests performed in TTA (from Hamouche, 1995)

state curve, which corresponds to a much smaller different shear strength. However, during self-boring pressuremeter tests, the strain rate up to failure was typically 10^{-4} s^{-1} and the temperature approximately equal to 10°C, whereas laboratory tests were performed at a strain rate of 3 x 10^{-7} s^{-1} and at a temperature of 20°C. When, according to the viscous behaviour previously indicated, the limit state curve C_I is corrected for a strain rate of 10^{-4} s^{-1} and then for a temperature of 10°C, it moves to C_2 and then C_3 (Fig. 22). The intersection of the stress path followed during the pressuremeter test with the yield curve C_3 corresponding to the same strain rate and temperature, i.e. R_3, then becomes closer to the yield conditions P deduced from the in situ test. Viscosity of clay thus seems to be the main reason for getting higher undrained shear strength with self-boring pressuremeter tests.

Viscous effects also have practical implications when we want to apply tests results to field conditions where strain rate and temperature are different. For example, Lacasse (1994) points out the importance of considering strain rate effects when we want to apply to foundations of offshore structures subjected to large waves with periods typically between 10 and 20 s laboratory test results in which failure is obtained in about 140 min.

Another implication is the evaluation of settlements of embankments where in situ strain rates are several orders of magnitude smaller than those existing in conventional oedometer tests. This author discussed that point several times, but recent data related to what are probably the most important actual projects on soft clays justify that this topic be examined again.

Detailed studies performed by Kabbaj et al. (1988) on four different sites from eastern Canada and Sweden showed that, compared to the end-of-primary (EOP) compression curves obtained in incremental loading oedometer tests on high-quality samples, the in situ stress-strain curves were well below, as schematically shown in Fig. 23. At a given effective stress, the in situ strain (or settlement) is larger than that expected on the basis of EOP laboratory curves. Leroueil (1988) reviewed other cases in which measured and computed behaviour in terms of settlement and pore pressure had been compared. In the 8 cases considered, a relatively good agreement was found between observed and computed settlements (Most of the time by modifying the hydraulic conductivity), but, except for one case, the measured excess pore pressures were higher than the computed ones. Similar behaviour has recently been reported by Cao. et al. (2001) who studied the consolidation of a marine clay deposit at Changi, Singapore, in connection with a pilot test for a large reclamation project. As indicated in Fig. 23, if at a considered strain, the measured excess pore pressure (FS in Fig. 23) is larger than the predicted one (FL in Fig. 23), it implies that the in situ stress-strain condition is below the laboratory compression curve. This is in agreement with the behaviour observed by Kab baj et al. (1988). Experience gained in Sweden (Larsson, 1986 and in France (Magnan, 1992 also confirms this

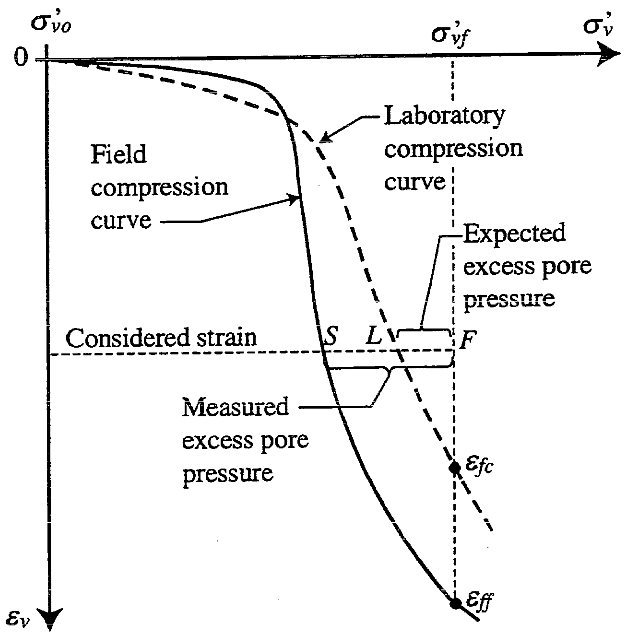


Fig. 23 - Typical compression curves both in situ and in the laboratory

behaviour. As indicated by Leroueil (1988, 1996), this behaviour is mostly due to the viscous behaviour of clays since strain rates in situ are several orders of magnitude smaller than those existing in conventional laboratory tests. Recent data related to settlements of the *Kansai International Airport (KIA)* island, Japan, provide complementary information on the topic.

