Modulus of sands from CPT's and DMT's
Module des sables au moyen des essays CPT et DMT

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G. Baldi, Engineer, ISMES, Bergamo, Italy

R. Bellotti, Engineer, ENEL CRIS, Milano, Italy

V.N. Ghionna, Assoc. Prof. of Geotech. Engineering, Università degli Studi di Pavia, Pavia, Italy

M. Jamiolkowski, Professor of Geotechnical Engineering, Politecnico di Torino, Turin, Italy

D.C.F. Lo Presti, Ph.D., Research Fellow, Politecnico di Torino, Turin, Italy

SYNOPSIS: Based on the results of a large number of in situ tests performed in a calibration chamber and in the field, the reliability of empirical correlations between the results CPT's and DMT's and the stiffness of predominantly silica sands are discussed. A tentative correlation between in situ test results and maximum shear modulus G_o of cohesionless soils is proposed.

1 INTRODUCTION

Since the appearance and first use of penetration tests, engineers have been attempting to assess the deformation characteristics of cohesionless soils and/or settlement of structures on sands and gravels from their results. This trend has arisen mainly because in cohesionless soils undisturbed sampling was and still remains impossible or at least not cost effective and the evaluation of total and differential settlements represents an essential step.

Numerous methods relating the deformation characteristics and/or settlements of geotechnical construction to the results of penetration tests, have been devised following one of two approaches:

- Working-out empirical correlations between a reference deformation modulus (RDM), measured in laboratory usually on reconstituted soil specimens and the result of a penetration test performed either in the field or in laboratory (De Beer, 1948; D'Appolonia et al., 1968; Schmertmann et al., 1978; Ohta and Goto, 1978).
- Attempting to correlate directly the observed settlements of constructions or large-scale prototypes against the results of a preselected penetration test. This method allows a direct assessment of the settlement of a given foundation (e.g. Burland & Burbridge, 1985), or the evaluation of an average value of the RDM (e.g. Schmertmann, 1970; Schmertmann et al., 1978; D'Appolonia et al., 1968).

However, in the last decade, the better understanding of the mechanical behaviour of cohesionless soils (e.g.: Ishihara, 1986; Lade, 1977, Lambrechts & Leonards, 1978) together with the experience gained in penetration testing performed in large calibration chambers (e.g., Holden, 1971; Schmertmann, 1978; Baldi et al., 1985) made clear the limitations involved when trying to evaluate settlements and especially deformation characteristics of cohesionless soil on the basis of penetration tests results. This work attempts to summarize the writers' points of view on the assessment of the different RDM on the basis of CPT's and DMT's results. The following experimental data incorporating ten years of research activity are examined:

- Numerous electrical CPT's (quasi static cone penetration tests) and DMT's (Marchetti dilatometer tests) performed in pluvially deposited Ticino (TS) and Hokksund (HS) predominantly silica sands (Figure 1), using large calibration chambers (CC) (see Bellotti et al. 1982; 1988; Baldi et al., 1986, 1986a).
- A large number of conventional laboratory tests performed on pluvially deposited specimens of the same test sands.
- A large number of in situ and laboratory tests performed in the quaternary deposits of the predominantly silica Po river sand and Gioia Tauro sand with gravel (see Figure 1).

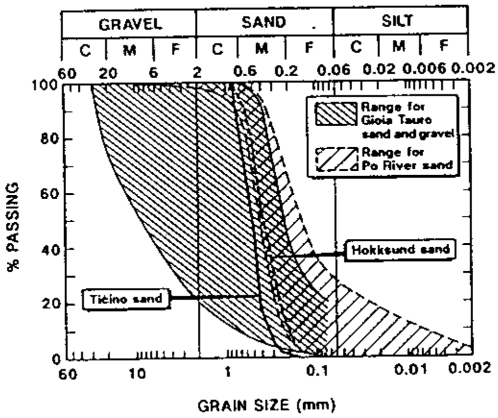


Figure 1. Granulometric characteristics of tested materials.

2 FACTORS ON WHICH ENGINEERING CORRELATIONS DEPEND

The deformation characteristics of a given soil depend on the factors summarized in Table 1.

