Stiffness-and damping-strain curves of sensitive Champlain clays through experimental and analytical approaches

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Abstract:

Stiffness degradation curves of Champlain clay at St-Adelphe, Québec and the associated variation of its damping ratio with shear strain are constructed in this paper using the new combined triaxial simple shear (T_xSS) apparatus. The apparatus offers the ability to obtain the stiffness and damping ratio of soils over a wide range of strain spectrum from 0.001% to 10%. The value of the small-strain stiffness of the tested clay is further confirmed through another series of piezoelectric ring-actuator technique (P-RAT) tests. Although, the stiffness degradation curve of the tested clay follows to some extent traditional curves suggested in literature, the examined Champlain clay exhibits different trend with respect to hysteresis damping especially at large strains ($>1\%$) and available analytical models couldn't successfully predict the damping behavior of the Champlain clay at such strain level. A new constitutive model is therefore presented as a modification of the original Sig4 model considering the pore water pressure built-up with shear strain. Stiffness degradation and damping ratio versus shear strain curves of Champlain clays estimated using the proposed soil model are compared successfully with their experimentally-determined counterparts even at large shear strains where other models tend to misjudge the damping behavior of the clay.

Keywords: T_xSS ; P-RAT; sensitive Champlain clay; Sig4 model; damping ratio; pore water pressure.

Résumé

Les courbes de dégradation du module de rigidité, $G/G_{\max}$, de l'argile de la mer de Champlain prélevé à St-Adelphe (Québec) et la variation associée de son taux d'amortissement avec la distorsion sont construites dans cet article en utilisant le nouvel appareil triaxial à cisaillement simple (TxSS). L'appareil offre la possibilité d'obtenir le rapport de rigidité ($G/G_{\max}$) et d'amortissement des sols sur une large gamme de déformation 0,001% à 10%. La valeur de la rigidité à faible déformation de l'argile testée est confirmée par une autre série de tests à l'aide de la technique de l'anneau piézo-électrique (P-RAT). Bien que la courbe de dégradation du module de rigidité de l'argile testée suive, dans une certaine mesure, les courbes classiques suggérées dans la littérature, l'argile étudiée présente une tendance différente en ce qui concerne l'amortissement hystérétique, surtout pour les grandes déformations (>1%). De plus, aucun model numérique existant ne permet de reproduire le comportement observé à un tel niveau de déformation. Un nouveau modèle constitutif est donc présenté comme une modification du modèle original Sig4 en tenant compte de la pression de l'eau interstitielle accumulée durant les déformations de cisaillement. La dégradation du module de cisaillement et l'amortissement en fonction de la distorsion estimé en utilisant le modèle proposé sont similaire même à de grandes déformations aux courbes de rigidité et d'amortissement déterminés expérimentalement contrairement aux autres modèles qui ont tendance à mal juger le comportement de l'amortissement de l'argile.

Introduction

The deformation characteristics of soil under cyclic and dynamic shear conditions are very nonlinear, and this is manifested in the degradation of its shear stiffness modulus, G and the associated variation of its damping ratio, $\xi\%$ with shear strain, γ , accompanied, in some cases, by the build-up of excess pore water pressure, R_u (e.g., Kramer 1996, Ishihara 1996). These strain-dependent variations of dynamic soil characteristics (i.e., G and $\xi\%$) have, in fact, a predominant role on the seismic site response as it substantially alters ground motion characteristics (i.e., amplitude, and frequency content). For this particular reason, extensive research has been carried out by several researchers to study these characteristics of various soils at different sites, chiefly through laboratory tests on highly undisturbed samples regarded as representing intact conditions in the field. The bender element (BE) (e.g., Brignoli et al. 1996; Yamashita et al. 2009; Clayton 2011), and the piezo electric (e.g., Éthier 2009, Karray et al. 2015) devices have been commonly utilized to evaluate low-strain dynamic soil properties. On the other hand, the resonant column (RC) is used to characterize the dynamic properties of soils from low to intermediate strain region, $\gamma=0.0001\%$ to 0.05% (e.g., Hardin 1970; Hardin and Drnevich 1972; Drnevich et al. 1978; Tatsuoka et al. 1978, Goudarzi et al. 2017) and therefore the stiffness degradation and damping versus strain ($G/G_0 - \gamma$ and $\xi\% - \gamma$, where G_0 is the maximum shear stiffness of soils) curves are obtained from RC experiment up to intermediate strain level. The other laboratory methods, direct simple shear (DSS) (e.g., Roscoe 1953; Bjerrum and Landva 1966; Boulanger et al. 1993; Lanzo et al. 1997; Wijewickreme and Soysa 2016), cyclic triaxial (CTX) (e.g., Peacock and Seed 1968; Kokusho 1980; Simcock et al. 1983; Gu et al. 2017), and cyclic torsional shear (e.g., Iwasaki et al. 1978; Bhatia et al. 1985) apparatuses are applicable to characterize

dynamic properties of soils for wider strain range, up to large strain level. In fact, RC and BE methods of measuring shear wave velocity, V_s and consequently G_0 ($G_0 = \rho V_s^2$, where ρ is the soil density) have their own difficulties such as the boundary and near field effects, the mixed radiation of both compression, P- and shear, S-waves, and the uncertain detection of first arrivals (Éthier 2009; Karray et al. 2015). Bedding errors and system compliance as well as membrane penetration effects in the CTX generally limit its measurements to shear strains greater than 0.01% (e.g., Ishihara 1996). The bulging of the rubber membrane due to generated pore water pressure and the lack of complementary shear stress represent the most significant deficiencies in the DSS devices that would of course affect their reliability (e.g., Bhatia et al. 1985; Boulanger et al. 1993; Hussien et al. 2015). Although, they are more versatile and very useful in investigating basic aspects of soil deformation characteristics, the torsional shear devices (cylindrical or hollow cylindrical) specimen are not suitable for practical purposes (Ishihara 1996). In addition, the use of two or more different laboratory techniques to construct a single ($G/G_0 \square \gamma$) or ($\xi\% \square \gamma$) curve of a given soil raises doubts about the coherence of the results and represents a great source of experimental uncertainties and arises a question whether the modulus reduction and damping curves thus obtained could indeed reflect actual behavior under field conditions.

