Assessment of shear stiffness at small strain level using an innovative monocell pressuremeter probe

A. Lopes dos Santos
Fugro France, Nanterre, France, a.lopes@fugro.com

A. Puech
Fugro France, Nanterre, France, a.puech@fugro.com

F. Cour
Calyf, Maisons-Laffite, France, courfrancis@gmail.com

J.C. Dupla[1], J. Canou[2]
Ecole des Ponts Paristech, Champs-sur-Marne, France, jean-claude.dupla@enpc.fr[1], jean.canou@enpc.fr[2]

ABSTRACT: The Francis Cour® Monocell probe is an innovative pressuremeter probe that takes advantage of recent developments in membrane technology. Its enhanced capabilities open access to the *in-situ* measurement of the shear modulus G at small strains (between 10^{-4} and 10^{-2}), a domain generally reserved for flexible dilatometers. The validation of its measurement capabilities is underway. This paper focuses on validation tests with the Francis Cour Monocell probe carried out on a reference field site. Special testing procedures including unload-reload loops were performed aiming to assess soil non-linear elastic response. The testing site subsoil, mainly composed by overconsolidated clay, has been previously characterized by various *in-situ* and laboratory tests. Testing procedures and interpretation methods are described and discussed. Shear moduli obtained are compared to geophysical and laboratory data collected on the site. This work is part of the French National Project ARSCOP.

Keywords: *In-situ* tests, pressuremeter test, dilatometer test, shear modulus, small-strain

1. Introduction

Pressuremeter tests are the most used *in-situ* test for underground investigation in French geotechnical engineering practice. Performed according to current standards, these tests make it possible to obtain the so-called Ménard pressuremeter modulus and the pressuremeter creep and limit pressures. Ménard pressuremeter parameters are used in standard foundation design through well established and accepted empirical correlations. However, the design of a number of geotechnical structures (e.g. retaining walls, foundations under cyclic loading) is more demanding and requires establishing the foundation response under low strain levels. The necessary deformability parameters cannot be obtained directly through standard pressuremeter testing protocols, nor using the most common testing equipment due to measurement limitations.

The Francis Cour® (FC) Monocell probe is an innovative pressuremeter probe that offers new possibilities in soils and soft rocks. It has been first presented by [1], (in French) and the first attempts to use it to derive elastic properties at small strains were presented by [2], confirming its potential capability.

The probe enables measurements in both the small strain and the large strain domains. The first is traditionally reserved for "flexible dilatometers" or probes equipped with local punctual strain sensors, and the second, for standard pressuremeters. The Monocell FC probe's maximum expansion capability also enables direct measurement of the conventional soil limit pressure, associated to very large strains.

Two different approaches were undertaken in order to validate this probe's measurement capabilities: the first is based on tests performed under fully controlled conditions in a laboratory calibration chamber [3]. The second approach is based on tests performed on sites in which the soil layers have been well characterized by a large set of geotechnical and geophysical tests.

This paper presents the testing procedure and the results obtained using the Monocell FC probe at the Merville testing site, mainly composed by oversonsolidated clays. The results were compared to stiffness assessed using other investigation methods, such as geophysics and advanced laboratory tests. A satisfying agreement was obtained, confirming the probe's capability to assess soil deformability properties at low strain levels.

1.1. The Monocell FC probe characteristics

The Monocell FC probe was primarily designed to overcome some of the recurring difficulties in measuring the conventional pressuremeter limit pressure in stiff soils (a combination of high inflation volumes and high pressures). It comprises a single water cell surrounded by a textile restraining sheath. This last component is made of hybrid elastic cables and enables controlling the membrane's geometry during its inflation. This sheath minimizes the uncertainties associated with the expansion of the measuring cell and allows an accurate assessment of the relationship between the injected water volume and the probe's outer diameter.

Geometry control also enables minimizing stress concentration near the membrane extremities, improving its durability. These characteristics make the probe interesting for use in repeated cyclic testing.

A detailed description of the equipment is presented by [3]. Figure 1 presents photographs of the probe when inflated to its maximum capacity.

Figure 1. Photo of the probe inflated to its maximum outer diameter. (a) the restraining sheath, (b) the polyurethane sheath protection.

2. Testing protocol and interpretation

Assessing moduli at small strains with cavity expansion tests is a delicate task and requires performing special testing protocols, more complete than those described in current pressuremeter standards. The success of the tests relies on (1) fully calibrating the probe within all of its operation domain using calibration tubes of various diameters; (2) undertaking so-called “membrane compliance” tests to correct for membrane compliance during unload-reload loops; (3) adequately placing the probe in the ground, avoiding heterogeneous layers that cannot be tested by cavity expansion tests; (4) applying a loading program that favors the assessment of soil’s elastic properties and its evolution with stress and strain; (5) interpreting the test results based on adequate non-linear elasticity background. The following sections briefly describe each of these procedures.

