Determining Damping Ratio Through Pressuremeter Tests with Unload-Reload Loops

Détermination du taux d'amortissement par des essais pressiométriques avec des boucles de décharge-recharge

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ABSTRACT

Pressuremeter tests are an efficient tool for determining the shear modulus of ground and its variation with shear strain and stress. Recent research has introduced recommendations to account for the non-linear behaviour of ground during cavity expansion, enabling a straightforward assessment of both the maximum shear modulus and the rate of shear modulus decay as a function of shear strain. The interpretation relies on either analytical solutions or empirical formulations, accounting for stress and strain dependency. This paper extends these methods to determine the damping ratio of ground using pressuremeter tests with unload-reload loops. The proposed approach interprets the hysteretic behaviour of the unload-reload loops through semi-empirical techniques. The theoretical framework is outlined, and the methodology is demonstrated using real test data from reference testing sites. Validation is performed by comparing the results with reference values available in the literature for the tested ground.

RESUME

Les essais pressiométriques constituent un outil efficace pour déterminer le module de cisaillement du sol et sa variation en fonction de la déformation et de la contrainte de cisaillement. Des recherches récentes ont introduit des recommandations pour tenir compte du comportement non linéaire du sol pendant l'expansion de la cavité, permettant une évaluation directe du module de cisaillement maximal et du taux de décroissance du module de cisaillement en fonction de la déformation de cisaillement. L'interprétation repose sur des solutions analytiques ou sur des formulations empiriques, en tenant compte de la dépendance de la contrainte et de la déformation. Cet article étend ces méthodes pour déterminer le taux d'amortissement du sol à l'aide d'essais pressiométriques avec des boucles de décharge-recharge. L'approche proposée interprète le comportement hystérétique des boucles de décharge-recharge au moyen de techniques semi-empiriques. Le cadre théorique est décrit et la méthodologie est démontrée à l'aide de données d'essai réelles provenant de sites expérimentaux de référence. La validation est effectuée en comparant les résultats avec les valeurs de référence disponibles dans la littérature pour le sol testé.

Keywords: Non-linear behaviour; unload-reload loops; small strains; damping ratio.

1. Introduction

Pressuremeter tests are cylindrical cavity expansion tests which basically provide deformation and failure parameters for the ground. Many testing and interpretation procedures exist and are used for different purposes.

In French practice, the most used is the Ménard type (ISO 2021), where the cavity is loaded through a monotonous loading program, resulting in two main parameters: the so-called Ménard pressuremeter modulus E_M and the Ménard pressuremeter limit pressure p_{lM}. They are used in semi-empirical relationships to assess soil-structure interaction parameters for the design of shallow and deep foundations, both for the serviceability and limit states. Procedures are standardized and defined on the French national annexes of Eurocode 7, being largely accepted amongst practitioners.

Other pressuremeter testing procedures, including unload-reload loops, pressure hold periods, etc, are described in (ISO 2023). Those procedures are much vaster than the Ménard one and there is no strict interpretation procedure imposed or suggested. It is up to the engineer using the test to set up the method that best suits the project for which the tests are performed.

There is a robust experimental and theoretical background supporting the use of pressuremeter tests with unload-reload loops for the determination of the shear modulus of the ground taking into account its non-linear behaviour (Briaud et al 1983, Bolton and Wood 1990, Whittle 1999, Lopes 2020). This includes the determinations of shear modulus at very small strains (G_0), as well as the rate of shear modulus decay as a function of the shear strain (this rate is usually described by the parameter $\gamma_{0.7}$, which corresponds to the shear strain at which the secant shear modulus is equals to 72.2% of the initial shear modulus). Recent research focuses on the determination of these parameters (Habert and Lopes 2024).

While the non-linear elastic behaviour can be precisely characterized using pressuremeter tests, this paper explores the possibility to characterize material damping (hysteretic damping) from PMT tests with unload-reload loops.

Ground damping is a fundamental quantity in the study of vibratory phenomena. It is a parameter difficult to determine, frequently requiring complex laboratory testing (such as resonant column or cyclic triaxial tests), which might be subject to sample disturbance. The determination of this parameter through *in situ* tests is an important lever for ground investigation campaigns associated to major projects such as nuclear power plants and dams.