On the other hand, the nonlinear formulations of transient soil behavior typically constitute the adjustment of soil stiffness and its damping ratio to the instantaneous levels of strain and loading path according to the mathematical description of nonlinear stress-strain model and hysteretic (loading and unloading) soil response. There are several nonlinear soil models, in literature, ranging from

relatively simple cyclic stress-strain relationships (e.g., Ramberg and Osgood 1943; Kondner and Zelasko 1963; Finn et al. 1977; Pyke 1979; Vucetic 1990) to advanced constitutive models incorporating yield surfaces, hardening laws, and flow rules (e.g., Dafalias and Popov 1979). In fact, advanced constitutive models are able to capture important features of soil behaviour such as anisotropy, pore water pressure generation, and dilation among others. Nonlinear models can be formulated so as to describe soil behaviour with respect to total or effective stresses thus allow the modelling of generation and dissipation of excess pore pressure during and after earthquake shaking (Stewart et al. 2008). Most of the available nonlinear soil models have a set of rules to define the initial loading behaviour (so-called backbone or skeleton functions) then to construct the unloading and reloading branches according to the Masing (Masing 1926) or the extended Masing rules. These models rely on backbone functions that have fixed formulas with a set number of experimentally calibrated parameters. Hyperbolic functions such as the original Kondner-Zelasko (KZ) (Kondner and Zelasko 1963) and the modified Kondner-Zelasko (MKZ) (Matasovic 1993) have been widely used to define the stress-strain backbone curve of soil. These functions are asymptotic to G_0 and maximum shear stress, τ_{max} at zero and infinite strains, respectively. Despite their simplicity, hyperbolic functions have some limitations reported in the literature (e.g., Santos 1999; Lourenço et al 2017) : i) it is difficult, in some cases, to specify both the G and $\zeta\%$ by means of only two parameters (i.e., G_0 and τ_{max}); ii) while the model representation is satisfactory in the range of small strains, it tends to deviate from actual behaviour of soils with increasing shear strains, thereby overestimating the damping ratio.; and iii) no more strength can be mobilized when shear strains tend to infinity. A number of modifications of the hyperbolic formulations were proposed to improve the fit with

available experimentally derived shear modulus reduction curves. For example, Fahey and Carter (1993) adopted a quasi-hyperbolic relation written in terms of shear stress rather than shear strain, and employing an exponent to adjust the shape of the curve. Hashash and Park (2001) introduced a new hyperbolic formulation in which the reference strain ($\gamma_{ref} = \tau_{max} / G_0$) is no longer a constant for a soil type, but a variable that depends on the effective stress. However, the model has the same limitations as the MKZ model regarding controlling for the maximum shear stress. Based on statistical analyses of resonant column and torsional shear test results from 122 specimens, Zhang et al. (2005) suggested raising the normalised shear strain in the original hyperbolic model to a power in order to better fit the data at small strains. These models tend, in fact, to give reasonably accurate results with respect to the shear modulus degradation with shear strain. However, several of these models sacrifice the capability of correctly modeling the soil damping behavior especially at large shear strains as the use of Masing rule results in hysteresis loops that are too large. This is a significant shortcoming when using these models in site response analyses where large strains are anticipated. Other models have been developed which address some of these issues by either a bounding surface plasticity approach (e.g., Borja and Amies 1994; Boulanger et al. 2011) or a hybridized combination of models and transition functions (e.g., Yee et al. 2013; Gingery and Elgamal 2013). For example, a damping reduction factor that reduces the size of hysteresis loops was proposed by Darendeli (2001) as a way to achieve damping curves that showed good agreement with those obtained from laboratory tests. Phillips and Hashash (2009) presented an alternative formulation of the reduction factor and a non-Masing unloading-reloading formulation for use with the original MKZ model. Sigmoidal (Sig3 and Sig4) (e.g., Itasca 2008) and hybrid hyperbolic (HH) (e.g., Shi and Asimaki

2017) models have been developed to predict the response of soils to cyclic loading over a wide range of strains. Such hysteresis models are capable of simultaneously matching shear modulus and damping data, thus resolving the issue of damping overestimation at higher strains that has been the major drawback of earlier rules. Nevertheless, even when such modified rules are implemented, the numerical scheme cannot accurately represent the actual damping behavior of certain soil types at larger strains if the stress–strain model is not flexible enough to fit the damping data.

In this paper, the stiffness degradation curves of sensitive Champlain clay at St-Adelphe, Quebec are constructed for the first time through a series of laboratory tests using the new combined triaxial simple shear (T_xSS) apparatus (Chekired et al. 2015). This apparatus offers the ability of measuring the soil stiffness over a wide range of strain spectrum from 0.001% to 10%, thus reduce the difficulties and the sources of uncertainty associated with other laboratory methods of constructing G/G_0 curves. The values of the small strain stiffness G_0 of the sensitive Champlain clays have been confirmed through another series of experimental tests using the piezoelectric ring-actuator technique (P-RAT) (Éthier 2009; Karray et al. 2015). This paper, then, proposes a modification for the Sig4 model to take into account the pore water pressure built up with shear strain. This modification will improve the performance of Sig4 model to characterize the dynamic response of the Champlain clay in comparison with the original model.

