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CHARACTERIZATION OF THE ANISOTROPY OF A NORMALLY CONSOLIDATED SOFT CLAY

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Abstract: The natural soils were deposited in successive layers during thousands of years. They develop different horizontal and vertical shear strengths, characteristic of their *in-situ* anisotropic behaviour. This article describes the anisotropy of the mechanical properties of resistance and deformability of a normally consolidated soft clay, based on the standard triaxial test results. The results presented highlight the anisotropic character of this natural clay and determine its parameters of aelotropy. The analysis carried out made it possible to note that the hyperbolic approach of determining deformation moduli (nonlinear elastic behaviour) is adapted probably better than the linear elastic approach (Hooke's law) and undoubtedly more relevant than the analysis of structures founded on anisotropic soils.

LIST OF SYMBOLS

σ'_c – effective confining stress,
σ'_{vo} – effective overburden pressure,
$(\sigma_a - \sigma_r)_f$ – deviatoric stress at failure,
Δu_f – excess pore pressure corresponding to $(\sigma_a - \sigma_r)_f$,
ε_{af} – axial strain corresponding to $(\sigma_a - \sigma_r)_f$,
ε_{vf} – volumetric strain corresponding to $(\sigma_a - \sigma_r)_f$,
s_u – undrained shear strength,
E_u – undrained deformation modulus,
E'_v – vertical Young's modulus,
E'_h – horizontal Young's modulus,
ν'_{hh} – Poisson's ratio corresponding to a horizontal plane,
ν'_{vh} – Poisson's ratio corresponding to a vertical plane,
G'_v – vertical shear modulus,
K_o – coefficient of earth pressure at rest,
CK_oIU – K_o consolidation-undrained triaxial compression test,
EK_oIU – K_o consolidation-undrained triaxial extension test,
CID – isotropic consolidation-drained triaxial compression test.

1. INTRODUCTION

From their mode of formation and preconsolidation under the action of gravity and overloads (embankments, buildings, etc.), the natural or transported clayey deposits present various types of anisotropy: an inherent anisotropy, due to the arrangement of the mineral or organic particles constituting the skeleton, and an induced anisotropy, dependent on the history of their deformations. The first type of anisotropy appears during the formation of material because of its granularity and develops during the seasonal variations, which involve the modification of its physical and mechanical characteristics. The second type of anisotropy results from a continuous modification of the structure of the material which the clayey deposits undergo under the action of anisotropic states of stresses or various physicochemical phenomena. The natural soils, in general deposited in successive horizontal layers, had to undergo in the past the same state of stresses. Taking into account these conditions of formation, the layers constituting the natural or transported deposits must have mechanical properties equivalent in all the directions of their bedding plane, but different from those corresponding to the planes containing the axis of revolution (normal axis in the bedding plane), from where the name of orthotropism of revolution or aelotropy is acquired.

Several experimental methods can be used for the study of the anisotropy of soils: laboratory methods based primarily on the direct shear test, the standard triaxial shear test and the cubic triaxial shear test, and the in-situ methods based on the vane shear test, the cone penetration test and the pressiometer test. A detailed bibliographical synthesis of these experimental methods is presented by MEFTAH [18]. The results obtained made it possible to privilege the standard triaxial shear test for all the types of soils. This test makes it possible to determine the principal parameters of anisotropy of the clayey soils, and to characterize the difference in their behaviour in the horizontal and vertical directions and the interactions between these two directions.

Compared to the results of theoretical studies, the experimental data on the anisotropy of the soils are very few and do not seem to provide, in the current state of knowledge, the bases necessary for introducing this property into the operational methods of the analysis of structures. The experimental studies dealing with the anisotropy of the natural soils were carried out in the traditional formalism of linear elasticity (LEKHNITSKII [11], PICKERING [21]). The following works can be cited: ATKINSON [2] on the London stiff clay, LO et al. [13] on the Léda sensitive clay (Canada), GRAHAM and HOULSBY [5] on the Winnipeg plastic clay (Canada), PIYAL and MAGNAN [23] on the Cubzac-les-Ponts organic soft clay (France) in an overconsolidated state and LINGS et al. [12] on the Gault stiff clay (the Netherlands). We can note that these works relate to the overconsolidated clays characterized by a reversible behaviour to which the linear elastic approach can well apply. Other contributions are based on nonlinear formulations to identify the deformation moduli of soils (ATKINSON [1], HOULSBY et al. [6], MAYNE and NIAZI [17]), but these approaches are not checked yet on all the types of soils. In addition, there exist other approaches based on the measure of the wave velocity in the elastic bodies (SHIBUYA et al. [25])

or on the triaxial tests with bender elements (VIGGIANI and ATKINSON [26], LOHANI et al. [14], FIORAVANTE and CAPOFERRI [4], NGUYEN et al. [19], PIRIYAKUL and HAEGEMAN [22]) to characterize the anisotropy of the reconstituted soils. However, the generalization of these experimental methods to the intact natural soils does not seem obvious.

This study has the aim of checking the applicability of the approaches evoked above to a normally consolidated soft clay: the Guiche clay (the Adour valley, France). It aims at characterizing the anisotropy of the mechanical properties of resistance and deformability of this natural clay and at determining its parameters of aelotropy, according to the traditional formalism of linear (Hooke's law) and nonlinear (elasticity of hyperbolic type) elasticity. The experimental procedures implemented in the study of the behaviour of the Guiche clay and the test program realized, thus the detailed results obtained, are in the publications of KHEMISSA et al. [8]–[9]. Also, we will describe hereafter only the test results that appear to be interesting.

2. SUMMARY DESCRIPTION OF THE GUICHE CLAY

The sampling was carried out between 10 and 15 m of depth in the experimental site of Guiche (the Adour valley, France), where significant experimental embankments were built for the construction of the A64 motorway between Bayonne and Pau. Table 1 gives the range of variation of the geotechnical characteristics of the samples tested and their mean values.

Table 1

Geotechnical characteristics of the Guiche clay (the Adour valley, France)

Parameters	Symbols	Range of variation	Mean values
Wet unit weight	γ (kN/m^3)	14.8–18.0	16.3
Moisture content	w (%)	46–85	55
Liquid limit	w_L	48–98	68
Plasticity index	I_p	26–61	49
Consistency index	I_c	0.1–0.4	0.28
Content of organic matter	c_{MO} (%)	-	4.3
In-situ void ratio	e_o	1.39–1.87	1.62
Compression index	C_c	0.46–0.99	0.74
Swelling index	C_s	0.05–0.13	0.08
Preconsolidation pressure	σ'_p (kPa)	40–90	70
Overconsolidation ratio	$R_{oc} = \sigma'_p / \sigma'_{vo}$	0.60–1.25	1.04
Compression ratio	$C_c/(1+e_o)$	0.19–0.36	0.28

According to LPC classification used in France (MAGNAN [16]), the Guiche clay consists of a slightly organic and very plastic silty soft clay (fo-Lt). The overconsolidation ratio and compression ratio values confirm the character of this natural clay normally consolidated and normally compressible.

3. EXPERIMENTAL PROGRAM AND TESTING PROCEDURE

To highlight the anisotropic character of the Guiche clay and to determine its parameters of aelotropy, the experimental program comprised:

- undrained triaxial compression tests carried out on samples reconsolidated with the effective stresses in place, then consolidated under isotropic stresses (CK_oIU-tests);
- undrained triaxial extension tests carried out on samples reconsolidated with the effective stresses in place, then consolidated under isotropic stresses (EK_oIU-tests);
- drained triaxial compression tests carried out on vertical and horizontal samples consolidated under isotropic stresses (CID-tests).