The subject has been studied by Mori and Tsuchiya (1981), as well as Dormieux (1988), assuming a viscous-plastic ground behaviour (viscous damping). Murthy (1992) studied the problem from a material damping perspective, using self-boring pressuremeters. Even though at that time the testing equipment limitations seemed to be the main obstacle regarding the determination of this parameter, the author concludes the pressuremeter has great potential for obtaining ground damping.

This paper deals with the subject from a material damping perspective, considering the hysteretic behaviour captured during unload-reload pressuremeter loops. The analytical developments regarding constitutive modelling proposed by Brinkgrieve et al (2007) are combined with the transformed strain approach for the interpretation of PMT loops (Lopes et al 2022) to derive the material damping ratio as a function of the shear strain of the ground.

2. Theoretical background

Concepts of ground damping are given by Pecker (1984) and the key aspects necessary to the theoretical developments within this paper are summarized in the following paragraphs.

The appearance of a hysteresis loop during a loading-unloading cycle (or loop) is a sign of energy dissipation within the material. The term *material damping* is used to describe the physical phenomenon of converting kinetic energy and potential energy (strain energy) into heat. Among materials that exhibit damping, two classes can be distinguished:

- Viscous damping: The dissipated energy depends on the strain rate (frequency for cyclic loadings). This is particularly the case for *linear viscoelastic materials* (such as polymers).
- Material damping: (hysteretic damping): The dissipated energy does not depend on the strain rate. These materials are characterized by significant non-linearities at high strain levels. The damping is attributed to plastic deformations at the level of the crystals or grains that make up the structure. Most metals and soils fall into this category.

Two quantities can be used to characterize the damping of a material: (1) the energy dissipated per cycle in the specimen, and (2) the ratio of this energy to a reference elastic energy. The last approach is most common.

In practice, it is usual to define the material damping by a dimensionless ratio, the *loss coefficient* η, which is defined as the ratio between the energy dissipated in a cycle (the whole surface containing the grey area in Figure 1), and 2π times the elastic energy stored, W (the surface of the shaded triangle in Figure 1).

$$\eta = \frac{D}{2\pi W} \tag{1}$$

Structures exhibiting viscous damping are usually modelled by a spring-damper model, such as Kelvin-Voigt. This type of model is characterized by the *critical damping ratio* β, which is a function of the natural frequency of the structure. At the resonance frequency (i.e. when the loading frequency is equals to the natural frequency), the critical damping ratio is:

$$\beta = \frac{D}{4\pi W} = \frac{\eta}{2} \tag{2}$$

With these considerations, the critical damping ratio (time dependent) is thus directly related to the loss coefficient (hysteretic dependent), enabling to characterize linear viscous-elastic models from static hysteretic measurements.

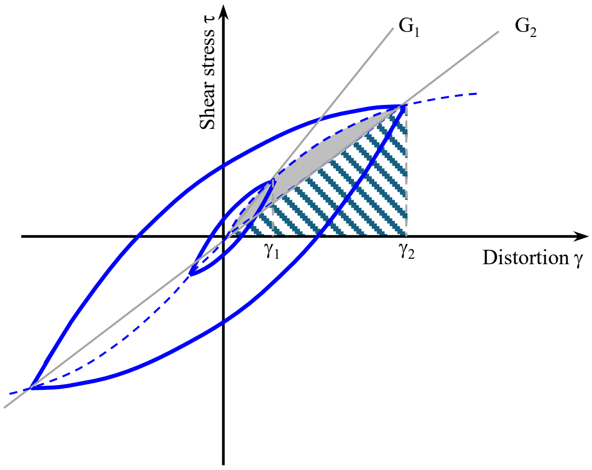

Figure 1. Hysteretic ground behaviour (Pecker 1984, adapted)

As it can be seen in Figure 1, ground response is highly non-linear. The energy dissipated in a cycle is thus dependent on the amplitude of the cycle, as well as the secant modulus G_s. Elastic modulus is maximum, and damping ratio is minimum at very small strains. Figure 2 schematically presents the variations of secant shear modulus and damping ratio as a function of shear strain.