On the other hand, in a given soil all types of penetration resistances (PR) are primarily controlled by the void ratio and/or the relative density D_R and the state of the effective stress. Their combined effect is successfully incorporated in the state parameter ψ as defined by Been & Jeffries (1985).

Table 1. Factors on which stiffness of cohesionless soils during monotonic loading depends.

	Factors	Soil Response
Material	Grading Mineralogy Angularity	Compressibility Crushability
Fabric	Grain [Arrangement Grain [Orientation	Compressibility Anisotropy
Stress	Stress–strain history Mean effective stress Shear strain	Compressibility Anisotropy Plastic Hardening
Drainage	Undrained Drained	Excess Pore Pressure Volumetric strain
Time		Creep, Rate Effects

From these statements it appears obvious that in a given soil, the deformation moduli reflect a very complex material behaviour while the PR is mainly controlled by ψ. This was initially evidentiated by small scale laboratory tests (Lambrechts & Leonards, 1978) and fully confirmed by special CC tests (Baldi et al., 1985; 1986; Bellotti et al., 1986; Jamiolkowski et al., 1988).

These special tests have been devised to investigate the influence of mechanical overconsolidation (OC) and cyclic prestressing on the drained Young modulus E′ and the results of CPT and DMT's in TS.

During mechanical overconsolidation the sand specimen experiences both plastic hardening, due to accumulated irrecoverable strain, and an increase of the coefficient of earth pressure at rest K_o. While, at the end of the cyclic prestressing K_o has the same initial value and only plastic strain hardening is induced to the sand.

The tests with cyclic prestressing have been performed according to the following procedures:

a) one dimensional consolidation along NC K_o–line;

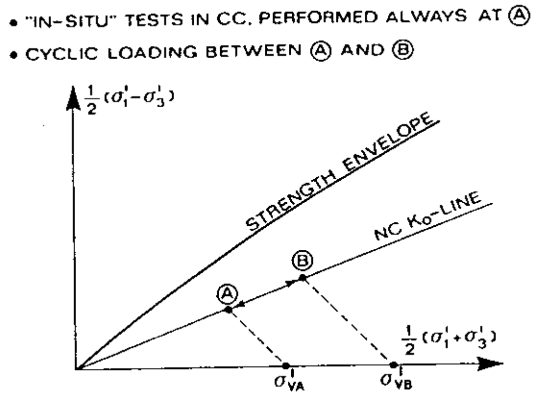
b) penetration of one half of CC specimen;

c) cyclic prestressing along different stress paths;

d) completation of penetration of CC specimen.

Figure 2 shows the results of one CPT and one DMT performed before and after the cyclic prestressing along a stress path following the NC K_o–line (ABA):

- As a result of cyclic prestressing E_t' increases 4.5 to 7 times while PR and DMT indices K_D and E_D (see Marchetti, 1980) change very little.
- As a result of mechanical OC, the ratio of $E_t'/\sqrt{\sigma_m'}$ increases 3 to 5 times, while all the above mentioned penetration parameters normalized with respect to $\sqrt{\sigma_m'}$ or $\sqrt{\sigma_h'}$ increase much less, less than 10 to 30%.



DMT	D_R (%)	σ'_{vA} (kPa)	σ'_{vB} (kPa)	K_o (–)	N_c (–)
	65	312	514	0.44	3

BEFORE				AFTER			
K_D (–)	E_D (MPa)	q_D (MPa)	E_t (MPa)	K_D (–)	E_D (MPa)	q_D (MPa)	E_t (MPa)
1.67	34.1	17.7	62.4	1.87	35.4	18.6	286

CPT	D_R (%)	σ'_{vA} (kPa)	σ'_{vB} (kPa)	K_o (–)	N_c (–)
	32	113	166	0.52	10

BEFORE		AFTER	
q_c (MPa)	E_t (MPa)	q_c (MPa)	E_t (MPa)
6.8	30.2	8.1	132

E_t =TANGENT YOUNG'S MODULUS AT A

N_c =NUMBER OF UNLOADING-RELOADING CYCLES

q_c =CONE RESISTANCE; q_D =DILATOMETER WEDGE RESISTANCE

E_D =DILATOMETER MODULUS; K_D =HORIZONTAL STRESS INDEX

Figure 2. Effect of prestraining on results of CPT and DMT in Ticino sand.