These tests were performed by means of standard triaxial cells, of the Wykeham Farrance type, modified for the automatic acquisition of measurements. The operations carried out successively in the various executed tests comprised:

- an initial phase, common to all tests comprising the dry assembly and the saturation of the drainage systems, the measurement of the initial pore pressure u_i and the application of a backpressure u_{cp} = 150 kPa comparable with the pore pressure in place;
- a phase of isotropic consolidation under an effective confining stress $\sigma'_c = 2/3\,\sigma'_{vo}$ for the CID-tests, or of reconsolidation to the effective stresses in place with K_o = 0.55 for the CK_oIU and EK_oIU-tests; σ'_{vo} being the effective overburden pressure. After cancellation of the deviatoric stress with constant volume and constant radial stress and after application of a cellular pressure σ'_{co} such that the pore pressure $u = u_{cp}$, the reconsolidation with the effective stresses in place of the samples used in the CK_oIU and EK_oIU-tests was followed by a new isotropic consolidation under $\sigma'_c \neq \sigma'_{co}$;
- a phase of drained shear for the CID-tests, or undrained for the CK_oIU and EK_oIU-tests.

The reconsolidation with the effective stresses in place aims at reproducing the loading state of the clay in place.

4. TEST RESULTS AND ANALYSIS

Tables 2 and 3 present the values of the shear and rupture parameters deduced from the shear curves depicted in figures 1 and 2. The deduced values show that for the CK_oIU and EK_oIU-tests the deviatoric stress at failure $(\sigma_a - \sigma_r)_f$ and the corresponding excess pore pressure Δu_f increase with the effective confining stress. In addition, it is noted that for the CID-tests some dispersion of the results which can be due to the effects of disturbance of the soil (KHEMISSA [7]) is related to the conditions of sampling (MAGNAN et al. [15]).

Table 2

Values of the shear and rupture characteristics deduced from the CK_oIU and EK_oIU-tests

Bore/Core sample	Depth z (m)	Tests	σ'_c (kPa)	$(\sigma_a - \sigma_r)_f$ (kPa)	ε_{af} (%)	Δu_f (kPa)
F2/E2	12.80–12.90	CK_oIU-1	50	74.8	3.4	25.9
		CK_oIU-2	40	63.4	6.3	21.4
		CK_oIU-3	30	41.6	5.4	11.9
F2/E3	13.12–13.22	EK_oIU-1	50	–40.9	–7.1	–3.6
		EK_oIU-2	40	–35.6	–7.6	–16.4
		EK_oIU-3	30	–23.2	–6.2	–14.2

Table 3

Values of the shear and rupture characteristics deduced from the CID-tests

Bore/Core sample	Depth z (m)	Tests	σ'_c (kPa)	$(\sigma_a - \sigma_r)_f$ (kPa)	ε_{af} (%)	ε_{vf} (%)
F1/E7	14.6–14.8	Vertical samples				
		CID-1V	60	116.0	5.16	5.46
		CID-2V		146.1	16.75	16.81
		CID-3V		162.6	19.31	19.46
		CID-4V		139.4	13.46	13.90
		CID-5V		118.4	11.64	12.10
		Horizontal samples				
		CID-1H	60	155.6	19.19	18.84
		CID-2H		156.8	15.71	16.37
		CID-3H		113.7	12.07	12.44
		CID-4H		115.9	9.53	9.93
		CID-5H		114.6	12.30	12.80