The construction of the Kansai International Airport island, a large reclamation project (511 ha) 5 km offshore in Osaka Bay, began in 1987 and was completed in December 1991. The airport was inaugurated in September 1994 (Arai et al., 1991; Akai and Tanaka, 1999; Akai, 2000). The water depth at this location was about 18 m. The subsoil consists of an about 18 m of soft alluvial clay over several hundreds of metres of Pleistocene clay. These latter deposits show alternating clay and relatively thin sand or gravel layers. The overconsolidation ratio of Pleistocene clay increases with depth to reach a value of about 1.4 at a depth of 160 m. Up to that depth, the pleistocene clay has, a plasticity index typically between 50 and 60 and a liquidity index that decreases with depth from about 0.7 to about 03. The alluvial clay layer was treated with sand drains and rapidly settled in about 6 months. The settlement observed since are due to compression of Pleistocene deposits over a thickness of about 150 m.

Fig. 24 shows the profile of excess pore pressures in the Pleistocene clays obtained early 1992 and in 1997, i.e. soon after completion and 5 years later. Fig. 25 shows the measured and calculated settlements of Pleistocene deposits as reported by Akai and Tanaka, 1999. The calculations were made on the basis of conventional oedometer test results. At the time of the inauguration (sept. 1994), the measured and calculated settlements were about the same, at 4.50 m. Since, the rate of settlement is larger than predicted with values of 40 cm in 1995, 37 cm in 1996, 33 cm in 1997 and 30 cm in 1998 (Akai, 2000). So, the observed settlement was approaching the predicted final settlement of 5.84 m (Akai, 2000) at the time the 1997 pore pressures were measured. However, as indicated in Fig. 24, the excess pore pressures, which decrease slowly, were still close to 200 kPa between depths of 90 and 120 m at that time. The Pleistocene clay deposit is thus still far from the end of primary consolidation. Also, due to clay viscosity, delayed settlements of the layers below a depth of about 150 m, that remained at vertical effective stresses only slightly smaller than the preconsolidation pressure, are expected (see Bjerrum, 1967).