2.1. Probe calibration

The Monocell FC probe is a volumetric measurement probe, which means that measurements of the water volume injected inside its expandable cell are used to evaluate the changes on its external diameter, and thus assessing the expansion of the cavity wall. Measurements are made at the ground level. A particularity of the design of this probe is that the relationship between its volume and its outer diameter is linear, enabling direct calculation of its diameter for all the operation range in terms of pressure and volume. Calibration tests allow obtaining this relationship for all the range of pressures and volumes and it is further used for the test interpretation. Figure 2 presents an example of calibration using four diameter calibration tubes performed with the probe used for the tests herein.

As the pressure measurements are made at the ground level, additional hydraulic water-head correction is necessary to account for the test depth. Membrane inertia corrections are also necessary to account for the difference between the pressure inside the probe and that effectively applied at the soil’s cavity walls. Membrane inertia calibration is similar to that generally proposed in standard recommendations (usually referred to as pressure loss calibration) and consists in inflating the probe up to its maximum volume in open air at a similar inflation rate of that used during tests in soil.

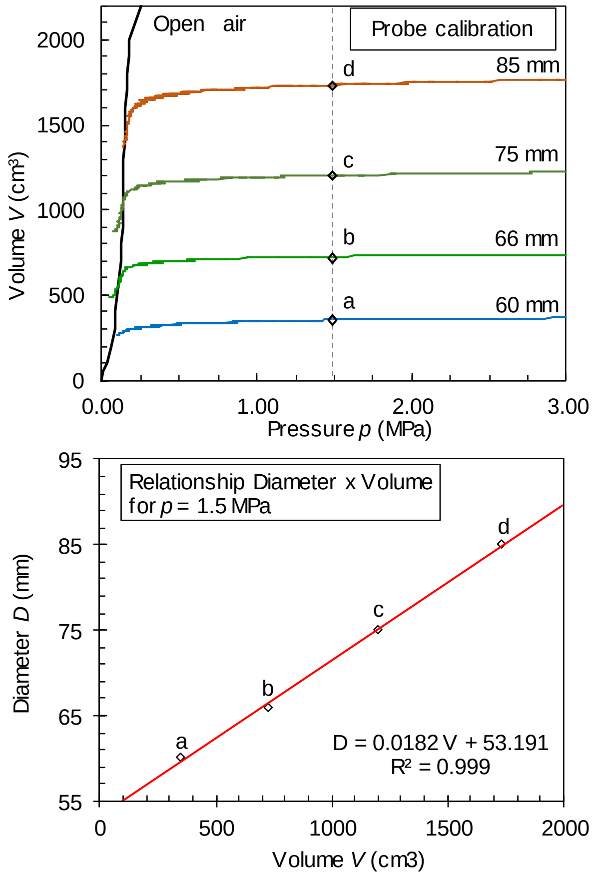

Figure 2. Example of probe calibration test and the linear relationship between probe volume and probe diameter

2.2. Membrane compliance calibration

Most inflatable probes exhibit a hysteretic behavior when an unload-reload loop is performed. The first evidence of this behavior was presented by [4] performing cycles inside a calibration tube using a monocell probe of the Texam type. This same phenomenon was further evidenced by [5 – 7].

The Monocell FC probe also needs to be calibrated for membrane compliance, as it has been confirmed in laboratory calibration tests using an instrumented elastic cylinder [3]. The procedure consists in performing a complementary calibration test after the probe has been fully calibrated. For this test, the probe is placed inside a calibration tube of diameter close to that of the borehole to be done in soil and a loading program including unload and reload loops at increasing pressure levels, $p_{cav,i}$, is performed. The compliance test is interpreted as if it was a test in soil. The slope of each unload-reload loop is calculated and yields a fictive “system shear modulus”, which is a function of the probe pressure before performing the loop, $G_{sys}(p_{cav,i})$. A plot of G_{sys} as a function of $p_{cav,i}$ enables determining the so-called “compliance law", which allows calculating corrected shear modulus according to equation (1) proposed by [5]:

$$\frac{1}{G_{corr(p_{cav,i})}} = \frac{1}{G_{meas(p_{cav,i})}} - \frac{1}{G_{sys(p_{cav,i})}} \quad (1)$$

in which $G_{meas(p_{cav,i})}$ is the shear modulus calculated before membrane compliance correction and $G_{corr(p_{cav,i})}$ is the corrected shear modulus (the soil's shear modulus).

2.3. Probe positioning

Another difficulty related to the pressuremeter test regards the position of the probe into the borehole. In multi-layered soils, in terrains where geological shear bands can be present or in any other heterogenous ground formation, care has to be taken in relation to the precise depth of the probe. If the probe is placed straddling between two or more layers of different stiffness or strength, the cavity expansion test will not deploy cylindrically and thus will not be valid for interpretation. The test operator needs to use parallel investigation methods, or previous knowledge of the terrain, to place the probe in ground conditions that are relatively homogeneous. It was suggested by [8] that the drilling parameters should be analyzed by the operator on-site, before performing the test, as a tool for helping to decide where to place the probe. Placing the probe in a zone with a contrast of stiffness can damage it.

