**Proceedings of the XVI ECSMGE**
**Geotechnical Engineering for Infrastructure and Development**
ISBN 978-0-7277-6067-8


doi:10.1680/ecsmge.60678

# Resonant column and cyclic torsional shear experiments on soils of the Trentino valleys (NE Italy)

## Expérimentations sur sols dans la vallée du Trentin (NE Italie) avec Colonne de résonnance et Cisaillement Cyclique Torsionnel

F. Fedrizzi[*1], P.L. Raviolo[2] and A. Viganò[3]

[1] *Geotechnical Laboratory, Geological Survey, Autonomous Province of Trento, Trento, Italy*
[2] *Wykeham Farrance Soil Mechanics Division of CONTROLS, Liscate (Milano), Italy*
[3] *Istituto Nazionale di Oceanografia e di Geofisica Sperimentale, Udine, Italy*
[*] *Corresponding Author*

ABSTRACT Experimental results from Resonant Column (RC) and Cyclic Torsional Shear (CTS) laboratory tests on clayey, silty and sandy soils of the Trentino valleys (NE Italy) are here presented. Main calibrations and checking of equipment are explained in order to describe the adopted testing procedures. The influence of experimental methods and soil index properties on shear modulus and damping ratio normalized values are investigated. Laboratory data are significantly affected by the testing method (RC or CTS), the applied effective pressure and some index properties (i.e., plasticity for fine soils and voids ratio for sands). The comparison of laboratory and *in-situ* results for the same reference site shows shear-wave velocities of the same order of magnitude and similar vertical profiles with along-depth increasing velocity.

RÉSUMÉ Les résultats expérimentaux d'essais de laboratoire avec Colonne de Résonnance (RC) et Cisaillement Cyclique Torsionnel (CTS) sur sols silteux, argileux et sablonneux de la vallée du Trentin (NE Italie) sont présentés. Les paramétrages préliminaires et étalonnages sont décrits pour expliquer les procédures d'essais adoptées. L'influence des méthodes expérimentales et les propriétés d'indices de sols sur le module de cisaillement et le taux d'amortissement sont évalués. Les résultats en Laboratoire sont affectés de façon significative par la méthode d'essai (RC ou CTS), la pression effective appliquée et les propriétés du sol (par ex. plasticité pour sols fins et pourcentage de vides pour sables). La comparaison des résultats en laboratoire et sur site montre des vitesses d'onde de cisaillement du même ordre de magnitude et des profils verticaux similaires avec un accroissement de vitesse en profondeur.

## 1 INTRODUCTION

Resonant Column (RC) and Cyclic Torsional Shear (CTS) experiments define the stress–strain pre-failure behaviour under cyclic load of undisturbed/reconstituted soil samples (e.g., Yokota et al., 1981; Lo Presti et al., 1997). Shear modulus and damping ratio curves, as a function of shear strain generally between 0.0001% and 0.1%, are obtained. These types of results are widely used for seismic response analyses at a regional/local scale.

The RC and CTS apparatus used for the work here presented is the *Stokoe* apparatus for a fixed-free configuration (Stokoe et al., 1980).

During RC tests, a sinusoidal torsional vibration at variable frequency is applied using a rotary excitation device mounted at the top of the specimen. The fundamental frequency is measured according to (Richart et al., 1970):

$$\frac{I}{I_0} = \frac{\omega_n h}{V_S} \tan \frac{\omega_n h}{V_S} \qquad [1]$$

where $I$ is the mass polar moment of inertia of the specimen, $I_0$ is the mass polar moment of inertia of the components mounted on the top of the specimen (drive system, top platen, etc.), $\omega_n$ is the circular frequency of the first torsional mode of vibration, $h$ is the height of the specimen and $V_S$ is the shear-wave velocity.

The shear modulus ($G$) can be calculated as:

$$G = \rho V_S^2 \qquad [2]$$

where $\rho$ is the specimen density.

During CTS tests, a sinusoidal torsional vibration at low constant frequency (~0.1–5 Hz), for a finite number of cycles, is applied.

Hysteresis loops are plotted and $G$ is obtained from:

$$G = \frac{\tau_{pp}}{\gamma_{pp}} \qquad [3]$$

where $\tau_{pp}$ and $\gamma_{pp}$ are the double-amplitude shear stress and strain, respectively.

