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NUMERIC ALANALYSIS OF MONOTONIC AND CYCLIC PRESSUREMETER TEST AND IDENTIFYING SOIL BEHAVIOUR PARAMETERS

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ABSTRACT:

The use of the pressuremeter test to determine soil parameters has been the subject of several empirical, analytical, and numerical studies. These studies are based on the parameters taken directly from the conventional pressuremeter curve, which are often affected by the boring quality, the soil remoulding, and the induced disturbances during installation. So, the reflection to integrate an unloading-reloading portion during the test becomes necessary. The present work deals with a numerical analysis of the cyclic pressuremeter test using the FLAC 2D calculation code and taking into account the Soft Soil Hardening model (SS). An identification of soil parameters using the conventional pressuremeter curve is performed, focusing on two parameters, the shear modulus and the undrained cohesion. The obtained parameters are then used to analyze the cyclic pressuremeter test. A comparison between experimental and numerical results shows that the cyclic pressuremeter test can be another means of identifying of soil parameters and allows gaining valuable information on the cyclic behaviour of the soil.

1. INTRODUCTION

The design of foundations is generally carried out using conventional methods or using numerical calculation codes. The use of these numerical methods has evolved considerably through the development of computers. However, these methods encounter a major problem, which is the characterization of the soil behaviour parameter values of the chosen model.

Generally, these soil parameter values are determined from laboratory tests on small soil samples (triaxial or oedometric tests) or from in situ tests (pressuremeter penetrometer tests). The determination of these parameters from laboratory tests is confronted with two major problems, which are the representativeness of the appropriate soil samples and the remoulding induced by the boring and extraction procedures. Whereas in situ tests are not subjected to the soil remoulding problem, parameters are determined by empirical or analytical relationships based on simplifying assumptions using the experimental curves (Ménard 1957; Gibson and Anderson 1961; Baguelin et al. 1972; Monnet 1995). Hence, this identification becomes a delicate task for the engineer and requires a lot of experience.

To limit these estimation difficulties, several authors have proposed the inverse analysis method of the pressuremeter test. This method consists of minimizing the difference between a numerical and an experimental curve, which corresponds to an optimization problem (Bahar 1992; Zentar 1999; Levasseur 2008).

Another problem that engineers meet in structure calculations is the choice of the deformation module value, which is often taken equal to the pressuremeter module. This module is often affected by the soil remoulding around the probe and the quality of boring (Cassan 1978; Briaud 1992; Combarieu et al. 2001), so it cannot be considered directly (as a Young's module), which led to thinking for a pressuremeter test with an unloading portion. Indeed, this principle is not recent, because Louis Menard introduced this procedure in 1962, where an unloading-reloading during the probe dilation test is carried out. A deformation module is determined on the unloading-reloading portion and is called the alternate module Ea. However, he suggested using this module to study the soil behaviour under vibrating machines (Ménard and Lambert 1966). However, despite the importance of this unloading-reloading portion, the use of cyclic modules in the calculations as conventional modules is not yet regulated.

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Therefore, this work deals with the numerical identification of shear module and undrained cohesion from the pressuremeter test by the inverse method, using the code FLAC2D and taking into account the SS elastoplastic model. Afterwards, the results obtained will be used to model the cyclic test, which will allow to make comparisons with cyclic tests already carried out and to have more information on the pressuremeter test with the unloading-reloading loop.

2. NUMERICAL ANALYSIS OF PRESSUREMETER TEST

2.1 The formulation of the problem

All the studies of the pressuremeter test are made considering the theory of expansion of a cylindrical cavity in an elastoplastic infinite medium (Gibson et al. 1961; Bahar 1992; Monnet 2007). Therefore, the expansion of pressuremeter probe is assumed to occur in plane strain and in axisymmetric conditions (Figure 1). Taking into account the assumption of plane strain and the axisymmetric, the equilibrium equation governing the pressuremeter loading is given in the radial direction and is written as:

$$\frac{\partial \sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \qquad (1)$$

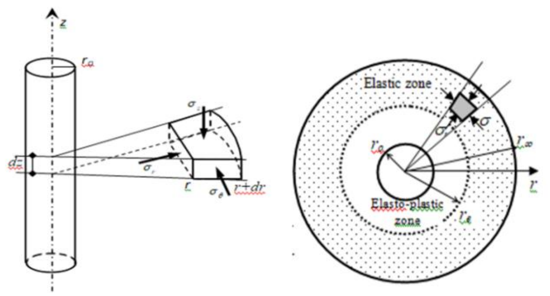


Figure 1. Equilibrium of a soil element around the pressuremeter probe

2.2 Modeling of the pressuremeter test

The simplicity of boundary conditions that govern the pressuremeter test makes it easy for its numerical modeling. So, the plane strain hypothesis in the vertical direction and the axisymmetric conditions are considered (Zanier 1985). The boundary conditions can be specified either in displacements or in stresses and given as follows (Bahar 1992; Zanier 1992).

Along the wall cavity ($r = r_o$): $\Delta U_r = \Delta u_o$ *or* $\Delta\sigma_r = \Delta p_o$
At an infinite distance ($r = r_\infty$): $\Delta U_r = 0$ *and* $\Delta\sigma_r = 0$

Where: r_o is the initial borehole radius and u_o is the radial displacement at the cavity wall.

The pressuremeter test modeling must take account of several factors, which make it possible to obtain a good soil response, namely, the discretization of the medium, the boundary conditions, the slenderness of the probe, and the behaviour law to consider. In this context, several numerical studies of the pressuremeter test were carried out, analysing the effect of these factors (Wachcowski 1976; Zanier 1985; Zentar 1999; Soegiri 1991).