Besides a careful analysis of geological conditions before inserting the probe, drilling quality and the operator's experience are of major importance to ensure a cylindrical cavity is formed. Regarding the eventual disturbance caused by drilling, it has been shown in literature that moduli obtained by unload-reload loops performed after first expanding the cavity to a significant expansion level are not sensitive to probe installation effects [9–12].

2.4. The loading program

It is important to keep in mind that cavity expansion tests are performed into non-homogenous stress and strain fields and that care must be taken to ensure that measurements made at the cavity walls (cavity pressure and cavity strain) can be interpreted to obtain intrinsic soil properties. Some mechanical phenomena may superpose during the test, such as elasticity and plasticity; creep, relaxation and consolidation; stress and strain dependency. The loading program must favor assessing each of these phenomena separately. It should be such as to avoid that time-dependent phenomena, such as creep and relaxation, superpose to the desired elasticity properties that can be evaluated during unload-reload loops. For this reason, sufficiently long pressure-hold steps must be performed before unloading the cavity walls.

A compromise procedure consists in:

1. Loading at a constant volume change rate up to a pressure p_1 just at the end or after the so-called "pseudo-elastic" domain.
2. Performing a pressure-hold step for a sufficiently long time so that creep reduces to a considerably low value.
3. Unloading at a constant shear or pressure-rate until a pressure amplitude $\Delta p'_{cav} \sim 0.4 \cdot p_1$ is reached.
4. Performing a pressure-hold step of sufficient duration to considerably reduce time-dependent phenomena after unloading;
5. Reloading at a constant volume change rate up to a pressure p_2;
6. Repeating this procedure for all unload-reload loops and then loading at a constant flow-rate up to achieving the limit pressure. The loading program is illustrated in Figure 3.

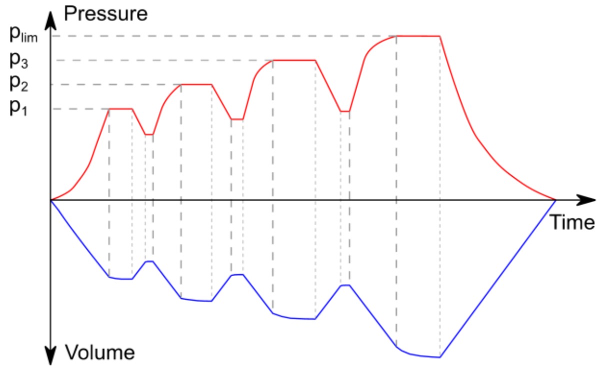

Figure 3. Principle of the proposed loading program for tests including unload-reload loops

During loops, the rate of loading should have only small influence on the results provided enough time was given on step (2). However, the value of limit pressure can be influenced by the loading rate adopted.

2.5. The interpretation methods

The derivation of shear modulus at small strains from pressuremeter tests requires the cavity expansion curve to be interpreted in the framework of non-linear elasticity, considering stiffness dependency both on stress and strain. The derivation of non-linear shear moduli from unload-reload loops has been discussed in the literature and different methods have been proposed. One possibility is interpreting each unload and reload loop through a non-linear power law model, such as proposed by [13], for obtaining secant shear modulus degradation curve for shear strain levels between 10^{-4} to 10^{-2}. Another possibility is interpreting each unload and reload loop using a hyperbolic curve, such as proposed by [4], for obtaining the full pressuremeter stiffness degradation curve. The transformed strain approach, proposed by [14], can be used for transforming secant pressuremeter moduli into equivalent intrinsic soil moduli. Tests performed under drained conditions require additional stress-state adjustment, which will not be discussed here.

In the present work, the following interpretation methods were used to derive soil's shear stiffness degradation in function of shear strain: (1) power law model proposed by [13] and (2) hyperbolic model proposed by [4] combined with the transformed strain approach [14]. A brief description of each method is presented in the following paragraphs.

2.5.1. Hyperbolic model

Reference [4] suggests that a hyperbolic model can be used to describe the cavity stress and cavity strain relationship measured during a pressuremeter unload-reload loop. In the proposed model, the reciprocal of the secant modulus is a linear function of the strain, as follows:

$$\frac{\varepsilon_c}{\sigma_c} = a + b\varepsilon_c \quad (2)$$

in which σ_c is the cavity stress; ε_c is the cavity radial strain; and *a* and *b* are the model parameters, obtained by the least square method. According to the authors, this model can be used to determine the soil's maximum modulus by calculating eq. (2) for $\varepsilon_c = 0$. Thus, by extrapolation of the fitted curve, the model yields the following estimation for G_{max} for each unload or reload loop (i):

$$G_{max,i} = \frac{1}{a_i} \quad (3)$$

Equation (2) can be rewritten to give stiffness degradation within each loop in function of cavity strain as follows:

$$G_{(\epsilon_c)} = \frac{1}{a+b\epsilon_c} \quad (4)$$

It is possible to derive soil's elementary shear modulus degradation curve by combining the pressuremeter moduli degradation (assessed at the cavity walls) with the so-called "transformed strain" approach. In the case of clays, [14] proposed a simplified method for transforming secant pressuremeter shear modulus into equivalent elementary soil modulus. The method consists in transforming secant radial strain measured at the cavity wall, ϵ_c, into elementary strain, ε_s, using the empirical equation (5):

$$\varepsilon_s = \frac{\varepsilon_c}{1.2+0.8*log10\ (\varepsilon_c*10^{-5})} \quad (5)$$

2.5.2. Power-law model

A power-law model was proposed by [13] to describe stiffness degradation in undrained conditions. The interpretation procedure is detailed below.