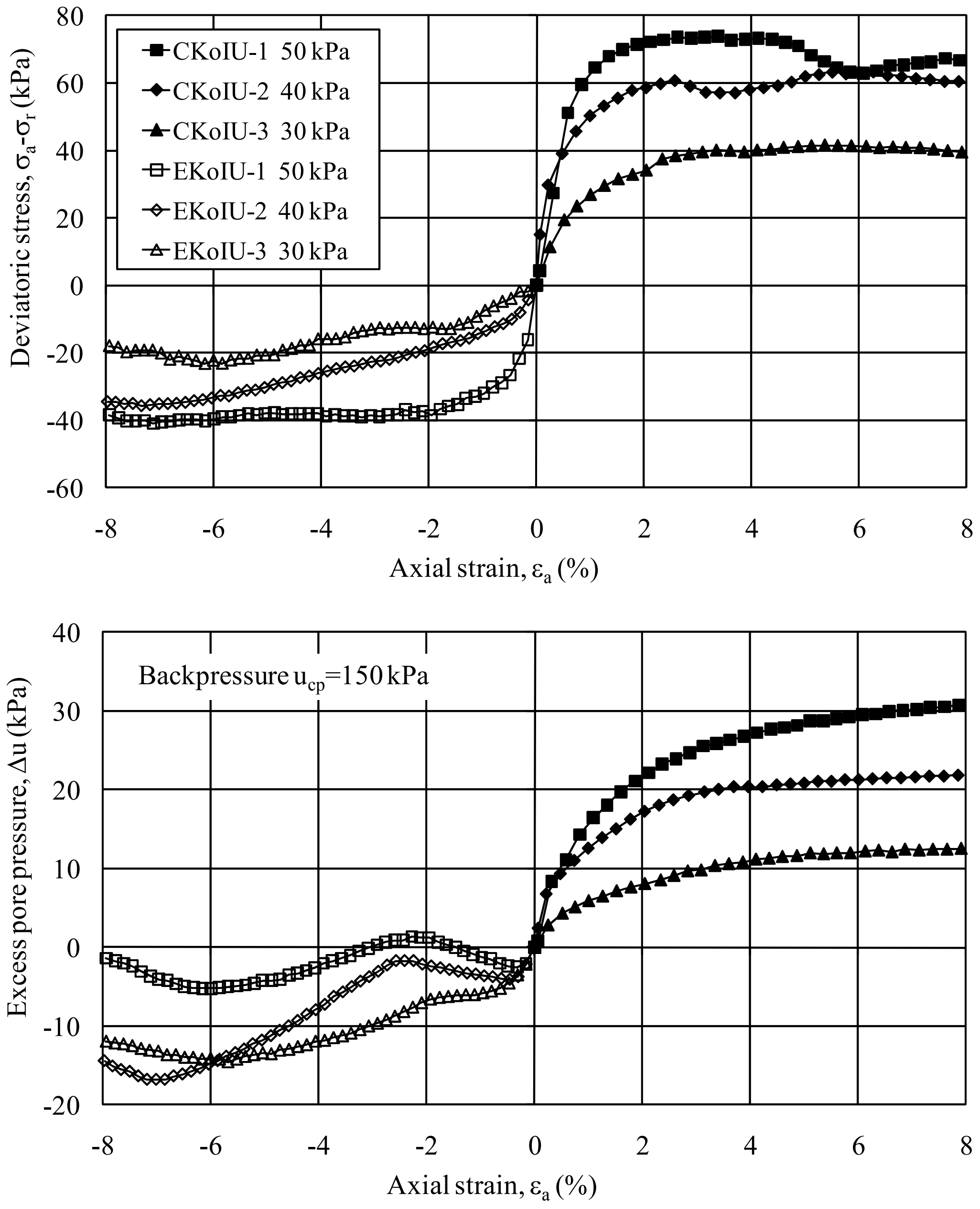


Fig. 1. Results of the CK_oIU and EK_oIU-tests

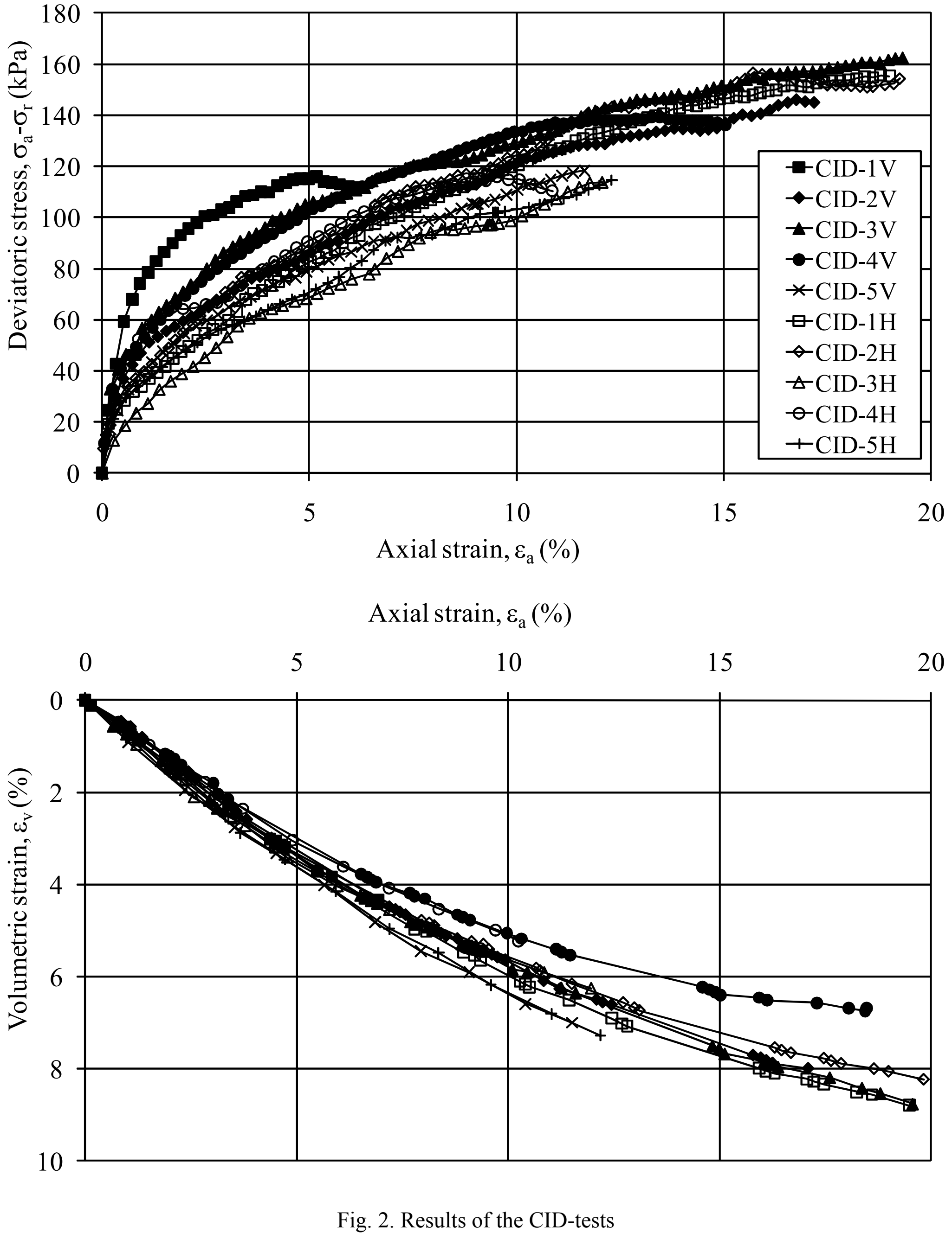


Fig. 2. Results of the CID-tests

4.1. ANISOTROPY OF THE MECHANICAL PROPERTIES

To highlight the influence of the direction of the loads on the resistance and the deformability of the Guiche clay we used the results of CK_oIU and EK_oIU-tests. Table 4 gives the values of the undrained shear strength $s_u = (\sigma_a - \sigma_r)/2$ and of the undrained deformation modulus E_u (corresponding to an axial strain equal to 0.5% because of uncertainty of the measures taken at the beginning of the shear phase and of their representativeness) obtained from the two types of tests, the ratios μ and κ of resistance and deformability anisotropy, and their respective values.

Table 4

Variation of the ratios of anisotropy with the effective confining stress

No.	σ'_c (kPa)	CK_oIU-tests		EK_oIU-tests		$\mu = \frac{s_u\|_{EK_oIU}}{s_u\|_{CK_oIU}}$	$\kappa = \frac{E_u\|_{EK_oIU}}{E_u\|_{CK_oIU}}$
		s_u (kPa)	E_u (MPa)	s_u (kPa)	E_u (MPa)		
1	50	36.91	8.79	20.46	5.35	0.55	0.61
2	40	31.71	7.95	17.81	2.08	0.56	0.26
3	30	20.78	3.77	11.60	0.82	0.56	0.22

4.1.1. ANISOTROPY OF RESISTANCE

The experiment shows that the undrained shear strength changes according to the orientation of the principal stresses or the plane of rupture. This change results in the ratios of the undrained shear strengths that relate to the principal horizontal and vertical planes different from 1. The values of the ratio μ of resistance, respectively obtained in extension and compression of the Guiche clay, vary from 0.55 to 0.56 according to the effective confining stress. Notwithstanding the problems of rigidity of the borders and imperfection of the triaxial shear test itself, this result seems to indicate that the undrained shear strength of clay in compression is higher along the vertical axis of revolution (i.e., in the direction of its deposit) that in the normal plane with this axis.

4.1.2. ANISOTROPY OF DEFORMABILITY

The experiment shows that the ratio of the undrained deformation moduli that relates to the principal horizontal and vertical planes can take different values according to whether the clay studied is normally consolidated or overconsolidated. The values of the ratio of the deformation moduli, respectively obtained in extension and compression of the Guiche clay, vary from 0.22 to 0.61 according to the effective confining stress. This result seems to confirm that a loading in the same direction of clayey deposit produces moduli of deformation greater than those corresponding to a normal loading in this direction.

4.2. PARAMETERS OF AELOTROPY

4.2.1. DEFINITION AND EXPRESSIONS

Figure 3 shows the distribution of the stresses in the ground and the corresponding parameters of aelotropy. In these notations, the vertical *z*-axis coincides with the symmetry axis, and the two horizontal axes *x* and *y* determine the bedding plane there, in which the properties of material are identical whatever the direction considered.

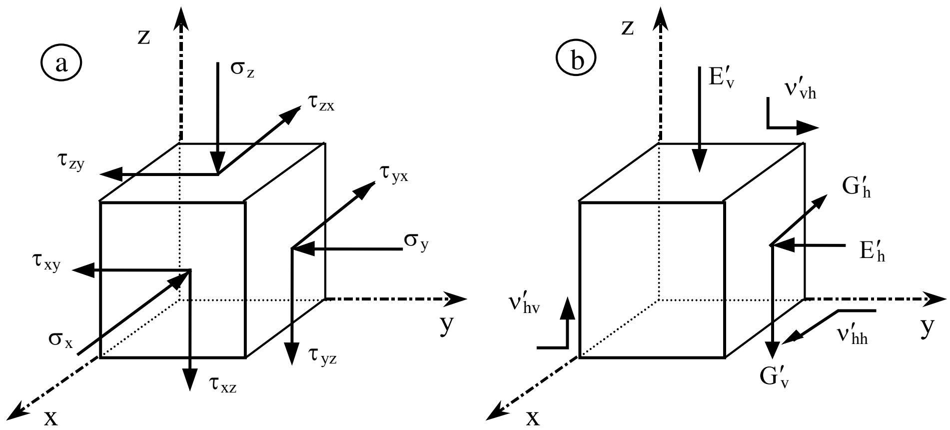


Fig. 3. Distribution of the stresses in the ground (a) and the corresponding parameters of aelotropy (b)

The parameters of aelotropy are five:

- E'_v : vertical Young's modulus in the direction of the symmetry *z*-axis;
- E'_h : horizontal Young's modulus in the bedding *x*–*y*-plane;
- ν'_{hh} : Poisson's ratio equal to the ratio of the strains along the *x*-axis and the *y*-axis under the effect of a variation of the normal stress according to the *y*-axis, and vice-versa;
- ν'_{vh} : Poisson's ratio equal to the ratio of the strains along the *x*-axis (or the *y*-axis) and the *z*-axis under the effect of a variation of the normal stress according to the *z*-axis;
- G'_v : vertical shear modulus, which characterizes the relation between the shear strain and the shear stress in a plane containing the isotropy axis.