Analyses are performed using the finite difference FLAC^{2D} program, which integrates various constitutive laws to simulate soil behaviour. The strain hardening/softening (SS) model is considered. This model is based on FLAC Mohr-Coulomb model with non-associated shear and associated tension flow rules [Itasca]. However, the strain hardening/softening (SS) model allows representation of nonlinear material softening and hardening behaviour based on variation of the model parameters (cohesion, friction, dilation, and tensile strength) as functions of the deviatoric plastic strain (Figure 2).

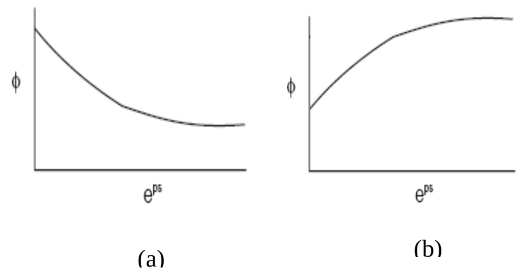


Figure 2. Variation of the friction as a function of plastic strain: a) softening behaviour, b) hardening behaviour (ITASCA)

The failure envelope is given by the Mohr-Coulomb yield function (formula 3). Plastic shear strain is measured by the shear hardening parameter e^{ps}, whose incremental form is defined (Vermeer and de Borst 1984; ITASCA 2008).

$$\boldsymbol{f^s = \sigma_1 - \sigma_3\sqrt{N_\varphi} + 2C\sqrt{N_\varphi}} \,. \qquad (2)$$

Where: $N_\varphi = \frac{1+sin\varphi}{1-sin\varphi}$(3)

φ: friction angle ,

C: Cohesion.

$$\Delta e^{ps} = \left\{\frac{1}{2}(\Delta e_1^{ps} - \Delta e_m^{ps})^2 + \frac{1}{2}(\Delta e_m^{ps})^2 + \frac{1}{2}(\Delta e_3^{ps} - \Delta e_m^{ps})^2\right\}^{\frac{1}{2}} \qquad (4)$$

Where:

$\Delta e_m^{ps} = \frac{1}{3}\left(\Delta e_1^{ps} + \Delta e_3^{ps}\right)$, and Δe_j^{ps}, j=1,3 are the principal plastic shear strain increments.

The meshgrid was chosen according to previous studies results; a grid of 4-meter length is considered and will be descretized into rectangular elements. The meshgrid must be refined in the vicinity of the probe (due to the application of the load). The refinement of elements follows a geometric progression with a ratio of 1.25; the formula is given by:

$$r_n = r_{n-1} + \Delta r_n$$

Where: $\Delta r_n = 1.25 \times r_{n-1}$

The grid comprises 30 elements, whose thickness of the first mesh is 1 mm (Bahar 1992; Zanier 1992) (Figure 3).

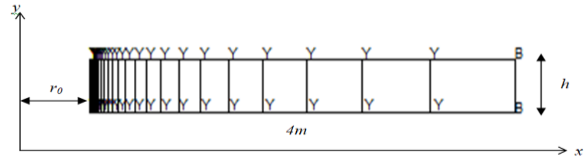


Figure 3. Mesh and boundary conditions

3. IDENTIFICATION OF SOIL PARAMETERS FROM PRESSUREMETER TEST

It is known that pressuremeter tests do not allow the direct identification of the constitutive parameters of soils; hence the methods of interpretation of the pressuremeter results are numerous. The exploitation of the pressuremeter results is based on the two parameters: the limit pressure (P_l) and the pressuremeter module (E_M when it is a pre-boring test, G when it is the self-boring test), which are used in the calculation of foundations and for the evaluation of the strength parameters either by using semi-empirical methods (Ménard 1957; Amar and Jézéquel 1972; Cassan 1978) or by theoretical methods based on the expansion of a cylindrical cavity in an elastoplastic medium study (Baguelin et al. 1972; Gibson et al.1961; Monnet et al. 1995).

Another type of method, allowing the identification of soil parameters using the pressuremeter curve directly, has been established by several researchers; it is the inverse method. The principle of this method consists in minimizing the difference between the numerical curve and the experimental curve by iterative modifications of the input values until the output values reproduce the experimental curve as best as possible (Abed 2013; Zentar 1999; Levasseur 2007).

In this context, several studies have used the inverse method to identify the soil parameters (Bahar 1992; Zentar 2001; Levasseur 2007) using different numerical programs for the modeling of the pressuremeter test and different optimization algorithms for identification.

$$Cu=\frac{Pl-P0}{5,5} \quad \text{(Ménard)} \qquad (5)$$

$$Cu=\frac{Pl-P0}{10}+25(kPa) \quad \text{(Amar et Jezequel)} \qquad (6)$$

$$G=\frac{E}{2\,(1+v)} \qquad (7)$$

Where: $E = E_M = 2(1+v)(V_0+V_m)\frac{dP}{dV}$ (8)

v : Poisson ratio.

V_0 : Initial volume of the probe at rest.

V_m : Volume change corresponding to average pressure in the pseudo-elastic phase.

$\frac{dP}{dV}$: The inverse of the slope of the linear portion of the pressuremeter curve.

3.1 Procedure of identification

In this study, the method used for the identification is based on the principle of decomposition of the pressuremeter curve established by Boubanga (Abed 2013; Boubanga 1990). The curve is divided into three parts; the first part corresponds to the small strains' domain, the second characterizes the volume variation, and the third is the large strains domain (Figure 4). Shear modulus G is determined in the small strains part; the interpolation considered is linear and has the following form:

$$G^n=G^{(n-1)}\, x \frac{S_e}{S_c^{(n-1)}} \qquad (9)$$

Where: G is the Shear module, Se and $S_c^{(n-1)}$ are the surfaces generated by the experimental and simulated curves respectively, in the small strain domain.