Very similar results have been obtained by Clayton et al. (1985), performing dynamic cone penetration tests in a CC on Reid Bedford sand samples.

The above indicates that the large strains caused by the penetration of devices like SPT, CPT, DMT, etc., obliterate to a large extent the effects of the stress and strain history in the soil.

3 CORRELATION BETWEEN E′ AND CPT AND DMT RESULTS

Figures 3 and 4 show the correlation between the results of CPT's and DMT's performed in the CC in predominantly silica TS and E_s' or M_t adopted as RDM. In these figures the following parameters are involved:

- Cone resistance q_c measured at mid-height of the CC specimen.
- Constrained modulus M_D inferred from the dilatometer modulus E_D according to Marchetti (1980). The E_D has been measured at mid-height of CC specimen.
- Tangent constrained modulus M_t measured on the CC specimen at the stress level at which q_c and E_D have been measured.
- E_s' corresponding to the secant drained Young modulus inferred from CK_oD triaxial compression tests performed on pluvially deposited TS sample in a Bishop–Wesley triaxial cell. The adopted values refer to the effective mean consolidation stress of the CC specimens and to a level of axial strain $\bar{\epsilon}_a$ equal to 0.1.%.

This value corresponds to the upper limit of the average vertical strain of practical interest when designing shallow and deep foundations in cohesionless soils (Jardine et al., 1986; Battaglio & Jamiolkowski, 1987). Substantially analogous results have been obtained for HS.

A review of Figures 3 and 4 reveals some common features that can be summarized as follows:

- Even for the same sand the ratio of RDM to the penetration test results (q_c, M_D) is substantially higher for mechanically OC specimens than for NC specimens. This trend is observed in all sands and for all kinds of penetration tests validated in CC's around the world. It reflects the very high sensitivity of RDM and the low sensitivity of penetration test results to the strain and stress history of the sand as shown by the special CC tests mentioned before.
- The ratio decreases, increasing D_R which, in first approximation, can be expressed as $q_c/\sqrt{\sigma'_{vo}}$ or $P'_o/\sqrt{\sigma'_{vo}}$. This reflects the different influence of D_R on RDM and penetration test results.

The above statements result from CC studies performed on freshly deposited silica sands. Further research is needed to evaluate to what extent these findings are applicable to natural and not necessarily silica sands.

For natural aged sand deposits (i.e. age ≥1,000 years) the writers believe that the correlations between PR and RDM, for example E'_s and q_c, may lie somewhere between the correlations developed for NC and OC unaged sands in the CC studies, as shown in Figure 3. In this figure the trend shown for aged NC sands has been developed assuming that the ratio E'_s/q_c might correspond to that of slightly OC ($1<OCR \leq 1.5$) pluvially deposited TS but further research is needed to validate this tentative working hypothesis. The results of two screw plate loading tests performed in a Po river sand deposit at least ≈3,000 years old seem to confirm the above assumption, see Figure 3. In view of the above statements it appears obvious that, for the same sand, no unique correlation exists between penetration resistance and non-linear deformation moduli. The non-linear moduli are defined here as the moduli at a strain level greater than the elastic threshold strain (Hardin, 1978). Below this strain ($\approx 10^{-5}$) the soil deformation moduli (small strain moduli) are practically constant and equal to the maximum moduli.