Description of the St-Adelphe site and soil conditions

The site is located in St-Adelphe, Quebec where there was a landslide following the Saguenay earthquake in 1988. The site has been the subject of several geotechnical investigations (e.g., Lefebvre et al. 1992). The geotechnical investigation conducted

by the Ministry of Transport, Sustainable Mobility and Transportation Electrification of Quebec in 2015 using continuous thin-walled tube soil sampling until refusal revealed that no silt or sand layer in the clay was detected at the sites except in the lower 0.5 m before the refusal. The clay has a plasticity index (PI) varying between 20% and 30% and a liquidity index, (LI) around unity down to a depth of about 3 m. The sensitivity, S_t is of the order of 20 down to 3 m deep and rapidly increases with depth. Between 7 and 11 m deep, where the water content is around 70%, the LI varies between 4 and 5 and the S_t is more than 100. The clay-size portion is around 65% except close to the clay-till contact where it decreases to 12%. The vane shear strength, between 3 and 8 m deep, is between 20 and 25 kPa, increases to 35 kPa at 9.5 m deep. The mineral characteristics of the Champlain clay are shown in Table 1 (Lebuis et al. 1982; Lerouiel et al. 1983).

Specialized laboratory apparatuses

The T_xSS apparatus

The cyclic triaxial simple shear test (T_xSS) apparatus (Chekired et al. 2015), employed in this study, was designed and manufactured to test cylindrical soil specimens with a diameter of 63-64 mm and 79-80 mm and varying heights in a triaxial pressure cell. Unlike the DSS apparatus that requires the soil specimen to be prepared in stacks of annular plates/rings or reinforced membrane, the T_xSS specimen is installed between relatively rigid bottom and top caps and is typically confined by a rubber membrane as shown in Fig. 1. The bottom and top caps that contain fine porous stones with spics provide a “frictional” surface while allowing for drainage into the porous stones. The clay specimen is consolidated to a desirable confining stress, and then simple shear strain is presumed to be imposed by displacing the

specimen's top cap using a shear ram connected to a shaker with linear shaft motor mounted on a horizontal table. The motor is characterized by its high-thrust capability (up to 100,000 N), its high precision (0.07nm), and its very low speed fluctuation ($\pm 0.006\%$ at 100 mm/s). It is very quiet due to the absence of friction (non-contact operation is also possible using air slider). A computer-automated feedback-loop-controlled system provides an excellent control of stresses and strains. The T_xSS system permits testing soil samples with different heights under either drained or undrained conditions as well as the direct measurement of the pore water pressure generation during the undrained shear test. It also provides the opportunity of testing undisturbed and reconstituted soil samples under either isotropic or anisotropic loading conditions.

It should be mentioned here that the connections between the soil sample and the top and base platens as well as the membrane would deviate the behaviour of the tested soil sample from its true behaviour especially at low level of shear strain. To reduce the effect of these losses on the measured response of the tested soil samples, the following procedures have been adopted:

- The lateral deformation of the soil sample has been directly measured as the displacement of the top platen excluding the mechanical losses of the connections between the sample and the actuator. In other conventional apparatuses, the deformation produced from the actuator is adopted as the soil deformation without excluding the connections losses. In the current study, the deformation of the soil sample has been monitored from the other side of the triaxial cell. In other words, it has been measured from the side of the cell opposite to the loading actuator side.

- Very rigid base and top platens have been used with vertical axis inserted deeply into the top cap (19 mm) and tightly fixed with it to ensure that there is no significant rocking is produced and consequently there are no significant losses. The top and base platens were also rough enough to transfer the shear stress to the tested soil sample as recommended by ASTM (D6528-17).
- Flexible membrane instead of armed membrane has been utilized to reduce the potential losses generally produced from the membrane.

In addition, values of the measured maximum shear moduli of tested soils have been confirmed through another series of piezoelectric ring-actuator technique (P-RAT) laboratory tests as it will be presented next.

The P-RAT technique

The piezoelectric ring-actuator technique (P-RAT) developed in the geotechnical laboratory at the Université de Sherbrooke (e.g., Éthier 2009; Karray et al. 2015) has been utilized in this study to verify the small-strain T_xSS results (i.e., G_0). The P-RAT has been incorporated into the conventional oedometer apparatus and the procedure is identical to that of the oedometer with the measurement of the soil shear wave velocity (V_s) to construct the relationship between the normalized shear wave velocity (V_{sl}) of the soil and its void ratio (e). V_{sl} can be estimated as (Youd et al. 2001):

$$V_{sl} = V_s (P_a / \sigma'_v)^{0.25} \quad (1)$$

where σ'_v is the effective vertical stress, P_a is normal atmospheric pressure in the same units as σ'_v (i.e., $P_a \approx 100$ kPa if σ'_v is in kPa).

Experimental results and discussions

Typical shear stress-shear strain hysteresis results of the cyclic T_xSS test on St-Adelphe Champlain clay samples extracted from a depth (4.17–4.29 m) with OCR of 2.6, PI of 21%, and $S_r=63$ are presented in Fig. 2. In the T_xSS tests, the St-Adelphe specimen is consolidated under a vertical stress (σ'_v) of 60 kPa and σ'_h of about 40 kPa. The confining pressures represent the in-situ effective stress that the soil experience at the depth where the samples are extracted. The coefficient K_0 was calculated using the correlation proposed by Schmidt (1966) - $K_0=(1-\sin\phi)OCR^{\alpha=\sin\phi}$, with $\phi'=34^\circ$.