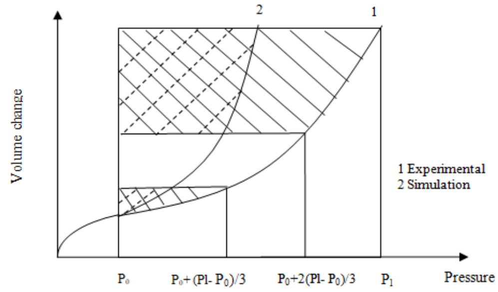


Figure 4. Method of decomposition of the pressuremeter curve

For the parameters of the large strains, the evolution law has the following form: (C or φ)

$$C^n=C^{(n-1)}+(S_e- S_c^{(n-1)})xC^{(n-1)}- C^{(n-2)} / S_c^{(n-2)}-S_c^{(n-1)} \qquad (10)$$

Where: C^{n-1} : Initial value of cohesion.

$C^{(n-2)}$: Value taken arbitrarily.

$S_c^{(n-2)}$and$S_c^{(n-1)}$: Surfaces generated by the simulated curves corresponding to $C^{(n-1)}$ and $C^{(n-2)}$ respectively in the large strains area.

3.2 A sensitivity study

In order to know the influence of the model parameters on the numerical response of the monotonic and cyclic pressuremeter path, a sensitivity study was carried out by using FLAC software taking into account the strain hardening/softening model. This model involves five parameters (Cohesion, friction, compressibility module, shear module, and dilatation).This study was carried out by varying the initial value of each parameter. The reference parameters used in this study are the Cran clay parameters (Table 1).

Depth (m)	γ_d (KN/m^3)	E (MPa)	C_u (bar)	φ' (°)	C' (bar)	Cc
1.0-2.2	11.0	13	0.45			0.45
2.2 - 4.0	6.6	29	0.15	30	0.0	1.64
4.0 - 8.0	9.3	18	0.40			0.70
8.0 - 17.0	9.1	17	0.39	34	0/0	0.85

Table 1. Results of laboratory tests at Cran site (Soegiri 1991)

The results obtained (Figures 5, 6, and 7), show that all three parameters have an influence on the pressuremeter response.

It is observed that the increase of cohesion and internal friction increases the limit pressure without any change in the slope of the initial part, but a significant influence on the reversible deformation is noticed, where the volume of the cyclic deformations is smaller when the cohesion and the friction are larger than when cohesion and friction angle are larger.

For the shear modulus, it is observed that the increase of G increases the limit pressure, but the cyclic deformations remain constant for the three values of G; on the other hand, an influence in the slopes of cycles is noticed (Bahar and Belhassani 2012).

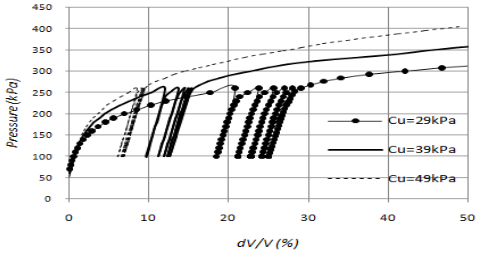


Figure 5. Influence of undrained cohesion on the cyclic curve

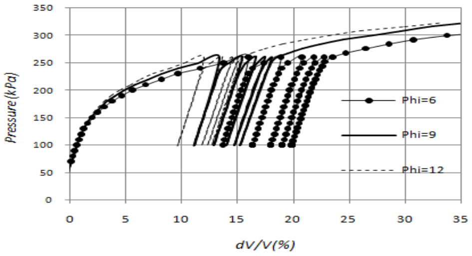


Figure 6. Influence of friction angle on the cyclic curve

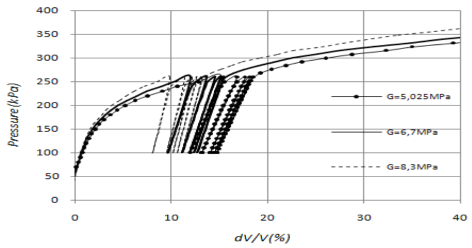


Figure 7. Influence of shear modulus on the cyclic curve

3.3 Comparison between an axisymmetric problem for a soil thickness unit and bidimensional mesh

In this study, two different meshgrids are considered to model the pressuremeter test. The first is the one presented above (Figure 3), it is defined by a soil thickness unit in the vertical plane, and the second one is bidimensional (Figure 8) with elements in both vertical and horizontal planes. The results given by the Figure 9 show that the two meshgrids produce the same pressuremeter curve. Which justify the choice of the meshgrid with a thickness unit in this study; also, it will be of great interest, and the results of pressuremeter modeling will be obtained much more rapidly.

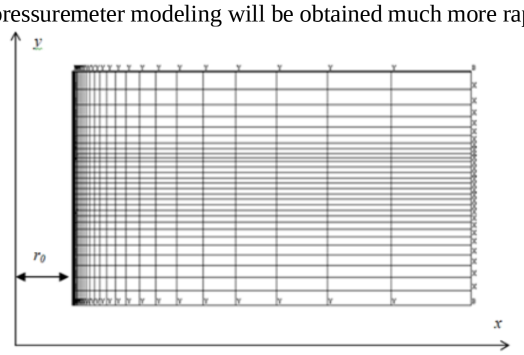


Figure 8. Model discretization with axisymmetric conditions in vertical plane

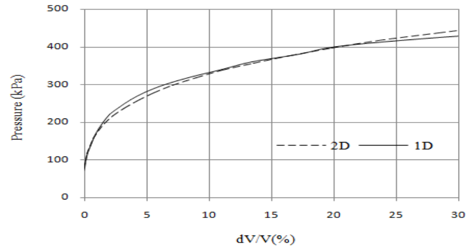


Figure 9. Comparison between an axisymmetric problem for a soil thickness unit and bidimensionnal mesh (Bahar and Belhassani 2012)

3.4 Comparison between the Pre-bored and the Self-boring pressuremeters

In the preboring pressuremeter test, the borehole is made before the introduction of the probe, wich causes a stress release. So the boring wall is unloading until it reaches the probe, then reloading, after the probe inflation. However, in the self-boring pressuremeter, the borehole is carried out by the probe itself, then the remoulding is minimal, thus allowing loading close to the initial state.