## 2 PRE-TEST CHECKS AND CALIBRATIONS

### 2.1 Shear strain

A correct measure of the torsional angle at the top of the specimen is necessary both to quantify shear strain (RC and CTS) and to calculate the resonance frequency in RC experiments.

If shear strain is related to a $\frac{2}{3}$ specimen radius, the following applies:

$$\gamma = \frac{2}{3}\frac{R}{h}\theta_{\max} \qquad [4]$$

where $\gamma$ is the reference shear strain, $R$ and $h$ are radius and height of the specimen, and $\theta_{\max}$ is the maximum single-amplitude torsional angle.

The RC torsional angle can be measured by an accelerometer which is integral with the excitation device or by two displacement transducers (i.e., proximitors) assembled at the top of the specimen. In CTS low-frequency tests only proximitors can be used.

A metrological comparison between the results obtained by accelerometer and proximitors shows that (i) RC resonance frequency values are almost the same, with discrepancies less than 0.4%, (ii) proximitors tend to underestimate the shear strain for frequencies higher than 45 Hz, with discrepancies proportional to vibration frequency, and (iii) shear strain lower than 0.001% is considered not reliable if measured by proximitors.

However, the shear modulus vs. logarithmic shear strain interpolated curve is less sensitive to measure uncertainties at low than high strain, because at low strain levels shows a typical sub-horizontal trend.

### 2.2 Apparatus support and calibration of the polar moment of inertia

The quality of results is due to many factors of the testing equipment, e.g., apparatus stiffness, specimen–base pedestal connection, and others (Clayton et al., 2009).

According to Clayton et al. (2009), the mass polar moment of inertia of the apparatus base should be approximately 500 times greater than the drive head and the apparatus must be strongly fixed to the base. This suggestion is particularly important when analyzing stiff soils; for example, in our investigations, gravelly sands samples with shear modulus greater than 600 MPa.

During our experiments the apparatus was firmly fixed to a base with a mass polar moment of inertia (13,000 kg $cm^2$) 1,000 times greater than the inertia of the components mounted on the specimen top. The fixity between the apparatus and its base is obtained by reinforced built-in-base supports, fixed with passing screws.

The mass polar moment of inertia of the components mounted on the specimen top ($I_0$) is determined by a specific calibration using bars of different stiffness and added masses with known mass polar moment of inertia (cf. eq. 1).

## 3 INFLUENCE OF EXPERIMENTAL PROCEDURE ON THE RESULTS

### 3.1 Differences between RC and CTS

With the *Stokoe* apparatus used for this study it is possible to carry out both RC and CTS tests on the same specimen, without changing the apparatus setting.

A comparison between two methods on the same specimen is infrequent for geotechnical laboratory testing, where mechanical tests usually reach failure conditions.

The comparison between RC and CTS results is possible only for a specific shear strain level, where strain is experimentally verified to remain under the elastic–plastic threshold by monitoring pre-straining effects and the cyclic degradation index.

Both CTS and RC tests were carried out on medium- and low-plasticity silty-clayey soils: 8 tests from the Sole Valley (Caldes, NW Trentino) and 5 tests from the Adige Valley (Villazzano, central Trentino) (Fig. 1).

In all the carried out tests, the absolute values of shear modulus from RC are higher than those from CTS. The normalized values have the trend shown in Fig. 1.