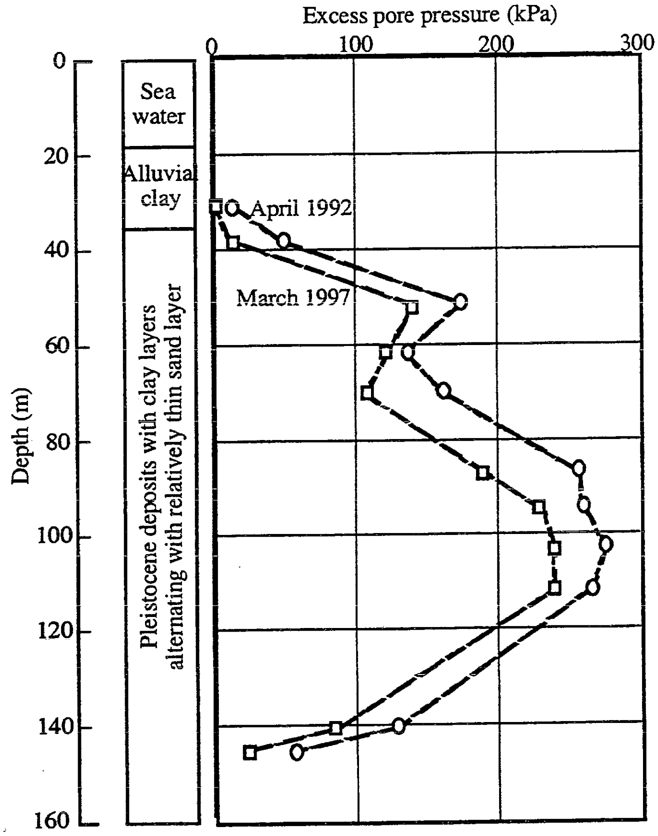


Fig. 24 - Excess pore pressure below Kansai International Airport Island (Monitoring location N 4 ; after Akai and Tanaka, 1999 ; Akai, 2000)

Due to the size of the project (stress applied by reclamation close to 500 kPa), the excess pore pressures observed under KIA Island are much larger than those observed in other reported cases. However, the behaviour appears to be the same: under the same measured and calculated settlement, the pore pressures measured in situ are larger than the predicted ones, as depicted in Fig. 23. Kobayashi (2000) suggested that the behaviour observed at Kansai could be due to the presence of microfossils, diatoms in particular, in the Pleistocene clay. This author does not agree with this hypothesis: (a) The microfossils were also in the specimens tested in laboratory and can not explain the discrepancy between laboratory and field compression curves; (b) the behaviour observed at Kansai is quite similar to the one observed elsewhere and thus seems normal. Strain rate effect on the compressibility of Kansai clay has been shown (Tanaka et al., 2001) and the fact that the strain rate in situ (about 5 x 10^{-11} s^{-1} in 1998) is more than 3 orders of magnitude smaller than the one in conventional laboratory tests can explain the observed behaviour. Kansai reclamation project thus confirms that under a given effective stress, in situ strain, and thus settlement, are larger than those deduced from laboratory tests, and this is mostly attributed to viscosity of clayey soils.

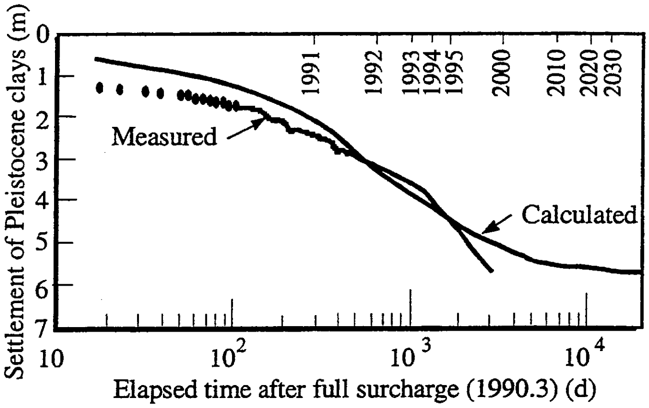


Fig. 25. - Calculated and measured settlement of Pleisto-Cene clays, Kansai International Airport Island (Monitoring location N 2-1 ; after Akai and Tanaka, 1999)

Most of the well documented cases analysed up to now were from Eastern Canada and Scandinavia, in clays that have a liquidity index generally larger than or close to 1.0. The cases of Kansai International Airport and Changi Airport (Cao. et al., 2001) show that strain rate also influences clays with liquidity index between 0.7 and 0.3. This is in agreement with laboratory studies and is thus not surprising. These results confirm the conclusion made by Leroueil (1988, 1996) that in highly compressible clays or for embankments of special importance (test embankments for which the magnitude of settlements may have important consequences) the viscous component of settlements can be significant and should be considered.

CONCLUSION

Beyond classical soil mechanics there are peculiar aspects that influence the behaviour of soils in laboratory and in the field. Five of these aspects are examined in this paper: non-linearity of stress-strain behaviour; anisotropy; microstructure; fabric and viscosity. After generalities, some practical implications are presented.

For the engineer, it is not always easy to quantatively evaluate the influence of this or that aspect. On the other hand, it is very important for the engineer to know what can be their implications in order to judge if they can have important consequences for the considered project.

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