Once the consolidation is completed, the drainage line is closed and the specimen is cyclically sheared under strain-controlled condition with a cyclic frequency of 1 Hz and very small strain amplitude (on the order of $\pm 0.004\%$) for six cycles without drainage. It has been observed, at such very small strain, that there is no significant built up of the pore water pressure inside the clay sample. Therefore, the same clay sample has been further tested under larger shear strain amplitudes (on the order of $\pm 0.006\%$, $\pm 0.008\%$, and $\pm 0.01\%$), similarly for six cycles without drainage and at 1Hz loading frequency. The excess pore water pressure built up, during these tests/strain levels, is also insignificant though the soil sample has directly tested under consecutive applied shear strains. For tests conducted utilizing shear strains beyond this shear strain threshold ($\gamma_{th} = \pm 0.01\%$), the pore water pressure has been slightly built up with the application of the strain cycles and this has been continued up to a shear strain amplitude of $\pm 0.69\%$. In this range of applied shear strains (between $\pm 0.01\%$ and $\pm 0.69\%$), the drainage valve has been opened, between consecutive applied shear strains, and the clay specimen has been allowed sometime to reach an

equilibrium state. The stress-strain hysteretic loops obtained under a maximum strain level up to $\pm 0.69\%$ are, in fact, an average of 6 cycles applied to the clay specimen. Because the observed built up of the pore water pressure becomes significant ($R_u = 0.05$) at shear strain of $\pm 0.69\%$, the T_xSS tests of shear strains larger than $\pm 0.69\%$ (i.e., $\pm 0.877\%$, $\pm 2.0\%$, and $\pm 3.25\%$) have been performed on three other new clay samples extracted from the same depth and have typical physical and mechanical properties as the originally-tested St-Adelphe clay sample. The hysteretic stress-strain relationships determined from the cyclic T_xSS tests on these clay samples have been portrayed also in Fig. 2, as the soil response of only one cycle of applied strain. Each hysteretic loop in Fig. 2 is, in fact, an average of 6 cycles applied to the specimen at the same max strain amplitude ($\pm 0.004\% - \pm 3.25\%$). As shown in Fig. 2, a gradual increase in the loop area is observed with the increase in the applied cyclic shear strain. The secant shear moduli corresponding to different strain levels in Fig. 2 can be defined as $G = \tau/\gamma$, where τ = the cyclic stress amplitude corresponding to the max applied shear strain amplitude, γ . At very small applied shear strains ($\gamma = 0.004\%$) portrayed in the upper left plot in Fig. 2, the secant shear modulus, G is approximately equal to the small-strain shear modulus, G_0 of 16 MPa. The area enclosed in a hysteresis loop corresponding to a given shear strain is used to define the equivalent material hysteresis damping ratio, $\xi\%$ at this strain. The successive hysteresis stress-strain loops of St-Adelphe clays are plotted in the graph at the lower-right corner of Fig. 2. This plot confirms the general compatibility between the sequential stress-strain loops from a very low to large strain level. This result affirms the reliability of the used T_xSS to measure the Champlain clays stiffness over a wide range of strain spectrum, thus reduce the difficulties and the sources of uncertainty associated with traditional methods of constructing similar stiffness depredation curves.

In addition, the experimental hysteresis stress-strain relationships determined from the cyclic T_xSS tests have been matched as shown also in Fig. 2 using the sigmoid function (Sig4 soil model) available in the FLAC library. As mentioned above, the Sig4 model is one of the soil models that have been proposed to predict the cyclic soil response over a wide range of strains. These models are also capable of simultaneously matching shear modulus and damping data, thus reducing the issue of damping overestimation at higher strains. In fact, the Sig4 soil model is a 4-parameters model thus it provides more flexibility/control in fitting the stiffness and damping data compared to other models. The backbone curve of the Sig4 soil model is given by:

$$F_{bb}(\gamma) = \tau = G_0 \left[y_0 + \frac{a}{1 + \exp(-(L_y - x_0)/b)} \right] \gamma \quad (2)$$

where, L_y is $\log_{10}(\gamma(\%))$, and a, b, x_0 and y_0 are four curve-fitting parameters. For best matching the experimental data these parameters are, respectively selected at 0.91, -0.45, -1.445, and 0.12. Up to shear strain level of $\pm 0.877\%$, Fig. 2 shows that the Sig4 model generates hysteretic shear stress-strain relationships that match reasonably well the dynamic behavior of the Champlain clay measured in the laboratory. For larger shear strains ($\gamma = \pm 2\%$ and $\pm 3\%$), the simulated stress-strain loops are very different from the measured loops. In fact, the Sig4 model overestimates the soil secant shear moduli and underestimates its hysteretic damping ratios at $\gamma = \pm 2\%$ and $\pm 3\%$. This can be revealed from the backbone curve obtained from the Sig4 model that coincides quite well with the experimental backbone curve up to strain level, of $\pm 0.877\%$, and begins to deviate from it beyond this shear strain limit. In other words, the Sig4 model successfully simulates the dynamic characteristics of Champlain sensitive clay in shear strain level up to $\gamma = \pm 0.877\%$ and

fails to reproduce dynamic soil characteristics at shear strain beyond this shear strain limit.