The stress–strain relations utilize two other parameters expressed as follows:

- $\nu'_{hv} = \nu'_{vh} E'_h / E'_v$: Poisson's ratio equal to the ratio of the strains along the *z*-axis and the *x*-axis (or the *y*-axis) under the effect of a variation of the normal stress according to the *x*-axis (or the *y*-axis);
- $G'_h = E'_h / 2(1 + \nu'_{hh})$: horizontal shear modulus, which characterizes the relation between the shear strain and the shear stress in the isotropy plane.

Equations (1) to (4) connect the effective parameters of behaviour characterizing a vertical sample (index V) or horizontal one (index H) to the sizes measured during the drained shear with the standard triaxial apparatus (see appendix). Thus,

- for the Young's moduli:

$$E'_v = \left.\frac{d(\sigma_a - \sigma_r)}{d\varepsilon_a}\right|_V , \tag{1}$$

$$E'_h = \left.\frac{d(\sigma_a - \sigma_r)}{d\varepsilon_a}\right|_H ; \tag{2}$$

- for the Poisson's ratios:

$$\nu'_{vh} = \frac{1}{2}\left(1 - \left.\frac{d\varepsilon_v}{d\varepsilon_a}\right|_V\right), \tag{3}$$

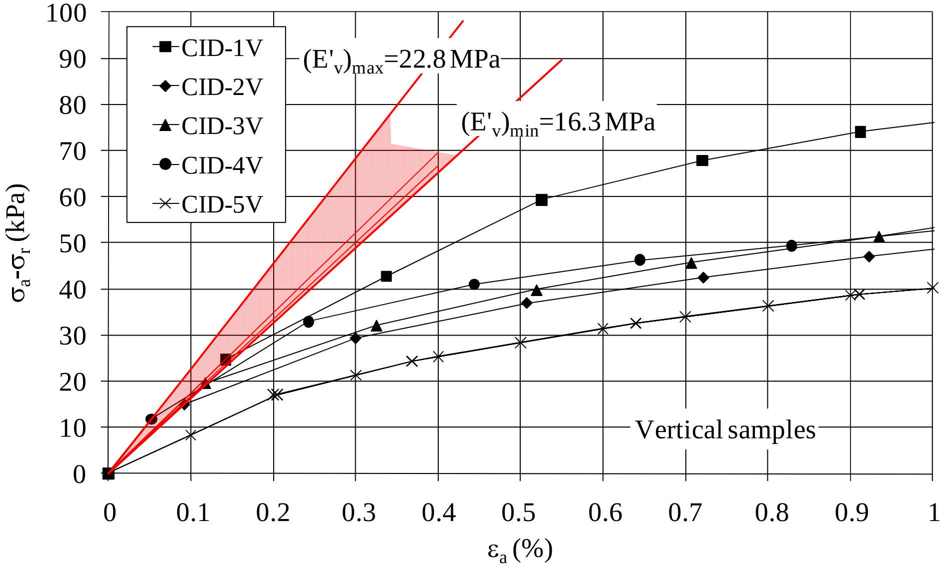
$$\nu'_{hh} = 1 - n\,\nu'_{vh} - \left.\frac{d\varepsilon_v}{d\varepsilon_a}\right|_H , \tag{4}$$

where σ_a and σ_r represent the applied axial and radial stresses, ε_a and ε_v stand for the measured axial and volumetric strains and $n = E'_h / E'_v$ is the degree of anisotropy.

The vertical shear modulus G'_v can be determined by empirical calculation. Its experimental determination requires the shearing of inclined samples. This operation is very delicate and poses significant technological problems so that it cannot be recommended to anyone studying the anisotropy of soils (LO et al. [13]). Certain authors (SAADA and ZAMANI [24], BOEHLER [3]) noted that the tests on inclined samples involve distortions on the planes parallel with the direction of the major principal stress. However, NGUYEN and REIFFSTECK [20] could carry out triaxial tests with bender elements (considered as reliable in the range of the very small deformations) on cut samples with 45° in core samples of clay taken at the experimental site of Cubzac-les-Ponts (France), using a cutting case provided with a system of adapted guidance. The results which they obtained are comparable with those obtained by PIYAL and MAGNAN [23].

4.2.2. EXPERIMENTAL VALUES

Figures 4 and 5 show the shear curves whose values are deduced from the parameters of aelotropy gathered in table 5. These values were determined with the weak deformations because of uncertainty about the measures taken at the beginning of shearing and about their representativeness. However, we can think that the number of the points measured at the beginning of each test is not sufficient to determine the representative values of Young's moduli.