To compare these two pressuremeter paths, a numerical simulation of the PMT and SBP tests is carried out by considering the Cran clay parārameters (Table 1).

In the first simulation, an unloading from the initial state (P_0) to 0.10 P_0 and a reloading until limit pressure P_l has been considered. Whereas in the second, it was a loading from the initial state P_0 until the limit pressure P_l.

The results of these simulations (figure 10) show the difference between these two loading paths, in particular at the beginning of the curves, where usual moduli were calculated (Table 2).

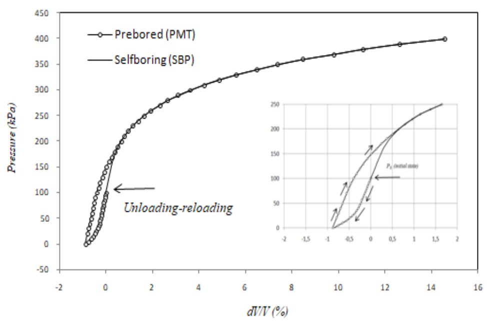


Figure10. Numerical comparison between the PMT and SBP tests

PMT	SBP	E_{SBP}/E_{PMT}
E_p=11400 KPa	E_s(2%)=11750(kPa)	1.030
	E_s(5%)=5800(kPa)	0.50
p_L=380 KPa	p_L=380(kPa)	

Table 2. Pressuremeter characteristics obtained from numerical simulation of PMT and SBP

The comparison between the moduli shows that the moduli obtained from SBP are higher than those obtained from PMT, which agrees with experimental observations (Baguelin et al. 1972). These results obtained by numerical simulations are similar to experimental results.

4. APPLICATION OF THE IDENTIFICATION PROCEDURE ON DIFFERENT SITES

Several pressuremeter tests will be subjected to the identifying procedure already mentioned above. These pressuremeter results relate to different sites; the first site is located in the center of Algiers, where will be realized the tunnel of the Algiers metro, the second is the Cran site, where several monotonic and cyclic pressuremeter tests were carried out; and the third is the Saint-Herblain, where the pressuremeter test results were exploited by R. Zentar (2001) in order to identify these behavior parameters. All these sites are clay soils. The identification procedure is carried out on the shear modulus G and the undrained cohesion Cu. In order to identify these parameters, a first numerical simulation of each test was carried out with an initial parameter set, these initial parameters are determined from the pressuremeter test results, where the shear modulus is calculated by the formula (8) when it is the pre-boring test or determined on the slope of the initial portion of the pressuremeter curve when it is the self-boring test, while the undrained cohesion values are calculated using the empirical relationships of Menard or Amar and Jézéquel (formulas 5 and 6).

4.1 Pressuremeter test results on the Algiers site

Several Menard pressuremeter tests were performed by the LNHC laboratory at the tunnel site, which houses the Algiers metro, at depths from 8 to 17 meters. The pressuremeter results given in the raw state have been corrected, thus allowing the determination of the limit pressure, the pressuremeter modulus, and the creep pressure for all the different depths (Table 3).

Depth [m]	Pl [bars]	E_M [bars]	G [bars]
8	6,809	31,61	11,88
9	7.0635	66,41	24,966
10	9,14062	70,83	26,6278
11	2,98676	5,5	2,090
12	3,6797	9,59	3,6052
13	11,390	54,05	20,319
14	14,473	201,62	75,796
15	24,806	374,59	140,82
16	25,0984	435,38	163,67
17	25 ,273	534,47	200,928

Table 3. Pressuremeter test results at Algiers site

4.2 Pressuremeter test results on Cran clay

Cran clay has been the subject of several laboratory and in situ tests, especially the self-boring pressuremeter. The site is located in Redon (France), it is a clayey-silty soil, slightly over-consolidated and very plastic. The pressurermeter tests are performed with unloading beyond the limit pressure (20% of strains). The results obtained are, the limit pressure which corresponds to 20% strains, the initial pressure, the shear modulus G_0 determined from the initial part of the curve, the shear modules G_{p2}, G_{p5} corresponding respectively to 2% and 5% of strains; and the results summarized in Table 4 will serve as initial parameters.

Depth [m]	G_{id} [MPa]	G_{p2} [MPa]	G_{p5} [MPa]	G_0 [MPa]
2,3	1,13	0,771	0,636	1,163
3,8	0,862	0,6191	0,485	1,216
5	1,155	0,920	0,871	0,59
7	3,95	3,214	2,294	0,101
8,9	2,55	2,152	1,678	0,806
10	4,9	4,877	3,485	3,637
12,6	3,2	3,298	2,122	2,7

Table 4. Pressuremeter test results on the Cran clay

4.3 Determination of the pressuremeter test results of the Saint-Herblain clay

The site is located in the Patissiere valley in the west of Nantes (France), essentially made up of clayey layers, very plastic and over-consolidated on the surface, moderately consistent in depth, which is deposited on a substratum (20 m). The ground water (water table) varies between 0.5 and 1m.