Both experimental and analytical results presented in Fig. 2 demonstrate that the small-strain shear modulus, G_0 of the St-Adelphe Champlain clay in question is on the order of 16 and 16.9 MPa, respectively. This value has been further verified through $V_s - e$ curves constructed using the P-RAT at the value of initial void ratio ($e = 1.7$) of the clay specimen tested in the T_xSS . Typical consolidation curves of undisturbed sample of St-Adelphe clays are shown in Figs. 3a and 3b. In particular, Fig. 3a presents traditional $\sigma'_v - e$ consolidation curve, while Fig. 3b presents $\sigma'_v - V_s$ consolidation curve. Ideal consolidation curves with easy-identified pre-consolidation pressure (σ'_p) of 127 kPa have been obtained from Fig. 3. Fig. 3b indicates also that $\sigma'_v - V_s$ curve can be used as an analogy to $\sigma'_v - e$ curve in the identification of the soil consolidation behaviour especially in determining the pre-consolidation pressure, σ'_p . As shown in Fig. 3c, $V_{s1} - e$ relationship of St-Adelphe clay indicates the strong dependence of the obtained V_{s1} on the void ratio in agreement with established results in literature (e.g., Schanz et al. 2016; Hussien and Karray 2016, Karray and Hussein 2017), and the $V_{s1} - e$ relationships were obtained from the tests are:

$$V_{s1} = 126.7OCR^{-0.452}e^{0.15} \text{ (Samples extracted from 4.17-4.29 m and 5.71-5.93 m)} \quad (3)$$

$$V_{s1} = 119.4OCR^{-0.53}e^{0.15} \text{ (Samples extracted from 8.09-8.17m)} \quad (4)$$

From Figs. 3a - c, the St-Adelphe sample extracted from 4.17- 4.29 m has a void ratio e of 1.7 and V_s of 97 m/s at a confining pressure σ'_c of 60 kPa (the confining pressure under which the soil sample has been tested in the T_xSS apparatus). Given that the St-Adelphe soil sample has a density of 17kN/m^3 , the small strain shear

modulus G_0 estimated from the P-RAT results will be 15.64 MPa which is very close to that estimated from the T_xSS small-strain results (16 MPa) and that calculated from Sig4 model (16.9 MPa) shown in Fig. 3d.

The influence of γ on G (the degradation of the shear modulus) and on $\xi\%$ can be then evaluated from the T_xSS results. In other words, both G and G_0 were measured in the T_xSS laboratory tests and divided to obtain the ratio G/G_0 . Figures 4a and 4b show, respectively the variation of the normalized shear modulus (G/G_0) and damping ratio ($\xi\%$) with shear strain obtained from the T_xSS results on the St-Adelphé Champlain clay samples under loading frequencies of 0.1 and 1.0 Hz. G/G_0 degradation and damping curves suggested by Vucetic and Dobry (1991) for clays with $PI = 0, 15, 30, 50$, and 100 as well as those obtained from the Sig4 analytical procedure are plotted in Figs. 4a and 4b for comparison. To plot the damping ratio using the Sig4 soil model, the initial Sig4 stress-strain curve $F_{bb}(\gamma)$ given in Eq. 2 is used in conjunction with the Masing criteria to analytically express the equivalent viscous damping ratio $\xi\%$. And thus the behavior of the soil can be characterized by a secant shear modulus, (G/G_0) , and an equivalent viscous damping ratio $\xi\%$ as:

$$\left. \frac{G}{G_0} \right|_{\gamma=\gamma_a} = \frac{\tau_a}{G_0 \gamma_a} = y_0 + \frac{a}{1 + \exp(-(L_y - x_0)/b)} \quad (5-a)$$

$$\xi \Big|_{\gamma=\gamma_a} = \frac{2}{\pi} \left[\frac{2 \int_0^{\gamma_a} F_{bb}(\gamma) d\gamma}{F_{bb}(\gamma_a) \gamma_a} - 1 \right] \quad (5-b)$$

In which γ_a = reversal shear strain and τ_a = reversal shear stress. From equation (5-b), $\xi\%$ depends exclusively on the shape of the $(G/G_0 - \gamma)$ curve, but is independent of G_0 . Figures 4a and 4b show that there is no practical effect of the loading frequencies

on the dynamic characteristics of the sensitive Champlain clay in question. Both experimental and analytical data shown in Fig. 4 clearly demonstrate the dependency of both G and $\xi\%$ on the strain level γ . More specifically, the data plotted within a wide range of strain spectrum from 0.001% to $\pm 3\%$ shows that the G/G_0 curve of St-Adelphe clay indeed tends to move down, while the $\xi\%$ curve tends to move up as γ increases. The experimental (G/G_0 - γ) data generally fall within the range suggested by Vucetic and Dobry (1991). On the other hand, the damping ratio of the sensitive Champlain clays exhibit different trend with respect to γ . For shear strain γ up to $\pm 0.3\%$, the experimental $\xi\%$ - γ curve of the clay ($PI=21$) follow typical curve of clays ($PI=15$) given by Vucetic and Dobry (1991). Beyond this strain ($\pm 0.3\% < \gamma < \pm 0.877\%$), the experimental $\xi\%$ slightly decreases with γ . Similar reduction of damping ratio with shear strain has been reported by EPRI (1993), and Chiaradonna et al. (2015) and is attributed by Matasovic and Vucetic (1993) to the dilative behavior of soils at such strains. Up to this strain level ($\gamma = \pm 0.877\%$), Figs. 4a and 4b shows that the Sig4 generally produces G/G_0 - γ and $\xi\%$ - γ curves that match quite well the experimental curves. For larger strains ($\gamma > \pm 0.877\%$), the experimental $\xi\%$ of the St-Adelphe clay re-increases with the increase of γ following the trend suggested by Vucetic and Dobry (1991) at PI of 100 (i.e., the theoretical curves of Vucetic and Dobry (1991) overestimate the damping behaviour of sensitive Champlain clay at higher strains, which in turn could lead to an underestimation of ground response to very strong input motions), while the $\xi\%$ from Sig4 model continues to decrease with γ . The later result implies that adopting analytical $\xi\%$ - γ curves directly from the Sig4 model in the seismic response analyses of Champlain clays deposits would amplify the ground response at relatively large strains ($\gamma > \pm 0.877\%$).