For a given constitutive model, the shape of the hysteretic loop is well defined, and it is thus possible to express G_s, D and W as a function of distorsion γ, and so obtain an analytical expression for $\beta(\gamma)$. This was done by Brinkgrieve et al (2007) for the *Hardening Soil Model with Small Strain Stiffness*, usually called *HSSmall* model, which is a hyperbolic, strain hardening model, frequently used on Finite Element modelling for geotechnical projects.

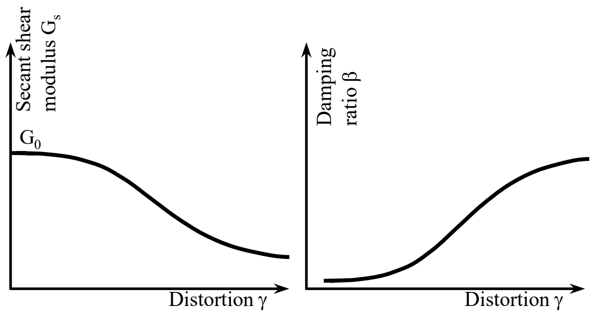

Figure 2. Variation of shear modulus and damping as a function of the shear strain (Pecker 1984, adapted)

3. Assumed ground behaviour

On the present work, we assume that ground stress-strain response can be modelled by hyperbolic model, similar to Hardin and Drnevich (1972) type model, and that the secant modulus can be described according to Eq. (3):

$$\frac{G_s}{G_0} = \frac{1}{1+d\frac{\gamma}{\gamma_{ref}}} \tag{3}$$

where G_0 is the initial shear modulus, G_s is the secant shear modulus associated with an certain shear strain level γ. Parameter d in Eq. (3) is related to the reference amount of secant modulus decay $d_{k,ref}$ obtained when the shear strain γ is equal to the reference shear strain γ_{ref}.

In the original Hardin and Drnevich (1972) formulation, $d = 1$, meaning that when $\gamma = \gamma_{ref}$, $\frac{G_s}{G_0} = d_{k,ref} = \frac{1}{2}$. Other authors formulate the equation differently, being usual to consider $d = 0.385$, meaning that when $\gamma = \gamma_{ref}$, $\frac{G_s}{G_0} = \frac{1}{1+d} = d_{k,ref} = 0.722$. This is the case of the constitutive model *Hardening Soil with Small Strain Stiffness* implemented in the commercial software Plaxis (Plaxis, 2019), that will be further used for the analysis in this paper.

In order to characterize soil behaviour in the small strain domain using this constitutive model, two parameters are required: the maximum shear modulus G_0 and the reference shear strain γ_{ref}. In this specific case, as $d_{k,ref} = 0.722$, γ_{ref} will be noted as $\gamma_{0.7}$. A stress dependency parameter m might also be required. In this paper, behaviour under pressuremeter loading will be considered as undrained and thus non stress dependent ($m = 0$).

For this specific constitutive model, Brinkgrieve et al (2007) established the following expressions for the energy dissipated in a cycle (D, Eq. 4) and for the elastic stored energy (W, Eq. 5), as a function of the shear strain γ_c reached during one cycle:

$$D(\gamma_c) = \frac{4G_0\gamma_{ref}}{d}\left(2\gamma_c - \frac{\gamma_c}{1+\frac{\gamma_{ref}}{d\gamma_c}} - \frac{2\gamma_{ref}}{d}\ln\left(1+\frac{d\gamma_c}{\gamma_{ref}}\right)\right) \tag{4}$$

$$W(\gamma_c) = \frac{1}{2}G_s\gamma_c^2 = \frac{G_0\gamma_c^2}{2+2d\gamma_c/\gamma_{ref}} \tag{5}$$

Equations (4) and (5) can be replaced in Eq. (2) to determine the critical damping ratio β for a given value of cyclic shear strain level γ_c, as well as to draw the whole curve $\beta(\gamma)$.

It should be noted that it has been observed that the analytical determination of the damping ratio curve usually yields values significantly higher than the damping ratios actually measured for soils. Authors such as (Hardin and Drnevich 1972) suggest that the damping ratio be limited to a given β_{max} value, which is a function of the nature of the ground. According to the authors, this is due to the fact that the actual ground behaviour is not perfectly hyperbolic. The authors suggest modifying the theoretical stress-strain relationship, considering soil-specific parameters.