For all unload-reload loops, unloading and reloading parts are isolated. The authors consider that analysis can be done either on the unloading or on the reloading portion: on the worked examples it was done on the unloading portion.

A change of origin is made for each loop so that the point of reversal of the direction of loading corresponds to $\sigma_c = 0$ and $\epsilon_c = 0$. For each loop, a power-law curve of the following type is fitted:

$$\sigma_c = \eta\gamma_c^{\beta} \quad (6)$$

in which $\gamma_c = 2\epsilon_c$, and the parameters η and β can be obtained by the least square method. The shear stress can be estimated as:

$$\tau = \gamma_c(d\sigma_c/d\gamma_c) = \eta\beta\gamma_c^{\beta} \quad (7)$$

Once those parameters are obtained for each loop, the secant shear modulus can be calculated as:

$$G_s = \eta\beta\gamma_c^{\beta-1} \quad (8)$$

The method was then simplified and extended to drained conditions enabling stress state adjustment, as presented by [15]. Since the test presented on this work was performed under undrained conditions in clay (no effective stress change), stress state adjustment will not be discussed herein.

3. *In situ* tests - Merville testing site

3.1. Site description

Merville testing site held many campaigns of static pile tests since the 80's. The most recent pile testing campaign on the site was performed in the context of the SOLCYP project [16], aiming to improve the design of piles under cyclic axial loads. The existing geotechnical characterization campaigns, composed of laboratory and in-situ tests, geotechnical and geophysical, were summarized by [17]. The site location, with a detail for the location of the pressuremeter campaign performed in 2018, is presented in Figure 4. The site's stratigraphy can be described as follows: Flander's overconsolidated clay is found below an approximately 2-meter silt layer, and extends down to 42 meters depth, below which Landenian sands are encountered. Groundwater table fluctuates between 1.5 and 1.9 meters depth, into the coverage silt layer. It is, however, difficult to estimate the phreatic level into the very impermeable, but micro-fissured, Flander's clay.

Figure 4. Location of the Merville testing site and of the pressuremeter testing campaign

Flander's clay is classified as very plastic. Atterberg's limit tests performed on specimens collected on the site resulted in plasticity index ranging from 40 to 69%. Bulk unit weight ranges between 18.5 and 19.5 kN/m^3 for depths between 4 to 12 meters [17]. At these same depths, CPT resistance, q_t, increases approximately from 1.5 MPa to 4.0 MPa (Figure 5), and undrained shear strength increases from 50 kPa to 150 kPa. Initial shear stiffness was assessed using cross-hole, down-hole and surface wave geophysical tests. Standard Ménard pressuremeter tests were also carried out in previous investigation campaigns. A synthesis of shear moduli assessed on the site using different techniques was presented by [18] (Figure 6).

The current pressuremeter testing campaign comprised tests from 9 to 15 meters depth. The results presented herein correspond to one test performed at 12 meters depth. Other test results are to be communicated in a further paper. As it can be observed from Figure 5, CPT's tip resistance (q_t) at the depth of interest ranges between 3 to 4 MPa and conventional Ménard limit pressure is of about 1.5 MPa. Undrained shear strength is about 150 kPa. Initial shear modulus ranges between 45 MPa and 70 MPa, according to Figure 6. Shear moduli derived using standard pressuremeter tests are also plotted in this figure and range between 10 and 15 MPa.

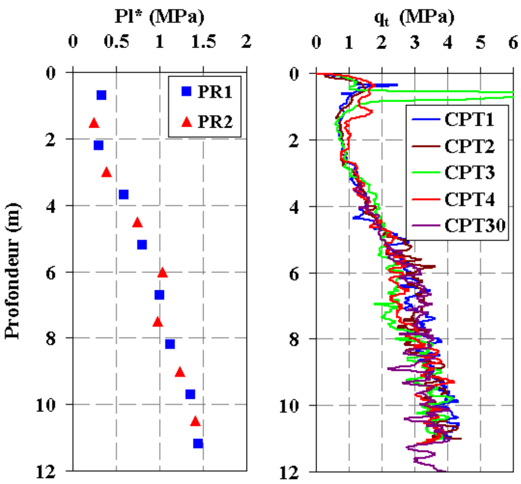

Figure 5. Profiles of limit pressure and cone tip resistance at the Merville site [19]