Modeling the cyclic behavior of Champlain clays

The inability of Sig4 model to correctly capture the damping behavior of Champlain clay at a strain level greater than $\pm 0.877\%$ despite the relatively good concordance between the damping and the stiffness degradation curves is, in fact, an important limitation of the model. Therefore, the development of a new model capable of better simulating the cyclic stress-strain behavior of the soil appears important.

To account for this aspect of strain dependent soil behavior, the mathematical Sig4 model should be refined. To this end, the effect of the parameter y_0 (one of the four parameters of the Sig4 model given in Eq. 5a) on the variation of the damping ratio has been discussed as shown in Fig. 5a. For initial Sig4 modelling (the reference curve) where y_0 was assumed to be constant of 0.12, the $\xi\%$ gradually increases with shear strain up to a shear strain of 0.15 then it decreases with further increase in the shear strain. Figure 5a shows that the reduction of the parameter y_0 significantly affects the shape of the $(\xi\% - \gamma)$ curve. In other words, it can be noticed that the reduction of the parameter y_0 leads to a significant increase in the $\xi\%$ shifting to the right the locations of the peaks. The dotted line in Fig. 5a is the theoretical $(\xi\% - \gamma)$ curve obtained from the Sig4 model but utilizing a variable y_0 parameter according to the shear strain. The modified version of the damping curve presented in Fig. 5a appears to be much better than the original damping curve in simulating the reduction and the re-increase in hysteresis soil damping at higher strains. The gradual reduction in the y_0 parameter adopted in Fig. 5a to produce reasonable soil damping behaviour compared to the experimental data can be viewed as an analogue to the pore water pressure built-up, R_u measured in the T_xSS tests as shown in Fig. 5b. As shown in this figure, the initial value of the y_0 is kept constant at 0.12 up to the threshold strain $\gamma_{th} = \pm 0.01$ where there is no generation of excess pore water pressure ($R_u = 0$). The

parameter y_0 starts to degrade with the excess pore water pressure generation ($R_u > 0$). According to Fig. 5b and to refine the results of the original Sig4 model with respect to the damping behavior of sensitive clay at higher strains, the parameter y_0 should no longer assumed constant but varies with R_u according to the following relationship:

$$y_0 = \left[1 - \alpha_1 R_u^{\alpha_2} \right]^{\alpha_3} \quad (7)$$

where α_1, α_2 and α_3 are curve-fitting parameters. The values of these parameters used in this particular case on Champlain sensitive clay respectively are 1.00, 0.65, and 3.45.

In fact, several researchers (e.g., Berrill and Davis 1985; David and Berrill 2001) attempted to correlate the damping ratio or the dissipated energy per unit volume of soils with the excess pore pressure water pressure ratio, R_u . Moreover, the difference between the $(G/G_0 - \gamma$ and $\xi\% - \gamma)$ curves when $R_u = 0$ and those when $R_u > 0$ has been documented. However, the main scheme of the current attempt is to make the original Sig4 model flexible enough to fit the damping data and this has been achieved by linking the parameter y_0 of the Sig4 model with R_u , and some of the refinement results are plotted in Fig. 6. Figure 6 presents a comparison between experimental and theoretical backbone curves computed by the Sig4 model assuming the parameter y_0 to be constant (original model) or variable (modified model). This figure demonstrates the poor fit of the original Sig4 model ($y_0 = 0.12$) to the experimental hysteresis T_xSS data of Champlain clay especially at large strains ($> \pm 1\%$). The flexibility given to the parameter y_0 in the modified version of the Sig4 model ($y_0(R_u)$) improve the theoretical results and render the model the capability to well reflect the experimental cyclic behavior of the soil over a wide range of shear strains. A close examination of the backbone curves from both original and modified

Sig4 models as well as the experimental hysteresis shear stress-shear strain loops would be beneficial to properly understand the hysteresis damping behavior of the Champlain clay observed in the experimental tests. The experimental stress-strain loops presented in Fig. 2 have been detailed discussed/grouped according to the shear strain level into three groups as illustrated in Figs. 7a-7c. For shear strain level less than $\pm 0.146\%$ (Fig. 7a-1), it could be observed that there is a nonlinear isotropic evolution of the stress-strain loops. In other words, there is a relatively uniform expansion of the hysteresis stress-strain loops with the increase in shear strains. It should be noted that this group of stress-strain loops include the small-strain stress-strain hysteresis loops portrayed in Fig. 7a-2. In the strain range between $\pm 0.146\%$ and $\pm 0.503\%$ (Fig. 7b), it was observed that the evolution of the stress-strain loops is of isotropic-kinematic linear type. However, the loops at such strain level gradually change their shapes to be rather condensed; this is expressed by the existence of a rigid boundary that circumvents the loops (isotropic-kinematic linear evolution). For strain level greater than 0.503% , the shape of the loops appears similar to those in Fig. 7b but with semi-rigid boundary. In other words, there is a nonlinear evolution of the stress-strain loops but with a slow variation in their shapes (Fig. 7c). According to the information given in Fig. 7, it appears that the St. Adelphe clay, under cyclic loading, behaves in three different ways according to the applied shear strain levels. The different zones of the clay behavior can be summarized as:

Zone I (Fig. 7a): where a nonlinear isotropic evolution takes places. The clay behaviour at this zone is characterized by the rapid variation of the secant shear modulus degradation with the applied shear strain level. It should be noted that at a very small-strain level ($\gamma < \pm 0.005\%$), the clay behavior is almost linear and there is only a small variation of the secant modulus (Fig. 7a-2).

Zone II (Fig. 7b): at which a linear kinematic evaluation come about. The clay behaviour at the second zone is characterized by quite linear behaviour that resulted from the insignificant variations of the shear modulus compared to those observed in *Zone I*.

Zone III (Fig. 7c): describes a new nonlinear kinematic-isotropic evolution with insignificant variation of the shear modulus compared to earlier zones.