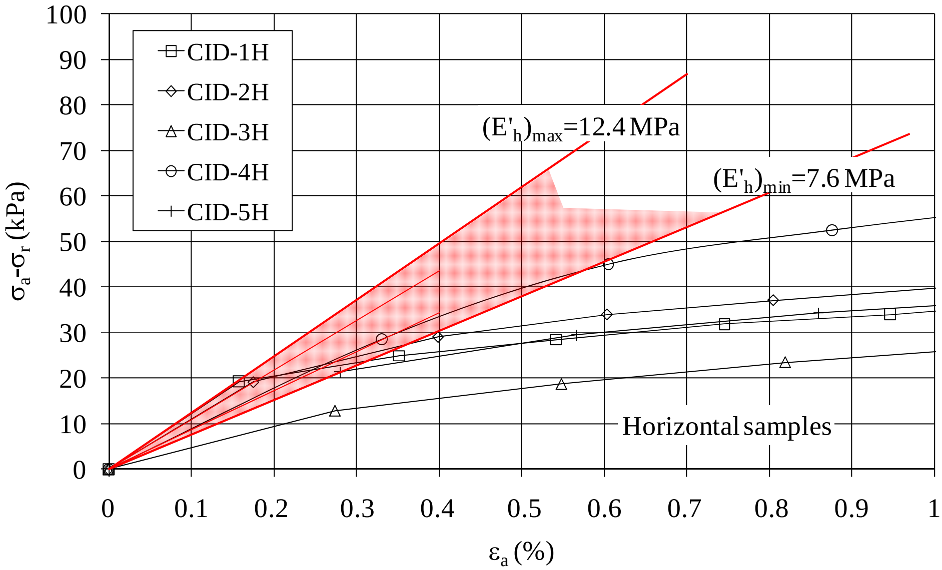


Fig. 4. Determination of the Young's modulus

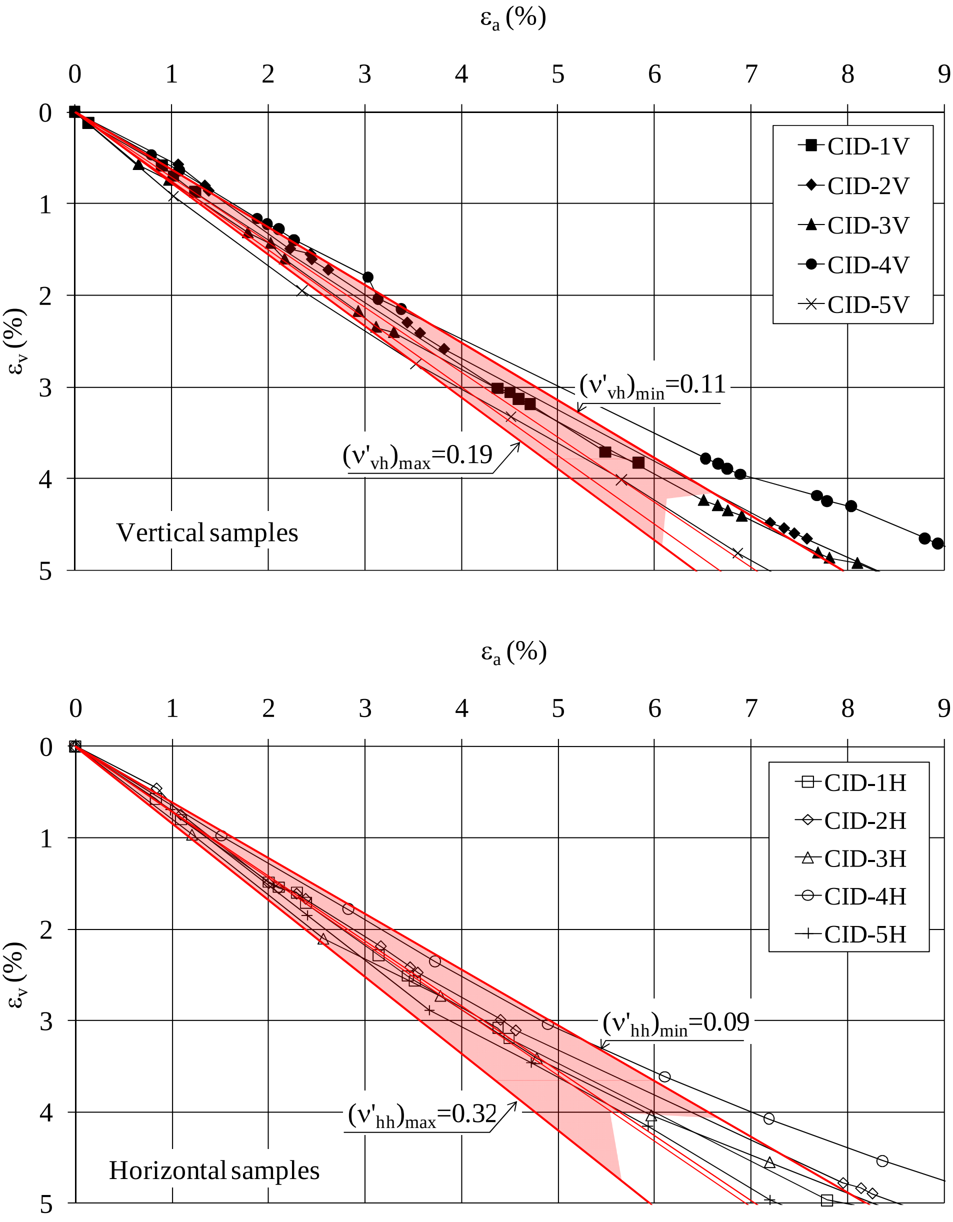


Fig. 5. Determination of the Poisson's ratio

Table 5

Young's modulus and Poisson's ratio values

Vertical samples			Horizontal samples		
Tests	E'_v (MPa)	ν'_{vh}	Tests	E'_h (MPa)	ν'_{hh}
CID-1V	17.4	0.13	CID-1H	12.4	0.21
CID-2V	16.3	0.11	CID-2H	10.9	0.22
CID-3V	16.7	0.15	CID-3H	4.4[a]	0.22
CID-4V	22.8	0.19	CID-4H	8.6	0.32
CID-5V	9.2[a]	0.11	CID-5H	7.6	0.09[a]
Mean values	18.3	0.14	Mean values	9.9	0.24
Degree of anisotropy $n = 9.9/18.3 = 0.54$					
[a] value considered as aberrant					

4.2.3. CONTROL VALUES OF THE DEGREE OF ANISOTROPY

To test the validity of this approach and to check the coherence of the values obtained, we control the degree of anisotropy n determined starting from the measurements taken during the consolidation of samples. We have first calculated for the vertical samples the ratio $[(1-\nu'_{hh})/n]_V$ by means of the following expression (see appendix):

$$\left.\frac{1-\nu'_{hh}}{n}\right|_V = \frac{1}{2}\left[\left(1-2\nu'_{vh}\right)\left.\frac{d\varepsilon_{vc}}{d\varepsilon_{ac}}\right|_V + 4\nu'_{vh} - 1\right], \tag{5}$$

where the ratio $[d\varepsilon_{vc}/d\varepsilon_{ac}]_V$ is given on the curve (ε_{vc}, ε_{ac}) obtained during the consolidation of the vertical samples (figure 6) and the values of ν'_{vh} are given previously by means of equation (3). Then, from equation (4), we deduced the expression of n given as follows (table 6):

$$n = \frac{\left.\dfrac{d\varepsilon_v}{d\varepsilon_a}\right|_H}{\left.\dfrac{1-\nu'_{hh}}{n}\right|_V - \nu'_{vh}}, \tag{6}$$

where the ratio $[d\varepsilon_v/d\varepsilon_a]_H$ represents the initial slope of the curve (ε_v, ε_a), corresponding to shear phase of the horizontal samples (only the tests whose values are not aberrant).

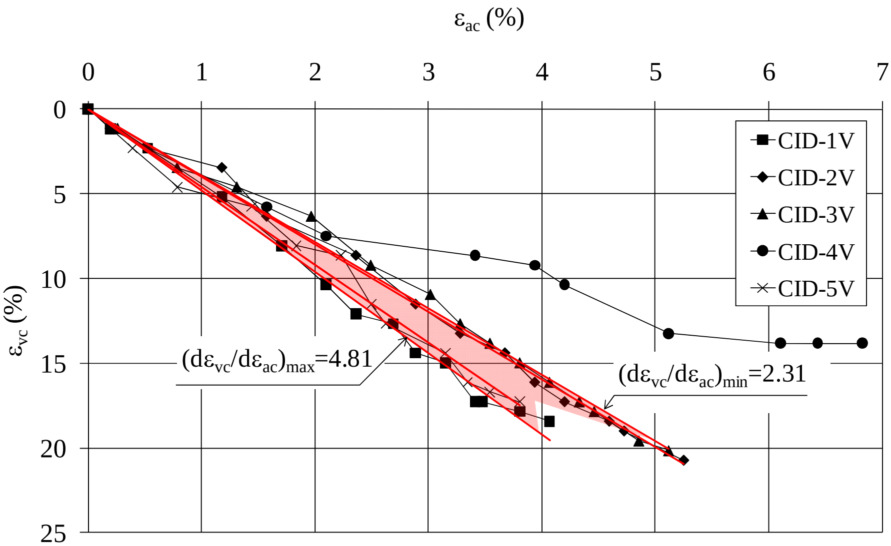


Fig. 6. Evolution of the volumetric strain with the axial strain during the consolidation of vertical samples

Table 6

Degree of anisotropy determined starting from the measurements taken during the consolidation of samples

Tests	ν'_{vh}	$\left.\frac{d\varepsilon_{vc}}{d\varepsilon_{ac}}\right\|_V$	$\left.\frac{1-\nu'_{hh}}{n}\right\|_V$	$\left.\frac{d\varepsilon_v}{d\varepsilon_a}\right\|_H$		
				0.61	0.71	0.84
CID-1V	0.13	4.81	1.55	0.43	0.50	0.59
CID-2V	0.11	3.99	1.28	0.52	0.61	0.72
CID-3V	0.15	3.91	1.18	0.59	0.69	0.82
CID-4V	0.19	2.31	0.60[a]	–	–	–
CID-5V	0.11	4.59	1.51	0.45	0.52	0.61
[a] value considered as aberrant						

The values of $[(1-\nu'_{hh})/n]_V$ are adequately grouped and range between 1.18 and 1.55 (except for the CID-4V sample whose value is considered as aberrant). The values of *n* range between 0.43 and 0.82. They frame well the given value *n* = 0.54 starting from the measurements taken during the shear phase. For clayey deposits, the experiment shows that the degree of anisotropy varies from 0.4 to 1, depending on the state of soil overconsolidation, and that the values of the Poisson's ratios vary apparently cancelling the order of magnitude between 0 and 0.35 in the drained tests.