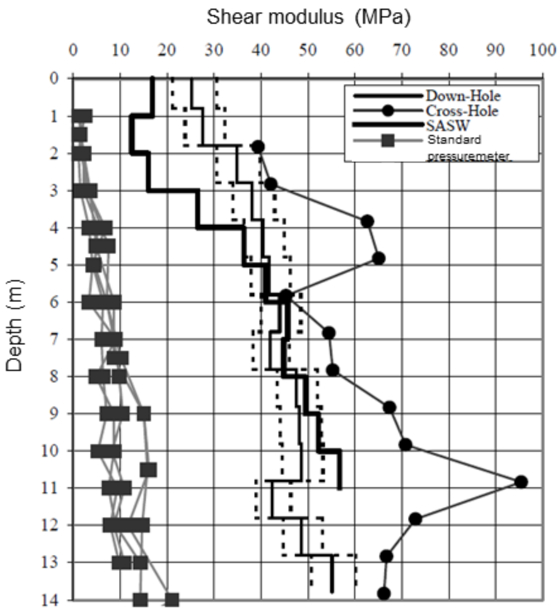

Figure 6. Profile of shear modulus assessed at Merville testing site using different techniques, adapted from [18]

The test pocket was drilled using a continuous auger of external diameter 60 mm without bentonite injection. Test interpretation showed that the initial cavity diameter has been over-drilled to an initial diameter of 62 mm, without prejudice to the test results. The coverage silt layer has been cased to avoid the upper groundwater table to penetrate the test cavity.

3.1.1. Test results

A loading program according to that illustrated in Figure 3 was performed (Figure 7). The cavity expansion curve (cavity pressure versus cavity strain) is presented in Figure 8. From this curve, the standard Ménard pressuremeter modulus can be calculated as the slope of the quasi-linear portion between 0.230 and 0.430 MPa and is equal to E_M = 35 MPa, which corresponds to a shear modulus G_M = 13.1 MPa. The test was intentionally stopped before reaching the conventional limit pressure, which is slightly superior to 1.5 MPa. A fourth loop (L4) could be performed during unloading, but it will not be interpreted on this work.

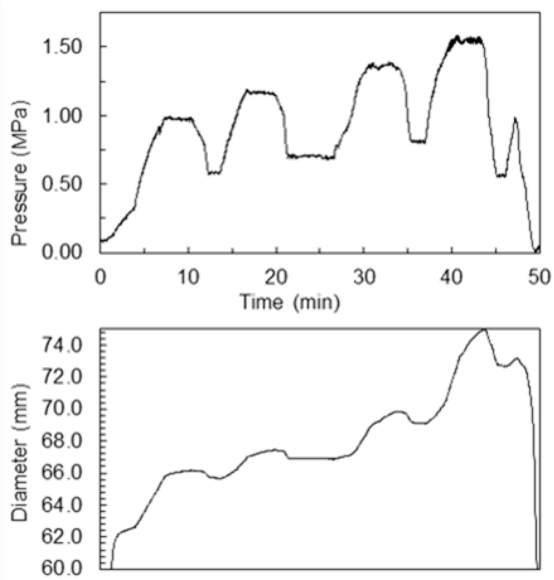

Figure 7. Testing program and soil response for the test at 12 meters depth at Merville site

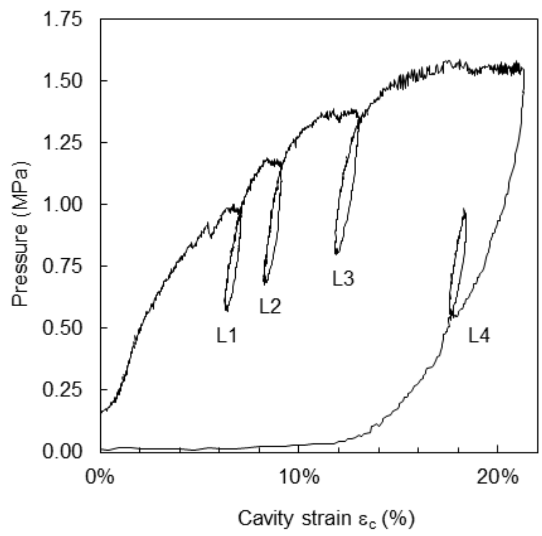

Figure 8. Cavity expansion results for the test performed at 12 meters depth at the Merville site.

Details of each of the three unload-reload loops (L1 to L3) are presented in Figure 9. Data was intentionally not smoothed. Figure 10 presents the interpretation of the unloading part of each loop after choosing an origin for the unloading path and changing coordinates to correspond to zero pressure and zero strain. One can notice that all the three unload loops present very similar slopes; the slight difference between them can be attributed to measurement uncertainties. The non-linear behavior observed confirms the effect of strain on stiffness for each loop. The obtained model parameters are synthesized in Table 1. Evaluations of maximum shear modulus using equation (3) are presented in Table 2, as values of maximum total cavity pressure before unloading, p_{cav}.