On the other hand, the original Sig4 model seems to predict only two types of hysteresis loops (Figs. 2 and 6), one with a nonlinear isotropic evolution (*Zone 1*) for strain level ($\gamma < \pm 0.15\%$) and the other (*Zone 2*) is, in fact, a combination of nonlinear isotropic and linear kinematic behavior types, and becomes more linear at higher strain levels. Schematic presentations of these two zones are given in Figs. 8a and 8b. When the mathematical model Sig4 has been refined by accounting for the variation of the excess pore water pressure, the resulting model (the modified Sig4 model) predicts well the three zones (*Zone I*, *II*, and *III* as shown in Figs. 8d, 8e, and 8f) of experimental hysteresis loops and produces results better resemble the Champlain clay cyclic behavior observed in the laboratory in term of the variations of G/G_0 (Fig. 8g) and $\zeta\%$ (Fig. 8h) with shear strain level.

Referring to the Sig4 results ($\gamma_0 = 0.12$) shown in Figs. 8g and 8h, it can be noted that the estimated damping ratio and the corresponding normalized secant modulus up to a strain level of 0.15% (*Zone 01*), respectively increase and decrease in a non-linear convex form, which is in a general agreement with the T_xSS test results. In this zone, except for very small-strain level ($\gamma < \pm 0.005\%$) where the soil behaviour is almost linear (Fig. 8j), the hysteresis stress-strain loops are generally characterized by their expansive shape as it can be shown Figs. 8k and 8l. It is also worth of note that the generated pore water pressure is insignificant (i.e, the maximum

value of the generated excess pore water pressure ratio, R_u at strain level of 0.15 is about 0.025 as shown in Fig. 8i).

For strain level greater than 0.15% (Zone 02), it has been noted that the shapes of the hysteresis stress-strain loops change and become rather linear in both unloading and reloading phases as the solid lines in Figs. 8m (point 4) and 8n (point 5) present it. The change in the loops shape would, in fact, affect the dynamic characterizations of the material. More specifically, it will, naturally, result in a reduction of the damping ratio magnitude, as the shape of the loops does not practically change with further increase in strain level (see Figs. 8 m and 8n). This is portrayed in Fig. 8h as a degradation of $\zeta\%$ from its maximum value of 16% at strain level 0.15% with further increase in the strain level ($\gamma \geq \pm 0.15\%$). A comparison between theoretical results of the Sig4 model and the measured stress-strain backbone data shown in Fig. 8c demonstrates the mathematical formulation of the model works quiet well in predicting the soil dynamic behavior for strain level up to 0.15% (Zone 1). However, the numerical results commence to deviate from the experimental results when the applied strain level becomes greater than the 0.15%. In other words, the Sig4 model greatly overestimate the shear stress induced in the soil compared to the stress experienced by the soil sample in the laboratory. One of the plausible reasons for this discrepancy is that at such strain level, there is a significant generation of excess pore water pressure as obviously shown in Fig. 8i, which is not accounted for in the original Sig4 model mathematical formulation. The results presented in Figs. 8g and 8h are very interesting as they imply that there is an intermediate zone in the range of strain level between $\pm 0.1\%$ and $\pm 1.0\%$ that deserves more attention. The existence of this zone indicates that the evolution of the damping ratio of the Champlain clay sample tested in the current study is of interest because one would detect two damping

summits (Fig. 8h): the first one corresponds to the change of the loops shape (point 3 Fig. 8l) from isotropic form to an almost linear one and this summit has been successfully simulated using the initial Sig4 model. The linearity of the hysteresis loops would naturally reduce the damping ratio as explained above. When the generation of pore water pressure ratio is accounted from in the modified Sig4 model ($y_0 = \text{variable}$), there will be a significant reduction in the shear stress induced in the soil (Fig. 8c) (which is in coincidence of the experimental data) producing another change in the hysteresis loops shape as the dashed lines in Figs. 8m (point 4) and 8n (point 5) present it and consequently producing a new evolution (re-growth) of the damping ratio (Zone III) (Fig. 8h). It is also worth noting that the shape of the normalized secant modulus G/G_0 curve estimated from the modified Sig4 model experiences a reduction at high strain level ($\geq \pm 1.0\%$) due to the considering of the excess pore pressure water built-up and this would reduce the shear stress at such high strain levels leading to an increase in the corresponding damping ratio.

The results of the T_xSS tests and simulations through the original and the modified Sig4 soil model presented schematically in Fig. 8 confirm that the shape of the $(G/G_0)|_{Ref} \square \gamma$ curve must be updated in medium and high levels of strains mainly due to the degradation of the shear modulus caused by the evolution of the excess pore water pressure. On the other hand, at low-strains levels ($\leq \pm 0.15\%$), the reference (original Sig4) model works well in predicting both $(G/G_0) \square \gamma$ and $(\xi\%) \square \gamma$ curves, while at strain level range between 0.1% to 1%, caution should be experienced especially when there is a change in the shape of the loops as this can be an indication of a modification of dynamic characteristics curves as it has been experimentally observed and theoretically confirmed throughout the present study.

Figures 9a and 9b show, respectively the variation of the normalized shear modulus (G/G_0) and damping ratio ($\xi\%$) with shear strain obtained from the T_xSS results on St-Adelphe Champlain clay samples extracted from different depths and tested under different confining pressures and loading frequencies. The curves suggested by Vucetic and Dobry (1991) at different Plasticity index and those calculated from the modified Sig4 analytical procedure at PI of 21 and 29 are also plotted in Fig. 9. Figure 9 demonstrates that the experimentally- and the analytically-determined (G/G_0 - γ) data generally fall within the range suggested by Vucetic and Dobry (1991). However, the measured and the computed damping ratios undergo different trend with respect to γ . More specifically, up to a strain level of $\pm 0.2\%$, the computed $\xi\%$ - γ curves of Champlain clay having PI of 21 and 29 respectively follow typical curves of clays at PI 15 and 30 given by Vucetic and Dobry (1991). Beyond this strain level, the St-Adelphe damping curves computed from modified Sig4 model at both PI of 21 and 29 tend to converge and follow the damping trend suggested by Vucetic and Dobry (1991) at PI of 100.