This site has been the subject of several laboratories and in situ tests, which resulted in several research studies. Zentar has identified the behavior parameters of this clay from pressuremeter test using the inverse method, which is based on the gradient method. Two pressuremeter tests have been reproduced, the test at 7 m and 5 m depth, these tests are used to identify the shear modulus and undrained cohesion by using the method already mentioned, the initial parameters are determined from the experimental results (Table 5). Both tests include an unloading-reload cycle in the vicinity of the creep pressure, as well as an unloading at the end of the test. The identification is carried out on the monotonic part (Zentar 1991).

Test	P_l [kPa]	P_f [kPa]	P_0 [kPa]
3/7m	205	150	80
3/5m	185	125	60

Table 5. Pressuremeter test results on the Saint-Herblain clay (Zentar, 1991)

4.4 Results and discussion

The numerical modeling of the pressuremeter test with initial parameters set has provided the curves shown in Figure 11 and Figure 12, where it is clear the large difference between the numerical and the experimental curves. So, the two parameters, G and Cu, were varied according to the given formulas (9 and 10). Parameters that give a closer curve are retained (Table 6).

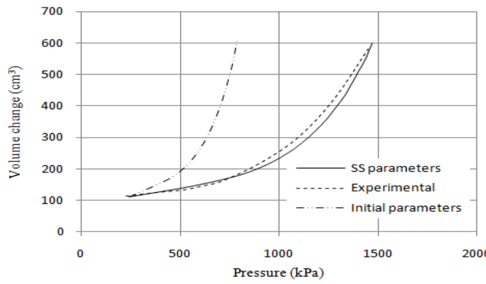


Figure 11. Identification of the model parameters from pressuremeter test (Algiers at 14m)

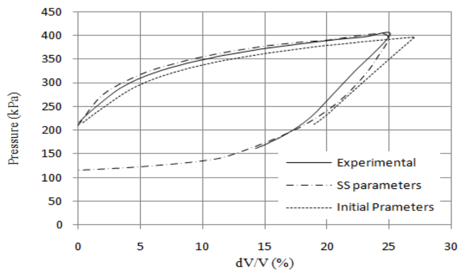


Figure12. Identification of parameters from pressuremeter test modeling, Cran clay (12.6m

	G (bars)	Cu (kPa)
Initial parameter	76	146
Identified Parameter	198	320

Table 6. Identified parameters (Algiers site at 14 m)

4.4.1 The identified undrained Cohesion

Figure 13, Table 7 and 8 illustrate the values of undrained cohesion identified from the pressuremeter test modeling and the values determined by the empirical formulas of L. Menard, Amar and Jézéquel and analytical methods of Baguelin and Gibson (Cassan 1976; Briaud 1992; Baguelin et al. 1972; Gibson et al. 1961), for the different sites. It is found that the values identified for the Tunnel and Saint-Herblain sites are higher than those determined by empirical formulas. This difference can be attributed to several factors, such as empirical relationships, which are based on correlations between pressuremeter parameters; these parameters are generally affected by boring quality.

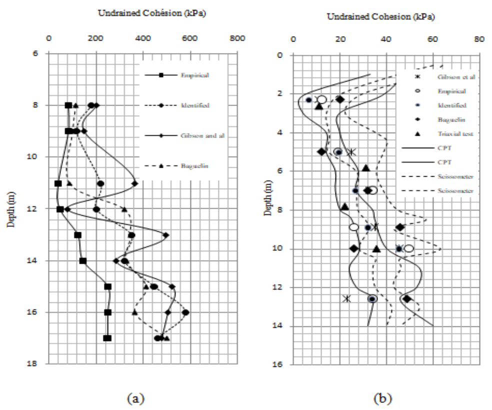


Figure 13. Undrained cohesion profiles:
a) Algiers site, b) Cran Clay

	Initial Parameters		Identified parameters		Identified parameters (Zentar)	
	G [kPa]	Cu [kPa]	G [kPa]	Cu [kPa]	G [kPa]	Cu [kPa]
Test3/5	440	22.5	552	36	540	32
Test3/7	375	23	530	38	506	33

Table 7. The results of the identification on the Saint-Herblain clay

Also, the choice of the initial pressure at rest is a major problem of numerical identification, and this task becomes difficult when it is a pressuremeter with pre-boring.

The results of the identification on Saint-Herblain clay presented in Tables 7 and 8, show the difference between the parameters identified by various empirical, analytical, and numerical methods. However, the values obtained numerically (by the decomposition of the pressuremeter curve) are close to those identified numerically by Zentar (using the conjugate gradient method).

	Ménard [kPa]	Baguelin et al [kPa]	Arnold et al [kPa]	Gibson et al [kPa]	Scissometer [kPa]
Test3/5	22.5	31	40	35	20
Test3/7	23	30	42	41	25

Table 8. Undrained cohesion values using different analytical methods (Saint-Herblain clay)

For Cran clay, the identified values of cohesion are compared with those determined by other tests, namely the scissometer, the CPT and the triaxial test, and other analytical methods, where it is found that the difference is not significant between the values identified numerically and those determined with in situ tests (Figure 13b).

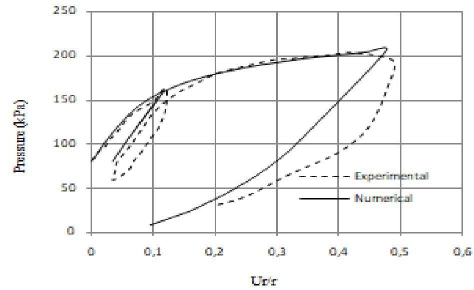


Figure 14. Simulation of the pressuremeter test on the Saint-Herblain clay 3/7m

4.4.2 Shear Modules

The shear modulus values identified at the Algiers site given by Table 9 are larger than those calculated from the pressuremeter modulus. For example, the shear modulus identified at 14 m depth is equal to 19.8 MPa, whereas the calculated value is equal to 7.5796MPa; so, the identified modulus is almost three times the calculated value. This appears logical, because the pressuremeter modulus E_M (Menard pressuremeter) is not determined in the first portion of the curve but in the pseudo-elastic part.