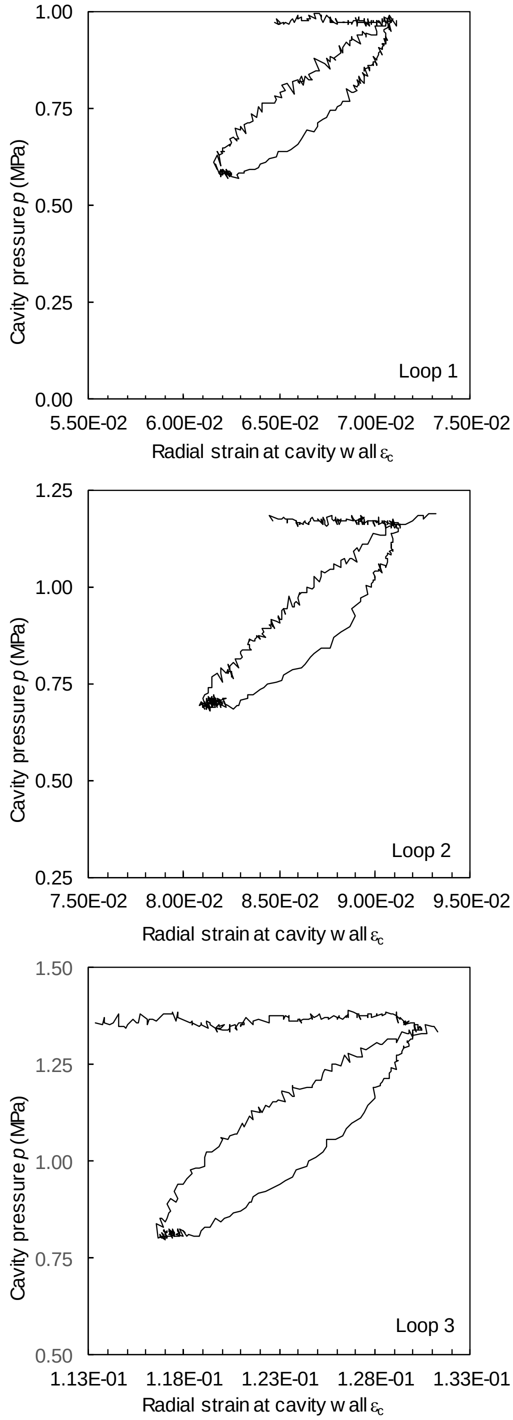

Figure 9. Detail of loops 1, 2, 3 and 4 performed 12m depth at Merville site

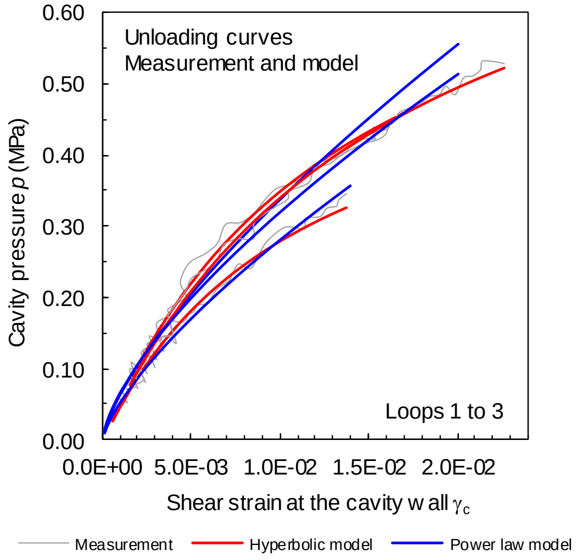

Figure 10. Unloading curves from loops 1 to 3 after origin change and fitted hyperbolic and power-law models.

Table 1. Hyperbolic and power law model parameters obtained for unload loops one to three in Merville site

	Power law model		Hyperbolic model	
Loop	η	β	a	b
L1	7.6	0.72	1.90E-02	1.68
L2	9.1	0.72	1.68E-02	1.20
L3	7.5	0.68	1.85E-02	1.10

Table 2. Evaluation of maximum shear modulus from equation (3).

Loop	G_{max} = 1/a (MPa)	p_{cav} (MPa)
L1	53	0.933
L2	59	1.144
L3	54	1.336

Derived values of secant shear modulus in function of cavity shear strain are plotted in Figure 11 for both hyperbolic and power-law models.

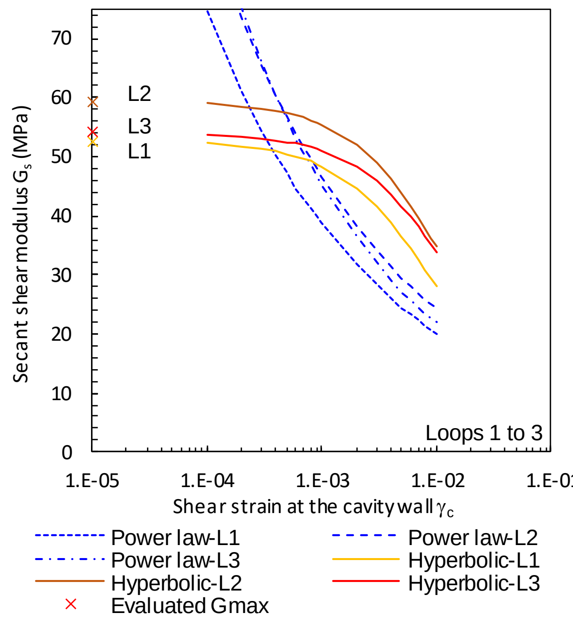

Figure 11. Derived values of secant shear stiffness using hyperbolic and power-law models for the three unload loops

Figure 11 shows that there is no evidence of dependence of the evaluated maximum shear modulus and cavity stress, which leads to a stress adjustment coefficient $n = 0$.