4 CORRELATION BETWEEN G_o AND CPT AND DMT RESULTS

Correlations between the small strain shear modulus (G_o) and PR (Ohta & Goto, 1978; Sykora & Stokoe, 1982; Robertson & Campanella, 1983; Rix, 1984; Bellotti et al., 1986; Lo Presti & Lai, 1988; Jamiolkowski & Robertson, 1988; Baldi et al., 1988a) show less uncertainties than those examined in the previous section. A large amount of experimental data show that G_o in cohesionless soils is influenced very little by the stress and strain history. For a given sand G_o

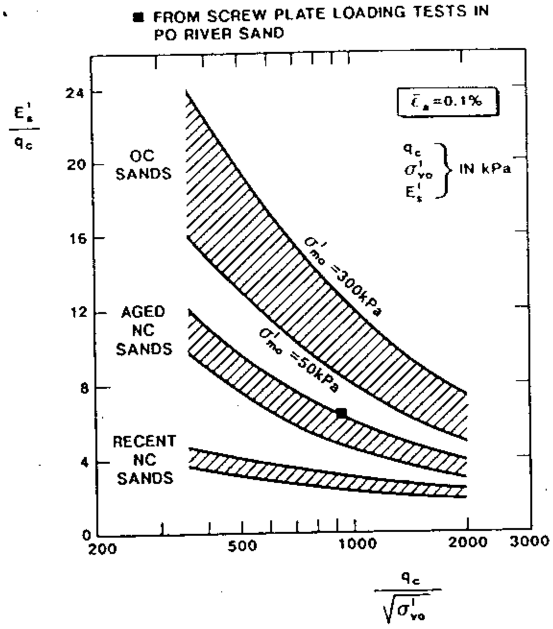


Figure 3. Evaluation of drained Young's modulus from CPT for silica sands.

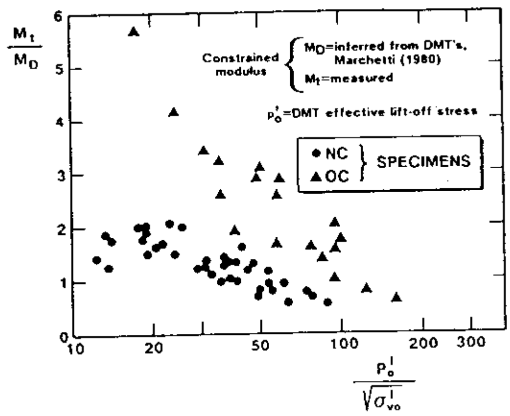


Figure 4. Constrained modulus of Ticino sand from DMT's performed in calibration chamber.

is mostly a function of the following variables (Lee & Stokoe, 1986):

$$G_o = f\,(D_R,\ \sigma'_a,\ \sigma'_b) \qquad \ldots (1)$$

where:

σ'_a = effective stress acting in direction of seismic wave propagation, and

σ'_b = effective stress acting in direction of soil particle displacement.

On the other hand, the same basic variables (D_R, σ') influence PR and DMT indices. This might explain the good performance of the Ohta & Goto (1978) formula (or other similar) which relate N_{SPT} and shear wave velocity V_s. As demonstrated by Seed et al. (1986), Jamiolkowski et al. (1988) and Baldi et al. (1988a), this formula provides a reliable estimate of V_s limited to cohesionless holocene deposits. Figures 5 and 6 show the correlations between G_o measured during

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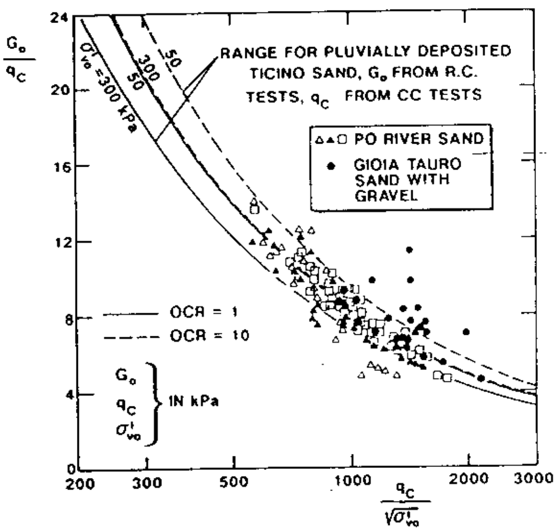

Figure 5. q_c vs. G_o correlation for uncemented predominantly quartz sands.