Eq. (2) is modified by (Darendeli 2001), who proposes an adjusted equation for the damping ratio β_{adj}. According to the author, this function acts as a damping cap at very high strains. Parameter b, which is called the scaling coefficient, is, in a sense, the ratio of the measured damping to the damping value which is estimated from analytical / theoretical behaviour at intermediate strain amplitudes. β_{min} is added to the theoretical curve, as the analytical models are not able to capture damping at very small strains.

$$\beta_{adj}(\gamma) = b\left(\frac{G_s}{G_0}\right)^{0.1}\beta(\gamma) + \beta_{min} \tag{6}$$

4. Method of determination using pressuremeters

Pressuremeter tests with unload-reload loops enable the determination of the ground response at small strain levels, as well as the shear stiffness decay as a function of the shear strain. The interpretation procedures rely either on (1) semi-empirical approaches (strain and stress transformation) (Jardine 1992, Lopes et al 2022), or (2) analytical solutions (Ferreira and Robertson 1992, Habert and Burlon 2021). These procedures enable the determination of the elementary stress-strain response of the ground, and thus, the determination of parameters that describe it, such as G_0 and γ_{ref}.

For ground exhibiting undrained behaviour, both methods give similar results (Habert and Lopes 2024). Under drained conditions, analytical solutions are less or not developed, and the semi-empirical approach is straightforward. This paper will focus on the semi-empirical approach, whilst the analytical one will be developed in a coming paper.

Measurements performed with the pressuremeter at the cavity wall are an integration of the elementary behaviour of the surrounding ground. The strain transformed approach is a straightforward method which consists on determining the relationship between the strain measured at the cavity wall and the elementary corresponding strain, resulting in a same value of secant shear modulus.

The response measured at the cavity wall during an unload-reload loop (i.e. the relationship between the shear strain at the cavity wall γ_{cav} and the variations in the pressure at the cavity wall Δp_{cav}) can be considered as hyperbolic (Briaud et al 1983), and described by Eq. (7) :

$$\frac{\gamma_{cav}}{\Delta p_{cav}} = \frac{1}{G_{s,cav}} = a_0 + a_1\gamma_{cav} \tag{7}$$

Parameters a_0 and a_1 can be determined by hyperbolic fitting of the unload curve. It is thus straightforward to show that the maximum shear modulus is:

$$G_0 = 1/a_0 \tag{8}$$

Expressions for strain-transforming have been presented by (Jardine 1992, Lopes et al 2022). On this paper, reference will be made to the second work, which was established assuming that the nonlinear elastic behaviour of the soil is hyperbolic and that it can be described by a constitutive model respecting Eq. (3). Undrained conditions and no stress dependency ($m = 0$) were assumed.

Using finite element models of the cavity expansion problem, the authors have shown that under these hypotheses, the relationship between the shear strains at the cavity wall (γ_{cav}, measured with the pressuremeter), and the elementary shear strain γ for an equivalent shear modulus (constitutive equation), can be reasonably related with a proportionality coefficient C_L as per Eq. (9) and illustrated in Figure 3.

Eq. (9) presents the expression for the strain transformation coefficient according to (Lopes et al 2022), which also propose Eq. (11) for the determination of γ_{ref} based on the interpretation of the pressuremeter unload-reload loops.

$$\gamma_{cav} = C_L\gamma \tag{9}$$

$$C_L = 4.77 + 0.004\left(\frac{a_1}{a_0}\right) \tag{10}$$

$$\gamma_{ref} = d\,\frac{a_0}{a1}\frac{1}{C_L} \tag{11}$$

It is thus straightforward to see that the critical damping ratio from Eq. (2) can be determined based on parameters a_0 and a_1 determined from a pressuremeter unload loop: Eq. (8) is used to determine G_0, and Eq. (11) is used to determine γ_{ref}, both to be used in Equations (4), (5) and (6) to evaluate D and W for any given value of cyclic strain γ_c.