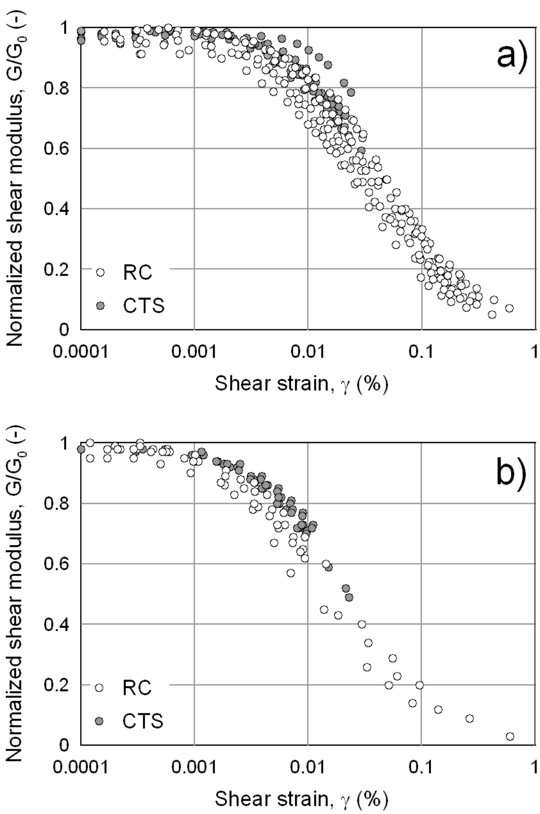


**Figure 1.** Comparison between RC and CTS test results, performed consecutively on the same specimens (medium- and low-plasticity silty-clayey soils). a) Sole Valley site (Caldes, 8 samples). b) Adige Valley site (Villazzano, 5 samples).

### 3.2 Influence of test frequency in CTS

For silty-clayey soils of the Adige Valley (Villazzano, central Trentino) and gravelly sands of the Fassa Valley (Canazei, NE Trentino) CTS tests were repeated at different frequencies (0.5, 2.0 Hz in the first case; 0.1, 1.0, 2.0 Hz in the second case), being always careful to remain under the elastic–plastic threshold (Fig. 2).

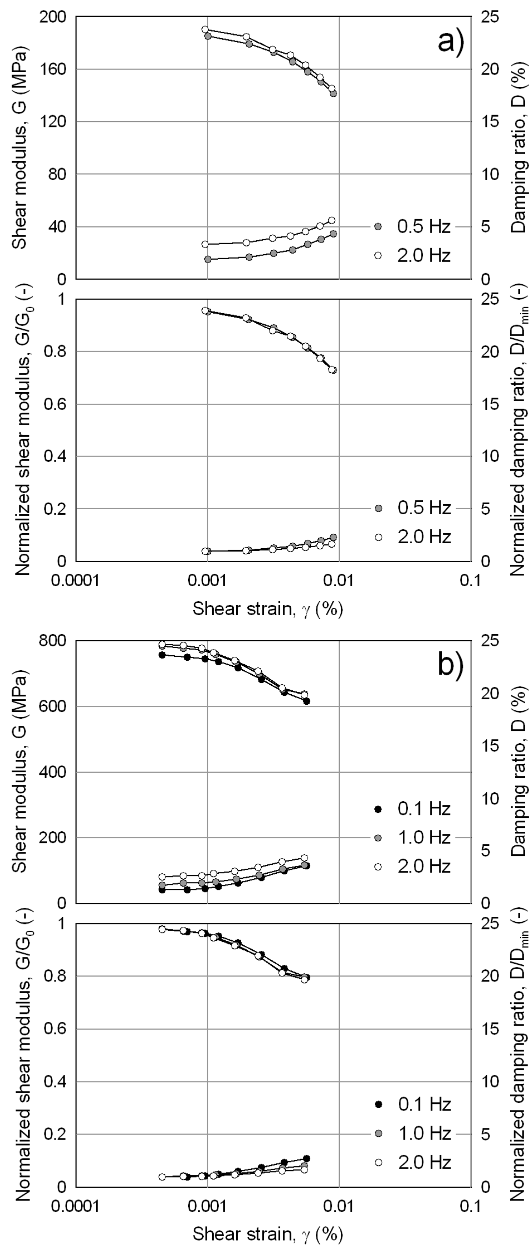


**Figure 2.** CTS results obtained at different test frequencies on the same specimens. a) Silty-clayey soils of Adige Valley. b) Gravelly sands of Fassa Valley.

Absolute values of shear modulus and damping ratio increase with frequency. On the contrary, normalized diagrams are very similar, especially for shear modulus curves (Fig. 2).

### 3.3 Influence of mean effective pressure

To investigate the role of mean effective pressure on the experiments, shear modulus and damping values have been measured at different consolidation pressures. To compare curves for the same specimen, the tests were performed up to the elastic–plastic threshold.

Figure 3 shows the normalized curves at 100 and 200 kPa effective pressures. Higher effective pressure produces higher shear modulus and lower damping ratio (on normalized curves).