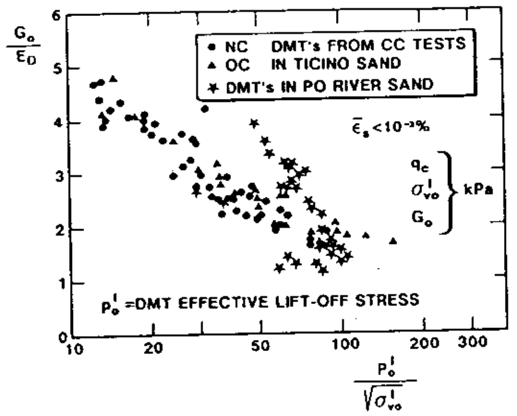

Figure 6. Evaluation of small strain shear modulus from DMT's for uncemented silica sands.

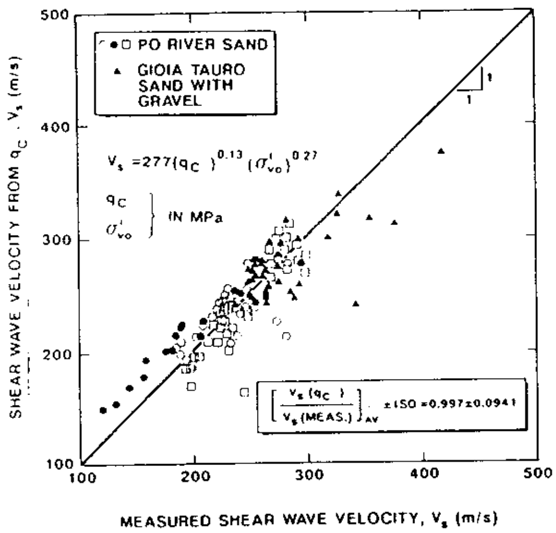

Figure 7. Shear wave velocity of uncemented silica sands from q_c vs. that measured.

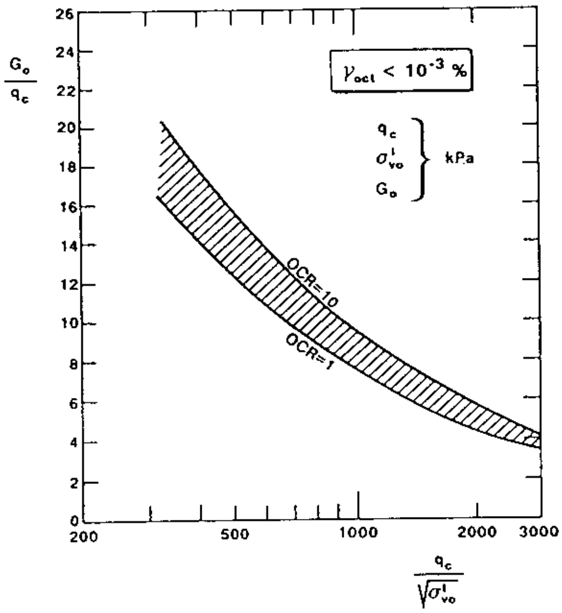

Figure 8. Evaluation of small strain shear modulus from CPT for uncemented silica sands.

resonant column tests (RCT) and the results of DMTs and CPTs performed in the CC, both related to the pluvially deposited silica TS. The same figures include also data from CPT (q_c), cross-hole (V_s) and seismic cone penetration (q_c, V_s) tests performed at the sites of Po river sand and of the Gioia Tauro sand with gravel. Good agreement between the measured V_s and those inferred from q_c for the above mentioned sites is indicated by the data shown in Figure 7 (Lo Presti & Lai, 1988). Although the data for TS were obtained on recently deposited, unaged sand the results in Figures 5 and 6 show good agreement with the data of natural cohesionless deposits having an age of ≈3,000 to ≈20,000 years at the maximum considered depth of ≈30 m. However Figure 6 shows more scatter between laboratory and field data than Figure 5. In view of the above statements, Figure 8 gives a tentative correlation between G_o and q_c for uncemented predominantly silica sands.