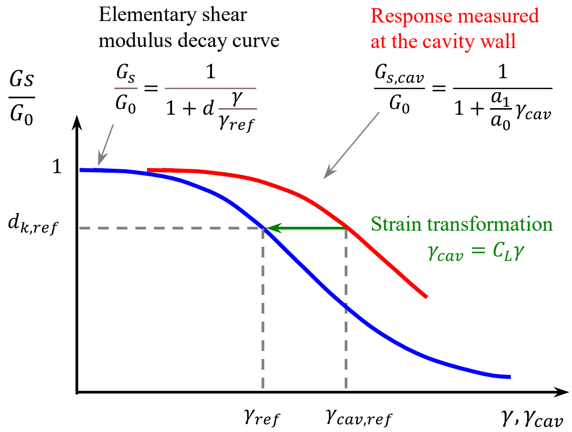

Figure 3. Principle of the strain transformation approach adopted in this work (Lopes et al 2022)

The simplicity of the solution is based on on the fact that there is consistency between the theoretical background used to interpret the pressuremeter test, and the constitutive model assumed for ground behaviour: both assuming a hyperbolic stress-strain relationship.

Based on parameters a_0 and a_1 determined from a pressuremeter unload loop, and a given strain transformation law C_L (here valid for undrained behaviour, assuming hyperbolic stress-strain relationship), we can then write the elementary shear modulus decay curve and the material damping ratio curve as a function of the shear strain, respectively, as per Eq. (11) and (12). Combining with Eq. (6) to include the cap proposal by (Darendeli 2001), Eq. (14) is obtained:

$$G_s(\gamma) = \frac{1}{a_0\left(1+\frac{\gamma\, a_1\, C_L}{a_0}\right)} \tag{12}$$

$$\beta(\gamma) = \frac{\frac{4}{a_1 C_L}\left(2\gamma - \frac{\gamma}{1+\frac{a_0}{a1}\frac{1}{C_L\gamma}} - 2\frac{a_0}{a1}\frac{1}{C_L}\ln\left(1+\frac{\gamma}{\frac{a_0}{a1}\frac{1}{C_L}}\right)\right)}{4\pi\frac{\gamma^2}{2a_0(1+\gamma a_1 C_L/a_0)}} \tag{13}$$

$$\beta_{adj}(\gamma) = b\left(\frac{1}{1+\frac{a_1}{a_0}C_L\gamma}\right)^{0.1} \cdot \frac{\frac{4}{a_1 C_L}\left(2\gamma - \frac{\gamma}{1+\frac{a_0}{a1}\frac{1}{C_L\gamma}} - 2\frac{a_0}{a1}\frac{1}{C_L}\ln\left(1+\frac{\gamma}{\frac{a_0}{a1}\frac{1}{C_L}}\right)\right)}{4\pi\frac{\gamma^2}{2a_0(1+\gamma a_1 C_L/a_0)}} + \beta_{\min} \tag{14}$$

Parameters b and $\beta_{\min}$ are dependent on the nature of the ground and cannot be determined only through pressuremeter tests. On this paper, the values have been adjusted to $b = 0.5$ and $\beta_{\min} = 2\%$ for a best fit of the representative empirical curves (Vucetic and Dobry 1991). These values are close to the ones presented by Darendeli ($b = 0.56$ and $\beta_{\min} = 1\%$).

5. Application to overconsolidated Flander's clays

On this paper, it is proposed to apply Eq. (13) to a pressuremeter test performed on overconsolidated clays. The quoted pressuremeter test has already been presented by (Lopes et al 2021), and here the goal is to extend the interpretation to include the analysis of the material damping ratio.

The quoted test was performed at 12m depth at the Merville reference testing site in France. Flander's clays found on the testing site are classified as very plastic. Atterberg's limit tests performed on specimens collected on the site resulted in plasticity index PI 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 (Borel and Reiffsteck 2006). At these same depths, CPT resistance, q_t, increases approximately from 1.5 MPa to 4.0 MPa, and undrained shear strength increases from 50 kPa to 150 kPa.

The pressuremeter test result is reproduced in Figure 4. A comparison with other assessments performed on the same site were presented by (Lopes et al. 2021).