Depth [m]	G[bar]	G_{id}[bar]	Ratio
8	11,88	46,5	3,9
9	24,966	126,5	5
11	2,09	7,2	3,44
12	3,6052	10	2,77
13	20,319	90	4,42
14	75,796	198	2,61
15	140,82	480	3,40
16	163,67	570	3,48
17	200,928	830	4,13

Table 9. Identified shear modules (Algiers site)

For Cran clay, different values of shear modules are determined experimentally and numerically (Figure 15 and Table 4), where it is observed that the shear modulus identified numerically at different depths is closer to the G_{p2} modulus (modulus determined at 2% of strains) than to the initial modulus. This may be attributed to the problem of perturbations induced during self-boring, which affects the initial part of the pressuremeter curve, so the initial modulus is affected. The same observation is made on the shear modulus identified on the Saint-Herblain clay; it is noticed that identified numerical values, either by the decomposition method or by the gradient method (Zentar 1999) are higher than those determined on the initial part of the experimental pressuremeter curve. The calculation of the shear modulus on the unloading-reloading cycle gives a value of 558 kPa and 550 kPa, for the 3/7 m and 3/5 m tests respectively; these values are close to the identified values, which are of the order of 552kPa for test 3/7 and 530 kPa for test 3/5.

From this result, it can be said that performing pressuremeter test with an unloading-reloading cycle is necessary to determine the shear modulus and consider it as an initial pressuremeter modulus especially when the clays are concerned or the initial portions of pressuremeter curves are affected by self-boring.

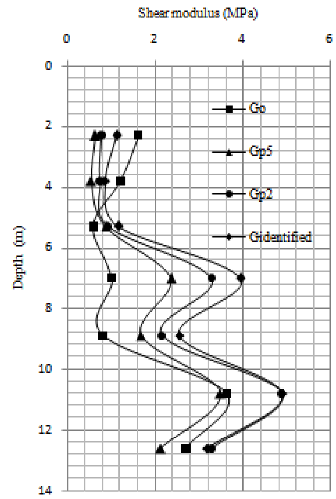


Figure 15. The different shear modulus (Cran Clay)

4.5 Cyclic pressuremeter test

To better understand the importance of performing pressuremeter tests with an unloading-reloading cycles, the interest is focused on a series of pressuremeter testswith three unloading-reloading cycles, carried out on the Chilly-Mazarin site (Camille 1981). The numerical modeling of these tests has been carried out respecting the initial conditions and the amplitude of the cycles; the parameters introduced for this modeling were previously identified on a monotonic test.

In Table 10 summarized the shear modulus determined from the experimental and numerical pressuremeter test performing at the Chilly-Mazarin site.

Test	G_0 [MPa]	$G_{identified}$ [MPa]	$G_{exp(cyclic)}$ [MPa]	ratio
1	2.067	18	16.8	1.07
2	1.57	12.9	11.08	1.16
3	1.92	19.5	13.6	1.43
4	3.45	19.1	14.89	1.22
5	6.72	22.4	19.55	1.14

Table 10. Experimental and numerical shear modulus (Chilly-Mazarin)

An initial modulus is computed on the first loading part [0.4, 0.8bars] of the experimental curve, which has been used for the set as an initial parameter, a second modulus is determined from the unloading-reload loop of the first cycle; it is noted *Gcyc*, and the third modulus is identified numerically. The values of shear modulus identified are higher than those determined experimentally on the initial part of the pressuremeter curve, but they are closer to those determined from the unloading-reloading part, where the ratio between the two values does not exceed 1.5. This confirms the result already found (Table 11). From this result, it can be concluded that the modulus derived from an unloading-reloading cycle can be considered as a pressuremeter modulus, this is in agreement with the suggestion of Menard, who introduced this idea for the first time, where he proposed to use this module to study the soil behavior under vibrating machines (Ménard and Lambert 1966). This use seems to be necessary when the pressuremeter curve has no initial part or is difficult to interpret or when the soil is affected by pre-boring or self-boring (Zentar 1999; Combarieu et al. 2001; Reiffstech et al. 2013; Ferreira et al. 1992; Combarieu et al. 2006).

Test	Amar and Jézéquel [kPa]	Identified [kPa]
2	97	180
4	100	108
5	92	118

Table 11. Identified and empirical cohesion values (Chilly-Mazarin)

5. INFLUENCE OF THE NUMBER OF UNLOADING-RELOADING CYCLES ON THE SHEAR MODULUS

The results of the numerical modeling of the pressuremeter test with three cycles (Chilly-Mazarin, Figure 16) show well that the cycles are almost confounded, where the volume variation of the three cycles is very negligible and does not exceed 1 cm^3.

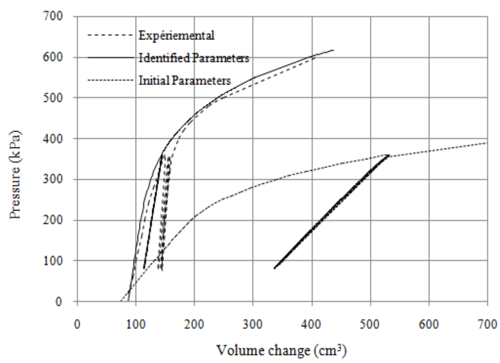


Figure 16. Modeling of the pressuremeter test with three cycles (Chilly-Mazarin)

The calculation of the modulus on the three cycles gave the same value, where it is clear that the slope of the cycles does not change from one cycle to another. The same result is observed in the experimental tests; the strains tend to stabilize after the first cycle, where the deformation of the first cycle is about 13 cm^3, whereas it is 11cm^3 for the second and the third cycles. For the shear modulus, it increases after the first unloading-reloading cycle and then tends to stabilize (Table 12).