4. Discussion

This section is dedicated to a comparison between the results obtained using the described pressuremeter approach and the expected behavior for the Flander's clay, considering the intrinsic parameters previously obtained using other soil tests.

Figure 12 presents a comparison between shear modulus derived from pressuremeter unload-reload loops using hyperbolic and power-law models, pressuremeter standard modulus, and moduli assessed using cross-hole and down-hole tests, $G_{max,CH}$ and $G_{max,DH}$, respectively.

The shear modulus degradation curves plotted refers to the proposal by [20], which enables accounting for the effect of the plasticity index on the curve slope. The grey area delimited by the two curves covers the possible values of shear stiffness that can be expected at 12 meters depth on site. The upper-bound limit starts from the higher value of initial shear modulus, the one obtained by cross-hole tests. Its curvature was determined as a function of the highest plasticity index reported on-site, $I_P = 69\ \%$. The lower-bound limit starts from the lower initial shear modulus, obtained by down-hole seismic tests. In order to enable an appreciation of the effect of the plasticity index on the stiffness degradation, the lower boundary was plotted for $I_P = 40\ \%$ and $I_P = 69\ \%$.

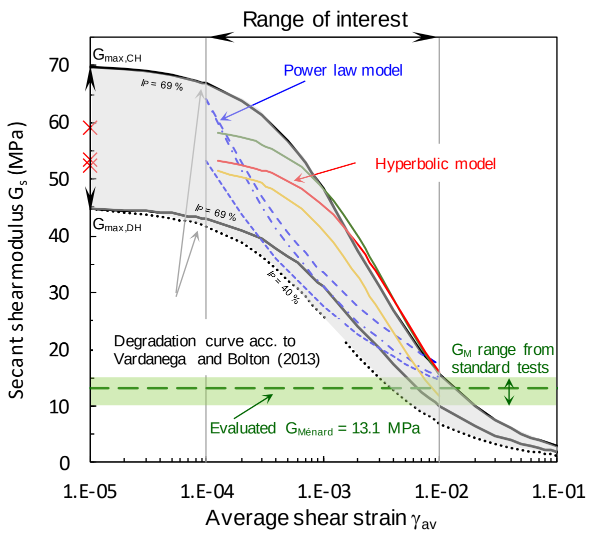

Figure 12. Comparison between the degradation curves derived from the three pressuremeter loops, the maximum shear modulus obtained using cross-hole and down-hole, the standard Ménard pressuremeter shear modulus and the theoretical degradation curves proposed by [20] for the upper and lower limits at 12m depth on Merville testing site

The following conclusions can be drawn from an analysis of the results presented in Figure 12:

- The evaluated Ménard shear modulus lies within the range of moduli obtained at the same depth on previous investigation campaigns using standard Ménard probes. This underlines the fact that the Monocell FC probe yields "standard" results similar to those obtained using other types of pressuremeters.
- The range of Ménard shear moduli plotted in the figure correspond to soil's stiffness degraded to a shear strain level of approximately 10^{-2}.
- On average, soil's initial shear modulus evaluated by geophysical tests was shown to be approximately 4 to 6 times higher than Menard shear modulus obtained using standard pressuremeter tests.
- Power law and hyperbolic models yield equivalent results at shear strain levels of approximately 2.10^{-4} and 1.10^{-2}. In the middle of the range of interest, the power-law model gives lower values of shear stiffness, approaching the lower bound of expected values.
- The hyperbolic model lies within the expected range of stiffness, nearing its upper boundary for strain levels higher than 10^{-3}. Predictions of G_{max} using this model lie in the middle of the expected range.
- There is some uncertainty on the soil's initial shear modulus evaluated from cross-hole and down-hole tests. The additional information provided by the pressuremeter tests is of great value for engineering, helping to link the small strain and the large strain behavior.
- The transformed strain approach (equation 5) gives good results for the shear stiffness decay.

5. Summary and conclusions

The testing procedures and the interpretation methods adopted on this paper enabled to assess soil's small strain elastic response using the innovative Monocell Francis Cour® probe. This work was undertaken as part of the validation program of this probe's measuring capabilities through *in situ* tests performed on reference soil layers previously characterized by other investigation methods. This work is inserted in the context of the French project ARSCOP, aimed at improving geotechnical site characterization using pressuremeters.

It has been shown that it is possible to assess soil's non-linear response using a volumetric measurement based pressuremeter probe. The technological improvements in the domain of inflatable membranes implemented in the Monocell FC probe plays a major role in this context since it enables increasing accuracy on the relation between the volume injected in the probe and the real cavity's radial strain. Besides those improvements, this result could not be achieved without the global and parallel developments related to the testing protocol. This comprises a more rigorous calibration procedure, engineer-oriented-judgment on the choice of the probe position on the ground, special cavity loading program and adequate interpretation methods.