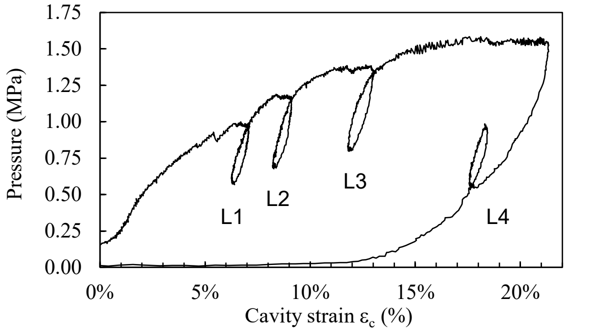

Figure 4. Result of the pressuremeter test performed at 12 meters depth in overconsolidated clays (Lopes *et al*, 2021)

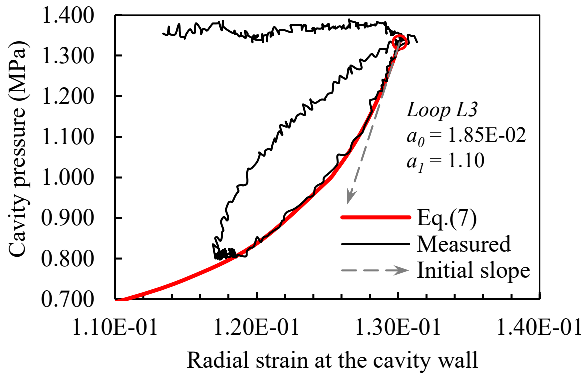

Figure 5. Detail of the loop L3: cavity response

Three unload-reload loops (L1 to L3) were performed in this test, all of them were interpreted according to Eq. 3, resulting in values of G_0, ranging from 53 to 59 MPa. As expected for the undrained cavity expansion and for the practical purposes of this paper, it can be considered that the ground response assessed with the pressuremeter is not dependent on the stress state changes during the test. An average initial shear modulus of 55 MPa can be assumed. The slight variation of modulus between the three loops can be attributed to experimental errors.

Table 1. Parameters a_0 and a_1 obtained for the unload loops L1 to L3 in the PMT test performed from Figure 4.

Determined from measurements				*Transformed strain*	
Loop	a_0	a_1	$G_0 = 1/a_0$ (MPa)	C_L	γ_{07}
L1	1.90E-02	1.68	53	5.12	8.5E-04
L2	1.68E-02	1.20	59	5.06	1.1E-03
L3	1.85E-02	1.10	54	5.01	1.3E-03

Figure 6 presents the interpretation of the loop L3 and the corresponding curves $G(\gamma)$ and $\beta(\gamma)$ determined using Eq. (12) and (14) for the values of a_0 and a_1 determined from this loop. On this Figure, the dashed lines represent the whole modelled behaviour, while the continuous lines represent the actual measurement domain (distortion γ between $8 \cdot 10^{-4}$ and $2 \cdot 10^{-2}$).

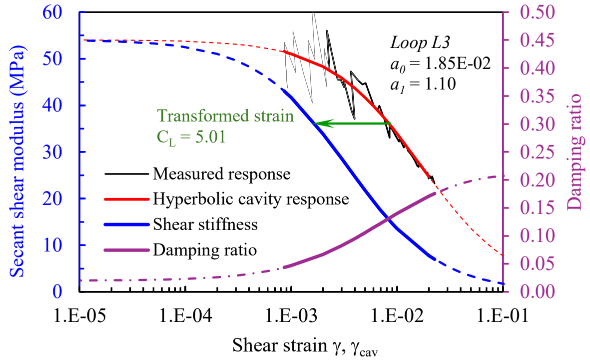

Figure 6. Interpretation of loop L3. Continuous lines correspond to the measured domain while the dashed lines extrapolate it

Figure 7 presents a comparison between the shear stiffness and damping ratio curves determined with the pressuremeter and the empirical curves for OC clays proposed by (Vucetic and Dobry 1991).