4.3. TAKING INTO ACCOUNT NONLINEARITIES OF SOIL BEHAVIOUR

To describe the nonlinear behaviour of the soils starting from the standard triaxial test results, KONDNER [10] proposes a hyperbolic relation of the following form:

$$\sigma_a - \sigma_r = \frac{\varepsilon_a}{\dfrac{1}{E_i} + \dfrac{\varepsilon_a}{(\sigma_a - \sigma_r)_{\text{ult}}}}, \tag{7}$$

where E_i represents the initial tangent Young's modulus and $(\sigma_a - \sigma_r)_{\text{ult}}$ is the ultimate deviatoric stress (figure 7).

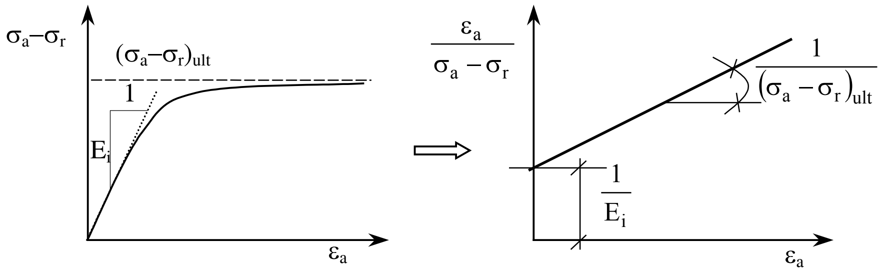


Fig. 7. Representation of the hyperbolic model of KONDNER [10]

Thus, the Young's modulus can be given like the reverse of the ordinate at the line origin obtained by the transformation of the shear curve. Figures 8 and 9 show the shear curves and their transforms for all CID-tests. Table 7 gathers the values of the corresponding Young's moduli.

Table 7

Young's modulus values determined by the hyperbolic approach

Vertical samples		Horizontal samples	
Tests	E'_v (MPa)	Tests	E'_h (MPa)
CID-1V	19.1	CID-1H	8.5
CID-2V	16.0	CID-2H	8.4
CID-3V	16.1	CID-3H	6.3[a]
CID-4V	17.1	CID-4H	8.2
CID-5V	10.2[a]	CID-5H	8.2
Mean value	17.1	Mean value	8.4
Degree of anisotropy n = 8.4/17.1 = 0.49			
[a] value considered as aberrant			