Test	G_{id} [MPa]	$G_{1st\ cycle}$ [MPa]	$G_{2nd\ cycle}$ [MPa]	$G_{3rd\ cycle}$ [MPa]
2	12.9	11.08	10.75	10
4	19.1	14.89	13.73	13.73
5	22.4	19.55	18.30	18.30

Table 12. Cyclic numerical shear modulus

Figure17 shows the variation of the tangent and secant modulus as a function of unloading-reloading cycles, which are determined from numerical modeling of the cyclic pressuremeter test carried out on Cran clay and Hostun sand (Bahar and Belhassani 2012). It is noted that the two modulus decrease after the first unloading and tend to stabilize after the third or the fourth cycle.

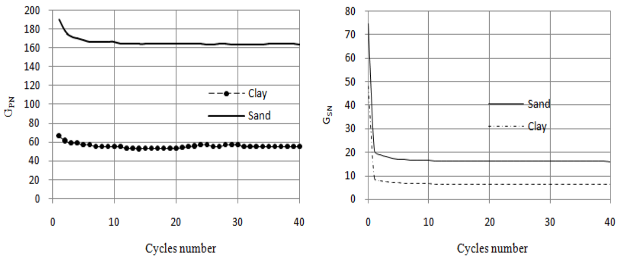


Figure 17. Simulation of pressuremeter test with variation of the cohesion performed at Chilly-Mazarin clay at 4m depth

6. INFLUENCE OF COHESION ON THE RESPONSE OF CYCLIC LOADING

In the sensitivity study,it has been shown that the undrained cohesion has a great influence on the soil response on the monotonic and cyclic pressuremeter test. A numerical modeling of pressuremeter tests with three unloading-reloading cycles has been carried out at Chilly Mazarin clay at 4m at depth, with varying undrained cohesion values. The resulting curve given by Figure 18 shows clearly the change in the soil response, it is noticed that the cyclic deformations are more important (unloading-reloading cycles are more open), the cohesion is equal to 40 kPa, whereas the identified value is equal to 108kPa, and the change in volume increases from 1 cm^3 to 5 cm^3.

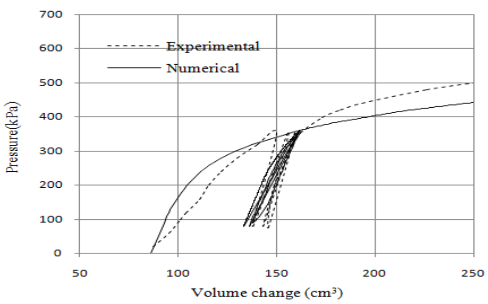


Figure 18. Simulation of pressuremeter test with variation of the cohesion performed at Chilly-Mazarin clay at 4m depth

6.1 Numerical modeling of pressuremeter test with one unloading-reloading cycle

A numerical simulation of the pressuremeter test with unloading-reloading portion is performed at Saint-Herblain clay, using the identified values on the monotonic path (Table 7). Resulting curves given by Figure 15, illustrate the difference between the numerical and experimental curves in the unloading-reloading portion part. The analysis of the influence of the cohesion and the friction on the shape of the cyclic curve made it possible to see how the volume of the cyclic deformations varies and leads to simulating this test with varying the cohesion and the shear modulus.

Figure 19 illustrates a simulation with varying the cohesion and shear modulus values. The best simulation compared to the experimental curve is obtained by introducing the values of cohesion and shear modulus equal to 22 kPa and 1.27 MPa, respectively. It is noted that the value of the cohesion is close to the value given by the sissometer test ,which is about 25kPa.

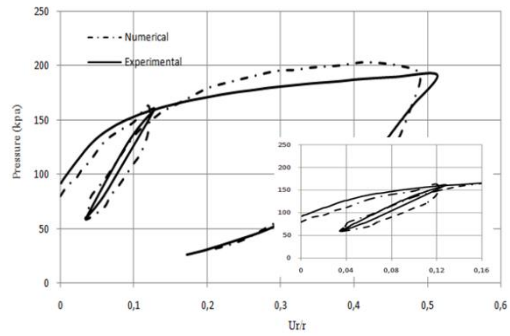


Figure 19**.** Simulation of pressuremeter test with variation of the cohesion performed at Saint-Herblain 3/7m

6.2 Numerical modeling of pressuremeter tests with several unloading-reloading cycles

Cyclic tests with the self-boring pressuremeter are performed at the Cran clay at 9.80 m depth, carrying out fifty (50) cycles of unloading-reloading between two bounds, 240 and 220 kPa (Figures 20, 21). A simulation of this test is carried out, considering the similar conditions of loading and varying the identified values of cohesion and shear modulus on the monotonic path. The closest simulation compared to the experimental curve is obtained for the values of cohesion and shear modulus equal to 30 kPa and 12.7 MPa, respectively. The shape of the two curves is similar in both the loading and unloading-reloading parts, and the volume of the reversible deformations is almost close for the two curves. It is noted that the identified value of cohesion agrees with those defined from other means (Figure13b).