The curves obtained with the pressuremeter are representative of the behaviour expected for a range of plastic clays. The determined curve sits near the empirical curve for Plasticity Index IP = 50% on the graphs, approaching the IP = 100% at smaller strains. This is actually consistent with what is expected for the Flander Clays, for which plasticity index IP ranges from 40% to 69%, confirming the potential of the approach.

It is important to note that the adjustment proposed by (Darendeli 2001) to the damping ratio (Eq. (6)) is of major importance for the success of the method. Indeed, between Eq. (13) and (14), there is an important offset attributed to the coefficient b. The magnitude of this offset is of about 2 in the present case. This is illustrated in Figure 7 which presents the damping ratio curves with and without cap (thus, as per Eq. (14) and (13), respectively).

Thus, despite the capability of the method to derive damping ratio curves based on the constitutive equations of the ground, the capping Eq. (6), purely empirical, plays a major role. The validation of the method for other types of soils requires a further calibration and validation of the parameter b.

It should be noted that the work from (Brinkgrieve et al 2007), which is directly comparable to Eq. (13), does not include the damping cap proposal quoted herein: for this reason, the authors obtained much higher values of damping than the observed values in literature. This overestimation is due to the fact that the real ground response is not perfectly hyperbolic, and that there exists some damping even at very small strains (captured here by parameter β_{min}), and that there should be a maximum damping ratio depending on the ground nature (captured here by parameter b).

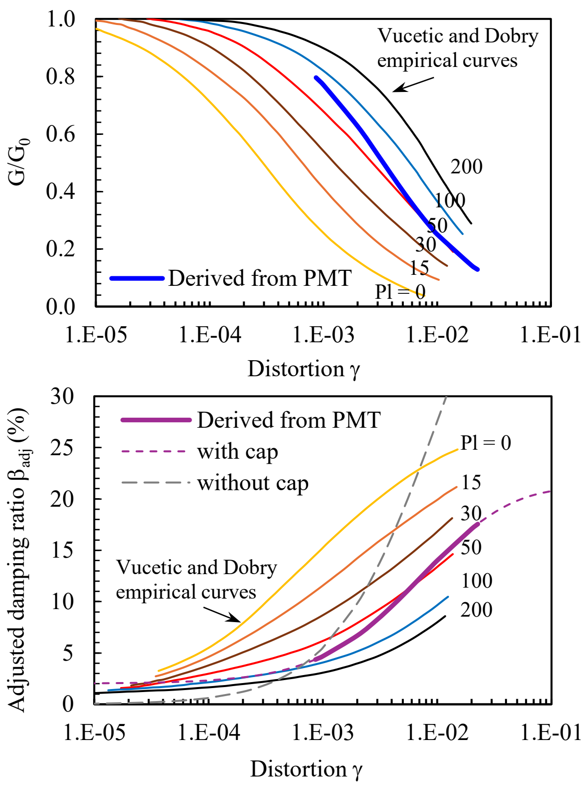

Figure 7. Comparison between the shear stiffness and damping ratio curves determined with the pressuremeter and literature values for OC clays (Vucetic and Dobry 1991)

6. Conclusions

This paper presents a new method to determine ground material damping ratio from pressuremeter tests with unload-reload loops. The approach consists of determining the area of the hysteresis loop, which is associated with the material damping.

The method presented herein is based on semi-empirical strain transformation approach, calibrated for soils exhibiting undrained behaviour, and a hyperbolic stress-strain relationship described by the maximum shear modulus G_0 and the reference shear strain γ_{ref}.

While the shear modulus decay curve determined from PMT loops has been previously shown to be consistent with other measurement methods, this paper presents for the first time a comparison between damping ratio evaluated in this manner.

Validation of the proposed method was carried out by comparing the obtained curve with empirical damping ratio curves from the literature. Great agreement was obtained for the overconsolidated Flander's clays. It is important to note, however, that the analytical interpretation of the unload loop only, is not sufficient. The damping ratio curve needs to be capped and scaled, as per the proposition by (Darendeli 2001), to avoid to obtain overestimated values.

The innovative procedure presented herein highlights the potential of the pressuremeter as a powerful in situ testing device, capable of delivering advanced ground parameters that have traditionally been assessed only through laboratory investigation. This opens new perspectives for more efficient and representative geotechnical characterization.

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