Since the pressure-dependence of RC and CTS results is recognised, both for absolute and normalized values, mean effective pressure should be made explicit for each experiment. In this sense, for a correct evaluation of literature normalized data, not only soil type and experimental methods but also tested mean effective pressure should be considered. In addition, this attention is crucial to appropriately select experimental results in order to consider *in-situ* conditions (depth of investigation) for the stratigraphic model.

### 3.4 Hints on result processing

To completely and correctly elaborate and show the results, some basic choices should be clarified and shared with data end-users.

As commonly accepted, shear strain is measured at a characteristic distance ($r$) equal to $\frac{2}{3}R$ ($R$ is specimen radius) from the specimen rotation axis (cf. eq. 4), assuming a linear dependence between shear strain and $r$ (Hardin & Drnevich, 1972). However, different authors proposed a different approach, where at high strain ($\gamma \cong 0.1$) a $r/R$ ratio equal to 0.79 can be reached (Chen & Stokoe, 1979).

The determination of shear modulus at minimum shear strain level ($G_0$) is fundamental to normalize shear modulus.

In this work, the shear modulus has been extrapolated at null strain by using the Hardin & Drnevich (1972) hyperbolic equations. Other methods are also possible, obtaining $G_0$ from an average of different measurements at very-low shear strain or from the *bender elements* technology.

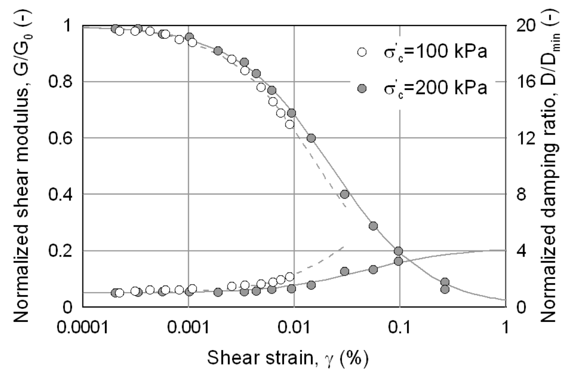


**Figure 3.** Normalized shear modulus and damping ratio values obtained at two different effective pressures on the same specimen (silty sands).

## 4 INFLUENCE OF INDEX PROPERTIES

### 4.1 Plasticity of fine soils

In geotechnics, Atterberg limits are one of the most common index properties for fine soils. According to Vucetic & Dobry (1991), plasticity strongly controls the dynamic response, in terms of shear modulus and damping ratio.

Figure 4 shows the normalized shear modulus of medium- and low-plasticity clays, compared to the experimental curves by Vucetic & Dobry (1991). At the same strain level, the higher is the plasticity index, the higher is the normalized shear modulus. The analyzed soils show a similar behaviour with Vucetic & Dobry (1991) curves, up to the elastic–plastic threshold, with a slightly different behaviour at higher shear strain.

Tested soils were classified as “clays” according to the plasticity chart, but as “clayey silts” on the basis of the particle size-distribution chart. Moreover, other laboratory tests (i.e., oedometer, triaxial CIU and hydraulic conductivity tests) indicate an intermediate behaviour between clays and silts.

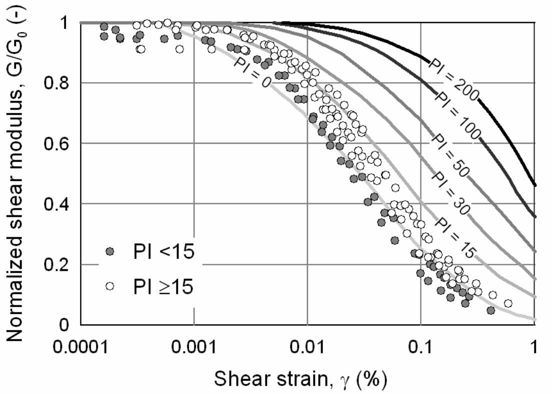


**Figure 4.** Influence of the plasticity index (PI) on experimental results (dots, 8 samples of Sole Valley clays), compared to literature data (Vucetic & Dobry, 1991).