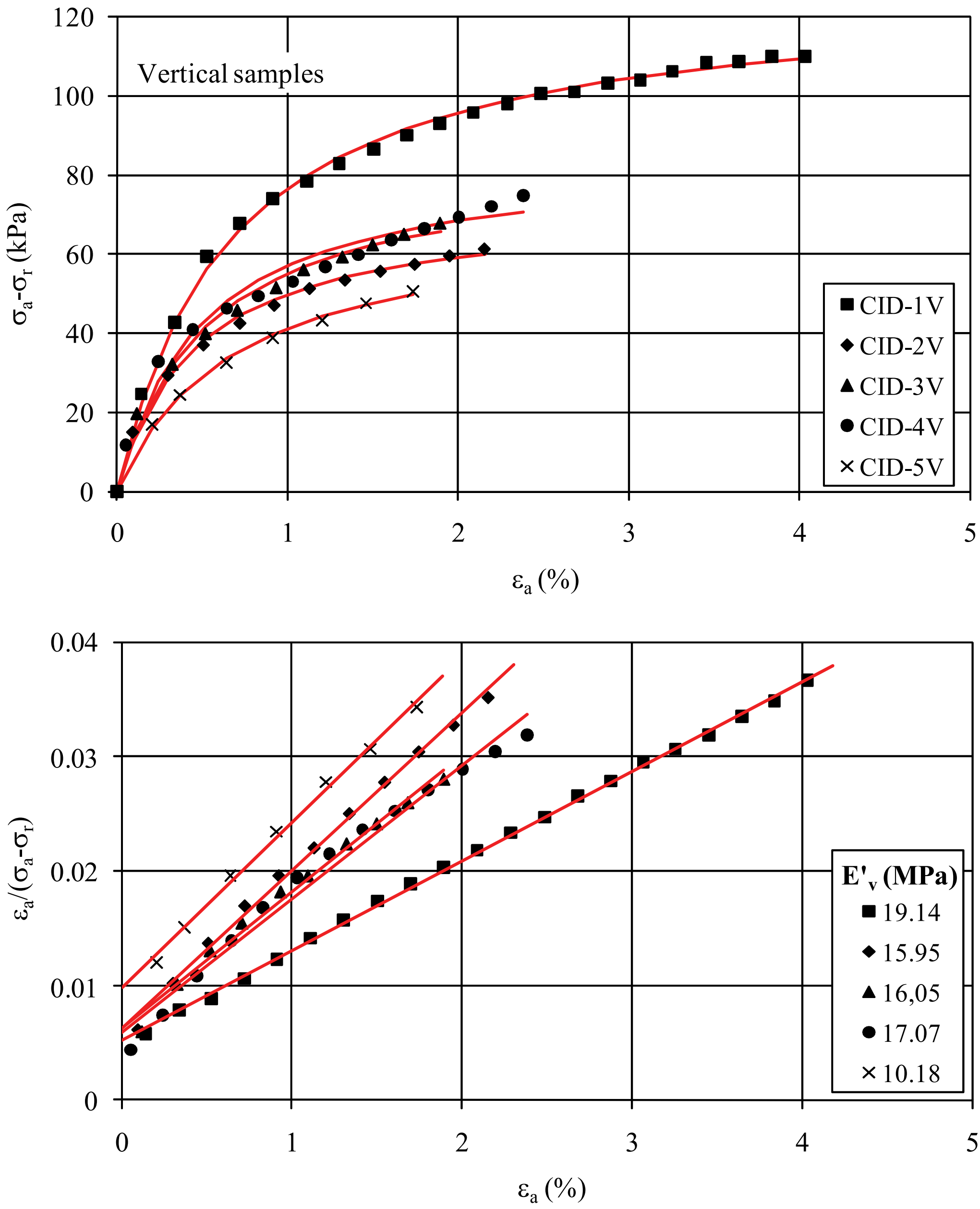


Fig. 8. Determination of the vertical Young's moduli by the hyperbolic approach

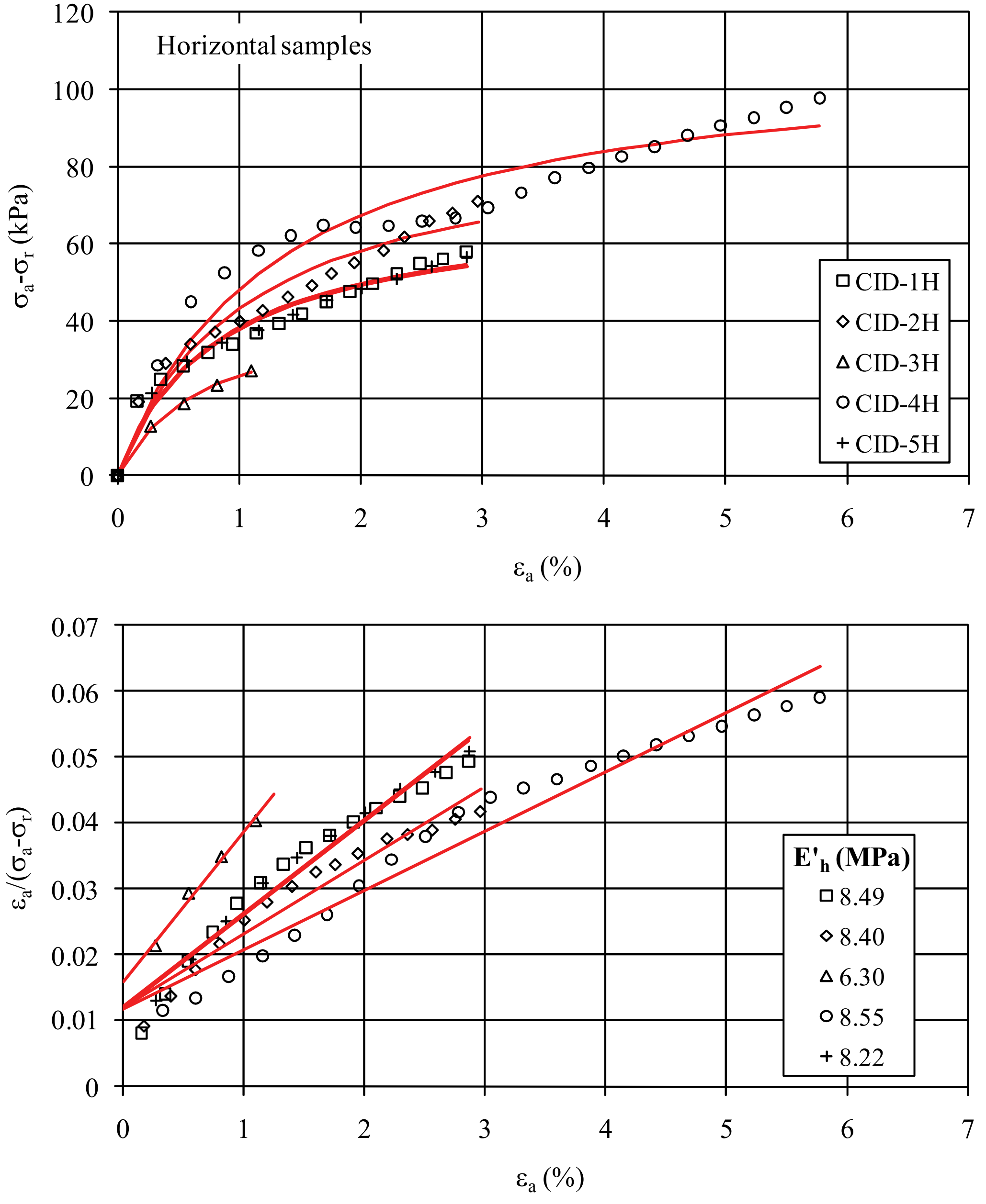


Fig. 9. Determination of the horizontal Young's moduli by the hyperbolic approach

4.4. CALCULATION OF THE VERTICAL SHEAR MODULUS

Table 8 gives the values of the vertical shear modulus G'_v calculated starting from some empirical expressions suggested by certain authors cited by MEFTAH [18], according to the parameters determined previously.

Table 8

Computed values of the vertical shear modulus starting from some empirical expressions

Authors	Expressions of G'_v	Values of G'_v (MPa)
WOLF (1945)	$\dfrac{nE'_v}{1+n+2\nu'_{vh}}$	4.7
CARRIER (1946)	$\dfrac{E'_v}{1-n\nu'^2_{vh}+2\nu'_{vh}(1+\nu'_{hh})+\dfrac{1-\nu'^2_{hh}}{n}}$	5.3
BARDEN (1963)	$\dfrac{nE'_v}{1+n+2n\nu'_{vh}}$	5.2
WIENDIECK (1964)	$\dfrac{nE'_v}{1+\sqrt{n}+2n\nu'_{vh}}$	4.6
ZIENKIEWICZ (1972)	$\dfrac{(1-n\nu'^2_{vh})(1-\nu'_{hh})E'_v}{(1-\nu'_{hh}-2n\nu'^2_{vh})\left(1-n\nu'^2_{vh}+\dfrac{1-\nu'^2_{hh}}{n}\right)}$	6.0
GARNIER (1973)	$\dfrac{nE'_v}{\sqrt{8(1+n^2)}-1-n-2n\nu'_{vh}}$	5.5
Mean value		5.2

4.5. DISCUSSION OF THE RESULTS

The experimental values of the Young's moduli and the Poisson's ratios recapitulated in table 5 are very dispersed in the two vertical and horizontal directions. The relatively low values of certain parameters and the corresponding significant deformations before the rupture allow a certain disturbance of the samples to be expected; they are considered as aberrant and are not taken into account. While, the Young's modulus values recapitulated in table 7 are not very different from one sample to another. These values prove that the hyperbolic approach is probably adapted better than the linear elastic approach and undoubtedly more relevant for the analysis of the structures founded on anisotropic soils.

The computed values of the vertical shear modulus seem to be reasonable. But, this observation cannot constitute the proof of the quality of the empirical approach used for its determination. However, the values of the Poisson's ratios do not violate the criterion of positive potential energy. This inclined us to think that the drained triaxial shear tests make it possible to obtain coherent values of the parameters of aelotropy of normally consolidated soft clays.

Thus, taking into account the experimental conditions and those of the soil itself (the effect of the depth on the state of core samples), we can think that these values are, on average, representative of the aelotropic behaviour of the Guiche soft clay in place. Table 9 gives the range of variation and the mean values of the parameters of aelotropy characterizing this natural clay.

Table 9

Parameters of aelotropy characterizing the Guiche clay (the Adour Valley, France)

Parameters	E'_v (MPa)	E'_h (MPa)	ν'_{vh}	ν'_{hh}	G'_v (MPa)
Range of variation	16–19.1	8.2–8.6	0.11–0.19	0.21–0.32	4.6–6
Mean values	17.1	8.4	0.14	0.24	5.2

5. CONCLUSIONS

The interpretation of the results of the undrained triaxial shear test in compression and extension, carried out on the samples reconsolidated to the effective stresses in place, made it possible to highlight the anisotropy of the mechanical properties of the resistance and deformability of the Guiche soft clay (the Adour Valley, France). Indeed, this clay has different undrained shear strengths according to vertical and horizontal directions and develops deformations according to these directions also different for the same state of loading.

The exploitation of the results of the drained triaxial shear test carried out on vertical and horizontal samples reconsolidated in an isotropic way with the effective mean stress of the soils in place made it possible to determine the parameters of aelotropy of this natural clay according to the traditional formalism of linear (Hooke's law) and nonlinear (the elasticity of hyperbolic type) elasticity.

This experimental approach allows us to conclude that it is possible, at the cost of some uncertainties, to obtain representative values by means of drained shear tests with the standard triaxial apparatus. However, it is important to note that the determination of the parameters of aelotropy of soils is a very delicate operation and it is not unlikely that the data obtained represent only the part of their behaviour.