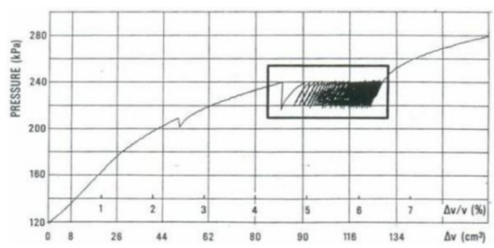


Figure 20. A cyclic pressuremeter test at Cran site, depth of 9.8 m (Fay 1990)

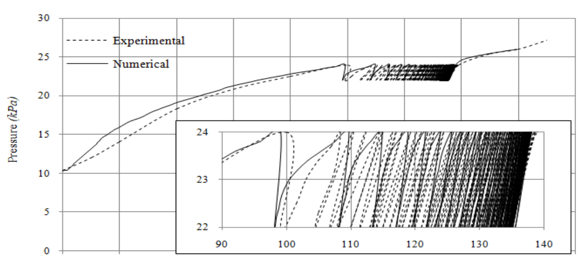


Figure 21. Simulation of pressuremeter test with 50 unloading reloading cycles at Cran Clay at 9.8m

The results are promising and allow gaining valuable information on the cyclic behaviour of the soil, which is very important to examine the stability of structures during earthquakes or wave loading. It can be concluded that the cyclic pressuremeter test can be used as a means and is helpful for the identification of soil parameters.

6.3 Comparison between monotonic and cyclic pressuremeter tests

Figure 22 illustrates the evolution of the pressure and the excess pore pressure versus radial displacement, resulting from the simulation of a monotonic pressuremeter test (a) and pressuremeter test with one unloading-reloading cycle (b), performed at Saint-Herblain clay. It is noted that in the two paths as loading or unloading, the evolution of the pore pressure follows the same way of the deformation, where, it increases when it is loading and decreases when it is unloading.

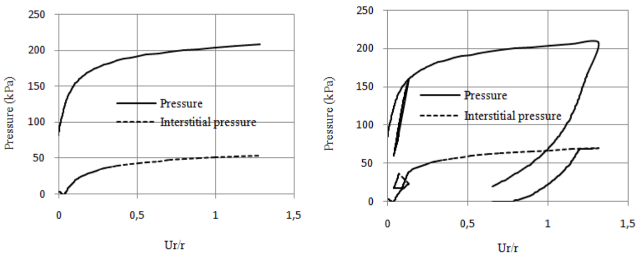
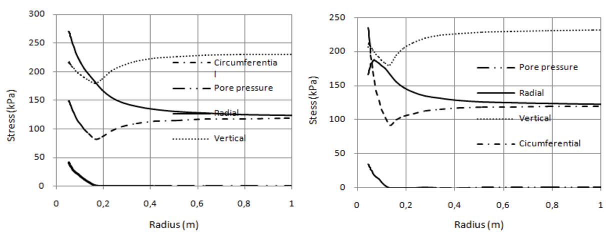


Figure 22. Pressuremeter and pore pressure curves for monotonic and cyclic test at Saint-Herblain clay

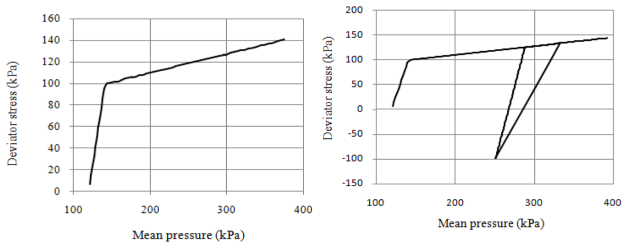
Also, the variation of total stresses; $\sigma_r, \sigma_\theta, \sigma_z$ and the pore pressure along the radius can be drawn from the simulation of the pressuremeter test carried out at Cran clay. Figure 23 shows that the total stress is important until a radius of approximately 40 cm around the probe; beyond this range, all the stresses and pore pressure tend to stabilize. However, a difference between the evolution of stresses in the monotonic test and cyclic test is observed.



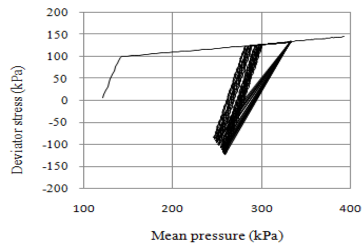
a) b)

Figure 23. Total Stress distribution along the radius (Clay 2.6 m): a) without unloading, b) with unloading-reloading cycles

Figures 24, a, b, and c present the paths of stress deviator versus mean pressure, resulting from the simulation of pressuremeter tests, (monotonic and cyclic test with one and several unloading and reloading cycles). It is shown that there is an evolution of deviator stresses and mean pressure, where the path follows the solicitation (loading, unloading, and reloading).



a) Monotonic b) with unloading reloading



c) Several unloading reloading cycles

Figure 24. Evolution of the deviator stress versus mean pressure at Cran clay (8,9m)

7. CONCLUSION

The numerical analysis of the pressuremeter test using the FLAC software code, taking into account the soft / hardening soil behavior model, was carried out in order to identify the soil behaviour parameters. The identification based on the method of decomposition of the pressuremeter curve was performed on the monotonic path, where two parameters have been identified: shear modulus (G) and undrained cohesion (Cu). The identified parameters resulting were used for the simulation of cyclic pressuremeter tests. The comparison between numerical and experimental curves showed that the numerical cyclic part is different from that observed experimentally, where the unloading and reloading portions are closed and the cyclic modulus does not change. However, the variation of the parameters made it possible to approach the experimental cyclic portion.

From these results, it can be said that the numerical modeling of monotonic and cyclic pressuremeter tests allows identifying the soil behavior parameters, although these two paths give different parameters; they fall within the range of values determined experimentally. Also, it is found that performing a pressuremeter test with an unloading-reloading portion is very important, and determining the shear modulus from this portion is much better than the initial part of the pressuremeter curve.

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