Shear moduli evaluated with the new pressuremeter approach, using either hyperbolic or power-law models, were in good agreement with the Flander's overconsolidated clay intrinsic behavior within the range of interest of shear strain (10^{-4} and 10^{-2}). The results show the importance of the choice of the interpretation model, hyperbolic or power law, to represent the rate of decay of the shear modulus with strain. It seems that the hyperbolic model, combined with the transformed strain approach, is more suitable for this case.

The results presented herein contribute to the validation of the measuring capabilities of the Monocell FC probe. It was shown that it was possible to establish, *in situ*, a link between the small strain and the large strain behaviour of soils using this testing equipment.

6. References

[1] Cour F, Lopes A. Sonde monocellulaire innovante pour la réalisation d'essais d'expansion de cavité cylindrique (Innovative Monocell probe for performing cylindrical cavity expansion tests - in French). Journées Natl. Géotechnique Géologie l'Ingénieur, Champs-Sur-Marne: 2018.

[2] Lopes A, Puech A, Droniuc N, Geisler J, Cour F. Mesures de G à faibles déformations à partir d'une sonde préssiometrique monocellulaire. Journées Natl. Géotechnique Géologie l'Ingénieur, Champs-Sur-Marne: 2018.

[3] Lopes A, Dupla J-C, Canou J, Cour F, Puech A, Droniuc N. Laboratory Evaluation of the Measuring Capabilities of an Innovative Pressuremeter Probe in Dry Sand. Geotech Test J 2020. https://doi.org/https://doi.org/10.1520/GTJ20190209.

[4] Briaud JL, Lytton RL, Hung JT. Obtaining moduli from cyclic pressuremeter tests. J Geotech Eng 1983;109:657–65.

[5] Fahey M, Jewell R. Effect of pressuremeter compliance on measurement of shear modulus. Proc. Third Int. Symp. Press., Oxford: Thomas Telford Publishing; 1990, p. 115–24.

[6] Schnaid F. A study of cone-pressuremeter test in sand. Ph.D. thesis. University of Oxford, 1990.

[7] Cunha RP. Interpretation of self-boring pressuremeter tests in sand. Ph.D. thesis. University of British Columbia, 1994.

[8] Cour F, Lopes A. Paramètres de forage pour un positionnement judicieux de la sonde pressiometrique. (Using Borehole Drilling Parameters for Improving Probe Positioning and Results Interpretation - in French). Journées Natl. Géotechnique Géologie l'Ingénieur, Champs-Sur-Marne: 2018.

[9] Fahey M, Randolph MF. Effect of disturbance on parameters derived from self-boring pressuremeter tests in sand. Géotechnique 1984;34:81–97.

[10] Hughes JMO, Robertson PK. Full-displacement pressuremeter testing in sand. Can Geotech J 1985;22:298–307.

[11] Robertson PK, Hughes JMO. Determination of properties of sands from self-boring pressuremeter tests. Press. its Mar. Appl. Second Int. Simp. Press., 1986, p. 283–302.

[12] Houlsby GT, Withers NJ. Analysis of the cone pressuremeter test in clay. Géotechnique 1988;38:575–87.

[13] Bolton MD, Whittle RW. A non-linear elactic/perfectly plastic analysis for plane strain undrained expansion test. Géotechnique 1999;49:pages 133-141.

[14] Jardine RJ. Nonlinear stiffness parameters from undrained pressuremeter tests. Can Geotech J 1992;29:436–47.

[15] Whittle R, Palix E, Donaghy D. The influence of insertion process on determining the stiffness characteristics of chalk, using pre-bored, self-bored and pushed pressuremeters. Offshore Site Investig. Geotech., vol. 1, 2017, p. 308–15. https://doi.org/10.1145/2505515.2507827.

[16] Puech A, Garnier J. Design of piles under cyclic loading: SOLCYP recommendations. John Wiley & Sons; 2017.

[17] Borel S, Reiffsteck P. Caractérisation de la déformabilité des sols au moyen d'essais en place. Laboratoire Central des Ponts et Chaussées; 2006.

[18] Ferber V, Abraham O. Apport des méthodes sismiques pour la détermination des modules élastiques initiaux : application au site expérimental de Merville. PARAM 2002 - Symp. Int. Identif. détermination des paramètres des sols des roches pour les Calc. géotechniques, 2002, p. 8 pages.

[19] Benzaria O. Contribution à l' étude du comportement des pieux isolés sous chargements cycliques axiaux. Ph.D. thesis. Université Paris Est, 2013.

[20] Vardanega PJ, Bolton MD. Stiffness of Clays and Silts: Normalizing Shear Modulus and Shear Strain. J Geotech Geoenvironmental Eng 2013;139:1575–89. https://doi.org/10.1061/(ASCE)GT.1943-5606.0000887.