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APPENDIX

RELATIONS BETWEEN THE PARAMETERS OF AELOTROPY OF A SOIL AND THE SIZES MEASURED DURING A DRAINED TRIAXIAL COMPRESSION TEST

GENERAL STRESS–STRAIN RELATIONS

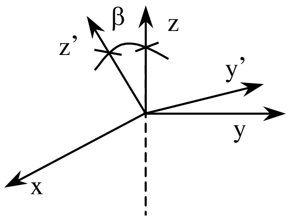
LEKHNITSKII [11] established the following stress–strain relations for an anisotropic elastic body according to the five independent parameters E'_v, E'_h, G'_v, ν'_{vh} and ν'_{hh}:

$$\varepsilon_x = \frac{1}{E'_h}\sigma'_x - \frac{1}{E'_h}(\nu'_{hh}\cos^2\beta + n\nu'_{vh}\sin^2\beta)\sigma'_y - \frac{1}{E'_h}(\nu'_{hh}\sin^2\beta + n\nu'_{vh}\cos^2\beta)\sigma'_z,$$

$$\begin{aligned}\varepsilon_y = &-\frac{1}{E'_h}(\nu'_{hh}\cos^2\beta + n\nu'_{vh}\sin^2\beta)\sigma'_x \\ &+\left[\frac{1}{E'_h}(\cos^4\beta + n\sin^4\beta - 2n\nu'_{vh}\cos^2\beta\sin^2\beta) + \frac{1}{G'_v}\cos^2\beta\sin^2\beta\right]\sigma'_y \\ &+\left[-\frac{\nu'_{vh}}{E'_v}(\cos^4\beta + \sin^4\beta) + \left(\frac{1}{E'_h} + \frac{1}{E'_v} - \frac{1}{G'_v}\right)\cos^2\beta\sin^2\beta\right]\sigma'_z,\end{aligned}$$

$$\begin{aligned}\varepsilon_z = &-\frac{1}{E'_h}(\nu'_{hh}\sin^2\beta + n\nu'_{vh}\cos^2\beta)\sigma'_x \\ &+\left[-\frac{\nu'_{vh}}{E'_v}(\cos^4\beta + \sin^4\beta) + \left(\frac{1}{E'_h} + \frac{1}{E'_v} - \frac{1}{G'_v}\right)\cos^2\beta\sin^2\beta\right]\sigma'_y \\ &+\left[\frac{1}{E'_h}(\sin^4\beta + n\cos^4\beta) - \frac{2\nu'_{vh}}{E'_v}\cos^2\beta\sin^2\beta + \frac{1}{G'_v}\cos^2\beta\sin^2\beta\right]\sigma'_z,\end{aligned}$$

where σ'_x, σ'_y and σ'_z represent the effective normal stresses applied and ε_x, ε_y and ε_z are the corresponding strains, $n = E'_h / E'_v$ stands for the degree of anisotropy and β is the angle which the z-axis of orthotropism forms with the reference z'-axis (figure).



Axis of orthotropism

APPLICATION TO AN ISOTROPIC TRIAXIAL CONSOLIDATION PHASE

Isotropic triaxial consolidation phase is characterized by: $d\sigma'_x = d\sigma'_y = d\sigma'_z = d\sigma'_c$, and the stress–strain relations become:

- For a vertical sample ($\beta = 0$):

$$d\varepsilon_x = \left(\frac{1-\nu'_{hh}}{n} - \nu'_{vh}\right)\frac{d\sigma'_c}{E'_v},$$

$$d\varepsilon_y = \left(\frac{1-\nu'_{hh}}{n} - \nu'_{vh}\right)\frac{d\sigma'_c}{E'_v},$$

$$d\varepsilon_z = (1 - 2\nu'_{vh})\frac{d\sigma'_c}{E'_v},$$

so that the volumetric strain is equal to:

$$d\varepsilon_v = d\varepsilon_x + d\varepsilon_y + d\varepsilon_z = \left[1 - 4\nu'_{vh} + \frac{2(1-\nu'_{hh})}{n}\right]\frac{d\sigma'_c}{E'_v}$$

and we can deduce the following relation:

$$\left.\frac{1-\nu'_{hh}}{n}\right|_V = \frac{1}{2}\left[(1-2\nu'_{vh})\left.\frac{d\varepsilon_{vc}}{d\varepsilon_{ac}}\right|_V + 4\nu'_{vh} - 1\right], \qquad \text{(A1)}$$

in which

$$\left.\frac{d\varepsilon_{vc}}{d\varepsilon_{ac}}\right|_V$$

represents the initial slope of the curve (ε_{vc}, ε_{ac}).

- For a horizontal sample ($\beta = \pi/2$):

$$d\varepsilon_x = \left(\frac{1-\nu'_{hh}}{n} - \nu'_{vh}\right)\frac{d\sigma'_c}{E'_v},$$

$$d\varepsilon_y = (1-2\nu'_{vh})\frac{d\sigma'_c}{E'_v},$$

$$d\varepsilon_z = \left(\frac{1-\nu'_{hh}}{n} - \nu'_{vh}\right)\frac{d\sigma'_c}{E'_v},$$

so that the volumetric strain is equal to:

$$d\varepsilon_v = d\varepsilon_x + d\varepsilon_y + d\varepsilon_z = \left[1 - 4\nu'_{vh} + \frac{2(1-\nu'_{hh})}{n}\right]\frac{d\sigma'_c}{E'_v}$$

and we can deduce the following relation:

$$\left.\frac{1-\nu'_{hh}}{n}\right|_H = \frac{1+\nu'_{vh}\left(\left.\frac{d\varepsilon_{vc}}{d\varepsilon_{ac}}\right|_H - 4\right)}{\left.\frac{d\varepsilon_{vc}}{d\varepsilon_{ac}}\right|_H - 2} \tag{A2}$$

in which $\left.\frac{d\varepsilon_{vc}}{d\varepsilon_{ac}}\right|_H$ represents the initial slope of the curve (ε_{vc}, ε_{ac}).

APPLICATION TO A DRAINED TRIAXIAL SHEAR PHASE

Drained triaxial shear phase is characterized by: $d\sigma'_x = d\sigma'_y = 0$ and $d\sigma'_z \neq 0$, and the stress–strain relations become:

- For a vertical sample ($\beta = 0$):

$$d\varepsilon_x = -\frac{\nu'_{vh}}{E'_v}d\sigma'_z,$$

$$d\varepsilon_y = -\frac{\nu'_{vh}}{E'_v} d\sigma'_z ,$$

$$d\varepsilon_z = \frac{1}{E'_v} d\sigma'_z ,$$

so that the volumetric strain is equal to:

$$d\varepsilon_v = d\varepsilon_x + d\varepsilon_y + d\varepsilon_z = \left(\frac{1-2\nu'_{vh}}{E'_v} \right) d\sigma'_z$$

and we can deduce the two following relations:

$$E'_v = \left. \frac{d(\sigma_a - \sigma_r)}{d\varepsilon_a} \right|_V , \tag{A3}$$

$$\nu'_{vh} = \frac{1}{2} \left(1 - \left. \frac{d\varepsilon_v}{d\varepsilon_a} \right|_V \right), \tag{A4}$$

in which $\left. \frac{d(\sigma_a - \sigma_r)}{d\varepsilon_a} \right|_V$ and $\left. \frac{d\varepsilon_v}{d\varepsilon_a} \right|_V$ represent the initial slopes of the curves ($\sigma_a - \sigma_r$, ε_a) and (ε_v, ε_a), respectively.

- For a horizontal sample ($\beta = \pi /2$):

$$d\varepsilon_x = -\frac{\nu'_{hh}}{E'_h} d\sigma'_z ,$$

$$d\varepsilon_y = -\frac{n\nu'_{vh}}{E'_h} d\sigma'_z ,$$

$$d\varepsilon_z = \frac{1}{E'_h} d\sigma'_z ,$$

so that the volumetric strain is equal to:

$$d\varepsilon_v = d\varepsilon_x + d\varepsilon_y + d\varepsilon_z = \left(\frac{1-\nu'_{hh} - n\nu'_{vh}}{E'_h} \right) d\sigma'_z$$

and we can deduce the two following relations:

$$E'_h = \left.\frac{d(\sigma_a - \sigma_r)}{d\varepsilon_a}\right|_H , \tag{A5}$$

$$\nu'_{hh} = 1 - n\nu'_{vh} - \left.\frac{d\varepsilon_v}{d\varepsilon_a}\right|_H , \tag{A6}$$

in which $\left.\frac{d(\sigma_a - \sigma_r)}{d\varepsilon_a}\right|_H$ and $\left.\frac{d\varepsilon_v}{d\varepsilon_a}\right|_H$

represent the initial slopes of the curves ($\sigma_a - \sigma_r$, ε_a) and (ε_v, ε_a), respectively.