### 4.2 Voids ratio of sands

For sands, Seed & Idriss (1970) proposed to consider the following relationship (in psf units) between shear modulus and effective pressure ($\sigma'_c$):

$$G = 1{,}000 K_2 (\sigma'_c)^{1/2} \quad [5]$$

where $K_2$ is a coefficient that considers the influence of voids ratio (or relative density).

The same equation, at low shear strain when $K_2$ reaches its maximum ($K_{2(\max)}$), in S.I. units becomes:

$$G_{\max} = 6.92 K_{2(\max)} (\sigma'_c)^{1/2} \quad [6]$$

where $G_{\max}$ is the maximum shear modulus.

**Table 1.** Comparison between $K_{2(\max)}$ values from experimental data obtained on two sandy soils of Sole Valley (Caldes) and from Seed & Idriss (1970).

| This study | | Seed & Idriss (1970) | |
|---|---|---|---|
| Voids ratio ($e$) | $K_{2(\max)}$ | Voids ratio ($e$) | $K_{2(\max)}$ |
| - | - | 0.4 | 70 |
| - | - | 0.5 | 60 |
| 0.61 | 44.2 | 0.6 | 51 |
| - | - | 0.7 | 44 |
| - | - | 0.8 | 39 |
| 0.89 | 35.4 | 0.9 | 34 |

Two samples of reconstituted uniform sandy soils (Sole Valley, Caldes) with different relative densities were analyzed. $K_{2(\max)}$ values were calculated interpolating $G_{\max}$ values obtained at different consolidation pressures (Fig. 5a). These results are compared with Seed & Idriss (1970) $K_{2(\max)}$ values (Table 1).

The normalized shear modulus variation with shear strain shows a good fitting with the Seed & Idriss (1970) curves (Fig. 5b). Also for this reason, the comparison of the listed $K_{2(\max)}$ values is more significant (Tab. 1).

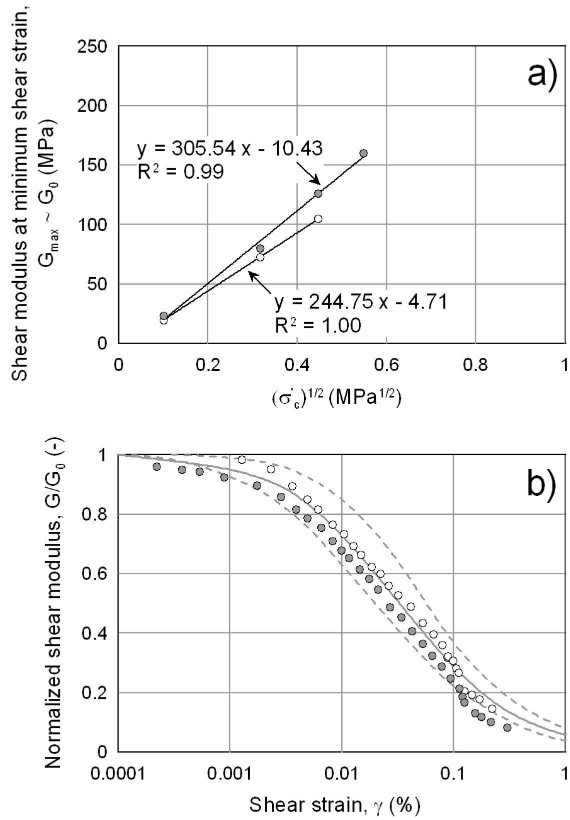


**Figure 5.** a) Relationship between effective pressure and shear modulus at low strain levels, to calculate $K_{2(\max)}$ values for two sandy soils with different voids ratios (gray and white dots) (cf. Table 1). b) Experimental data compared to literature data (average and dispersion range curves; Seed & Idriss, 1970).

## 5 COMPARISON WITH *IN-SITU* DATA

The compatibility between laboratory and *in-situ* results for the Sole Valley (Caldes) reference site has been verified, because of the presence of 8 samples at different depths analyzed at their own stress levels.

In the laboratory, $G_0$ values of each sample were measured at different consolidation pressures. For each specific *in-situ* depth interval, these values were interpolated to obtain the $G_0$ value at the specific effective pressure.