Conclusion

Based on a series of laboratory tests using the new combined triaxial simple shear (T_xSS) apparatus, normalized stiffness G/G_{max} and hysteretic damping ratio $\xi\%$ versus γ curves of sensitive Champlain clays at St-Adelphe site in Quebec are constructed for the first time. The T_xSS apparatus offers the ability to measure the soil stiffness over a wide range of strain spectrum from 0.001% to 10%, thus reduce the difficulties and the sources of uncertainty associated with traditional methods. The values of the small strain stiffness G_0 of the sensitive Champlain clay has been confirmed through another series of experimental tests using the piezoelectric ring-actuator technique (P-

RAT). Moreover, the obtained G/G_0 degradation curves of Champlain clays are compared successfully to typical deprecation curves of clays suggested in the literature. The damping ratio of the sensitive Champlain clays exhibit different trend with respect to γ . Although, the experimental $\xi\%-\gamma$ curves of the Champlain clay follow typical curves of clays up to certain strain level. Beyond this level, damping ratio curves tend to be very different from those found in the literature which would overestimate the seismic response of the Champlain clays obtained from analyses implementing typical $\xi\%-\gamma$ curves. A modification of the original Sig4 soil model to take into account the pore water pressure built up with shear strain has been therefore presented in this paper. The stiffness degradation and hysteresis damping ratio versus shear strain curves of Champlain clays estimated using the suggested soil model are compared successfully with their experimentally-determined counterparts even at large shear strains. The plausible reasons of the inability of the original Sig4 model to successfully simulate the dynamic behavior of the Champlain clay and some recommendations to be accounted for in future computations of the dynamic characteristics of soils using traditional soil models have been also discussed.

List of notations

RC : Resonance column

BE : Bender element

DSS : Direct simple shear

CTX : Cyclic triaxial test

ρ : Soil density

T_{xSS} : The triaxial simple shear

P-RAT : The piezoelectric ring-actuator technique

G_0 : The small-strain shear modulus

$\tau_{\max}$: maximum shear stress at infinite strains

γ_{ref} : The reference strain

V_s : Shear wave velocity

V_{s1} : The normalized shear wave velocity

P_a : The normal atmospheric pressure

σ'_v : The effective vertical stress

PI : The plasticity index

LI : Liquidity index

S_t : Sensibility

σ'_h : Effective horizontal stress

K_0 : Coefficient of earth pressure

γ_{th} : Shear strain threshold

R_u : The pore water pressure

G : Secant shear modulus

τ : Cyclic stress amplitude

γ : Shear strain amplitude

ξ : The equivalent material hysteresis damping ratio

$F_{bb}(\gamma)$: The initial Sig4 stress-strain curve

L_y : The $\text{Log}_{10}(\gamma(\%))$

a, b, x_0, y_0 : are curve-fitting parameters for Sig4 model

σ'_p : The pre-consolidation pressure

e : The void ratio

OCR : Over consolidation ratio

$\alpha_1, \alpha_2, \alpha_3$: are curve-fitting parameters for degradation value of the parameters y_0

$(\xi)_{|_{Ref}}$: The estimated damping ratio referred to $R_u=0$

$(G/G_0)_{|_{Ref}}$: The normalized secant modulus referred to $R_u=0$

σ'_c : Effective confining pressure

e_c : Consolidated void ratio

F_r : Frequency

τ_a : Reversible shear stress

γ_a : Reversible shear strain

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Figure Captions

- Figure 1:** Preparation of undisturbed sensitive clay sample for T_xSS tests.
- Figure 2:** Experimental and numerical shear stress-shear strain loops of Champlain clay at different strain levels
- Figure 3:** Consolidation curves of undisturbed sample of St-Adelphe Champlain clay.
- Figure 4:** Shear stiffness and damping vs shear strain for St-Adelphe clay.
- Figure 5:** (a) comparison between measured and calculated values of damping at different strains and (b) variation of the parameter γ_0 with the excess pore water pressure, R_u .
- Figure 6:** Experimental and computed backbone curves.
- Figure 7:** Compatibility between successive loops from very small to large strains.
- Figure 8:** Analysis of original and modified Sig4 performance in predicting dynamic characteristics of Champlain St-Adelphe clay.
- Figure 9:** Shear stiffness and damping vs shear strain of different St-Adelphe clay samples tested in the T_xSS .

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Figure 4: Shear stiffness and damping vs shear strain for St-Adelphe clay.

Figure 5: (a) comparison between measured and calculated values of damping at different strains and (b) variation of the parameter y_0 with the excess pore water pressure, R_u .

Figure 6: Experimental and computed backbone curves.

Figure 7: Compatibility between successive loops from very small to large strains.

Figure 8: Analysis of original and modified Sig4 performance in predicting dynamic characteristics of Champlain St-Adelphe clay.

Figure 9: Shear stiffness and damping vs shear strain of different St-Adelphe clay samples tested in the T_xSS.

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Table 1: Mineral characteristics of Champlain clay

Type of mineral	Proportion (%)
Quartz	14-40
Plagioclases	25-50
Potassium feldspars	2-15
Amphiboles	0-15
Calcite	0-15
Dolomite	0-3
Phyllosilicates and amorphous materials	10-45

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