Starting from these $G_0$ values, the laboratory shear-wave velocities ($V_S$) were obtained and compared to the shear-wave velocity depth-profile from *in-situ* geophysical investigations (ERT, MASW, FTAN, HVSR techniques by University of Padua) (Fig. 6). The comparison shows the expected discrepancy between the two distinct approaches (laboratory vs. *in-situ*), even if absolute values are of the same order of magnitude and both the profiles show an increasing along-depth velocity, starting from very similar values at shallow depth.

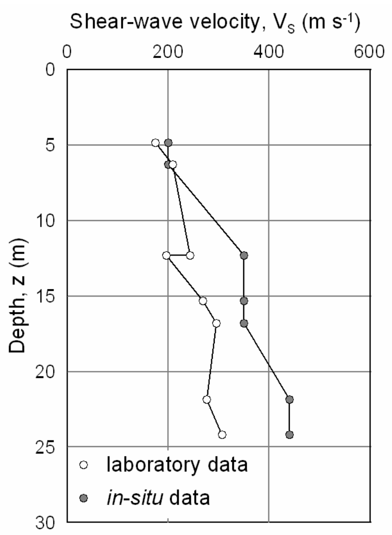


**Figure 6.** Comparison between laboratory and *in-situ* shear-wave velocities (Sole Valley sedimentary deposits).

## 6 CONCLUSIONS

RC and CTS tests can be performed on the same specimens, up to the elastic–plastic threshold. The different applied methods (i.e., different theoretical principles, instruments and data processing) help to evaluate global accuracy on final results.

Results are less sensitive to test frequency (in CTS) than to the applied test method (RC or CTS). Normalized values obtained at different frequencies in CTS are very similar.

Normalized shear modulus and damping ratio are significantly affected by mean effective pressure. Not only should soil types or associated geotechnical properties be taken into account, but also mean effective pressure at which values are determined. This is particularly important to correctly assign dynamic properties (experimentally determined or by literature) to real stratigraphic levels.

The influence of plasticity (for clayey and silty soils) and voids ratio (for sandy soils) were verified, semi-quantitatively assessed and compared to available literature data.

For this study, laboratory shear-wave velocities differ from *in-situ* values, even if they are of the same order of magnitude. Laboratory tests are complementary to *in-situ* investigations, which remain the principal method to constrain a realistic physical model. In any case, laboratory data are especially useful to complete *in-situ* information at higher shear strain.

## REFERENCES

Chen, A.T. & Stokoe, K.H. 1979. *Interpretation of strain dependent modulus and damping from torsional soil tests*, USGS-GD-79-002, NTIS No. PB-298479, Menlo Park CA.

Clayton, C.R.I., Priest, J.A., Bui, M., Zervos, A. & Kim, S.G. 2009. The Stokoe resonant column apparatus: effects of stiffness, mass and specimen fixity, *Géotechnique* **59**, 429–437.

Hardin, B.O. & Drnevich, V.P. 1972. Shear modulus and damping in soils: design equations and curves, *Journal of Soil Mech. and Found. Division* **98**, 667–693.

Lo Presti, D.C.F., Jamiolkowski, M., Pallara, O., Cavallaro, A. & Pedroni, S. 1997. Shear modulus and damping of soils, *Géotechnique* **47**, 603–617.

Richart, F.E. Jr., Hall, J.R. & Woods, R.D. 1970. *Vibration of soils and foundations*, Prentice Hall, Englewood Cliffs NJ.

Seed, H.B. & Idriss, I.M. 1970. *Soil Moduli and Damping Ratios for Dynamic Response Analyse*s, report No. EERC 70-10, Earthquake Engineering Research Center, Univ. of California, Berkeley.

Stokoe, K.H. II, Isenhower, W.M. & Hsu, J.R. 1980. *Dynamic properties of offshore silty samples*, Proc. 12th Ann. Offshore Technology Conference, Houston, Texas, 3771-MS.

Vucetic, M. & Dobry, R. 1991. Effect of soil plasticity on cyclic response, *Journal of Geotechnical Engineering* **117**, 89–107.

Yokota, K., Imai, T. & Konno, M. 1981. *Dynamic deformation characteristics of soils determined by laboratory tests*, OYO Technical Rep. 3, 13–37.