5 USE OF G_o IN SETTLEMENT PREDICTION

The application in practice of the correlations, like the one shown in Figure 8, depends strongly on the development of a link between the small strain shear modulus and soil moduli involved in the geotechnical design problems as strain level and mode of deformation are different (Swiger, 1977; Robertson, 1982; Bellotti et al., 1988b; Yoshida et al., 1984). To evaluate settlements of shallow foundations in cohesionless soils, the following simplified tentative procedure is suggested:

- Supposing, that in first approximation the theory of isotropic elasticity is valid, small

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strain Young's modulus can be computed as $E'_o = 2\,(1+\nu')\,G_o$, assuming a Poisson ratio ranging between 0.15 and 0.25 (i.e. 0.2), see Lade (1988).

- With the so obtained E_o, compute Young's modulus degradation vs ϵ_a using one of the available empirical rules such as the hyperbolic (Hardin & Drnevich, 1972) or Ramberg–Osgood (Ishihara, 1986) equations.
- Once the E'_s vs ϵ_a relationship has been established, the appropriate ($\epsilon_a < 0.1\%$) Young's modulus for settlement calculation can be selected.

It is extremely important to realize that the obtained E_s vs ϵ_a curve refers to the secant stiffness of soil after it has been subjected to a large number of unloading–reloading cycles. This means that:

- The obtained E'_s vs ϵ_a relationship is appropriate to describe the stiffness of sand in the OC range only.
- Even in this range, the above relationship reflects the stiffness of sand which has experienced plastic hardening during the first 10 to 30 loading cycles.
- The increase of sand stiffness during cyclic loading is strain level dependent. In the strain range of interest ($\epsilon_a \simeq 0.05\%$) this phenomenon can be responsible for an increase in E'_s of 25 to 50%, see Alarcon–Guzman et al. (1988).

It is postulated that the above outlined approach to infer E'_s from G_o can also be applied to the NC cohesionless deposits having age exceeding a few thousands years. In fact, the writers believe that due to the processes occurring in the natural cohesionless deposits such as: ageing, small strain cyclic stressing due to low amplitude earthquakes, weak cementation, etc., their behaviour matches with that of slightly mechanically OC deposits (see Figure 3).

To investigate the validity of the above mentioned approach for estimating E'_s from G_o, the results of a series of RCTs and TX–CK$_o$D compression tests, performed on the pluvially deposited TS and HS specimens, have been compared.

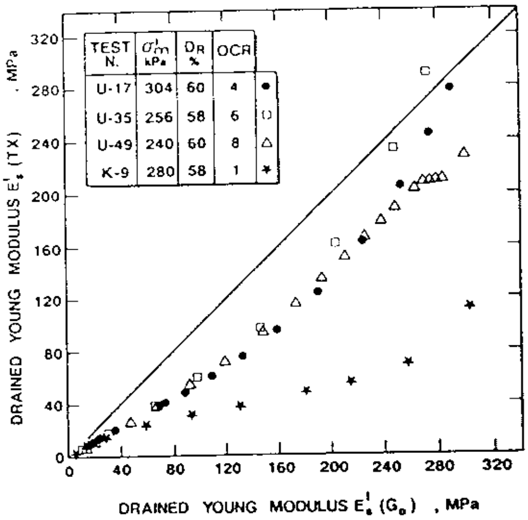
The comparison has been made at the same level of octahedral shear strain γ_{oct} and octahedral mean stress σ'_m, adopting $\nu'=0.20$. Degradation of shear modulus with strain have been simulated using Hardin & Drnevich (1972) hyperbolic stress–strain relationship. The results of this comparison are summarized in Table 2 and Figure 9. The Table 2 shows the ratio of E'_s(TX) to $E'_s(G_o)$ both evaluated at $\epsilon_a = 0.1\%$ which represents the upper limit of axial strain of practical interest. Figure 9 compares E'_s(TX) against $E'_s(G_o)$ at different strain levels for three OC and one NC specimens of TS.

What emerges from the Table 2 and Figure 9 can be summarized as follows:

- In OC sands E'_s(TX) at $\epsilon_a=0.1\%$ is about 20% lower than $E'_s(G_o)$. This fact might be: attributed to the following factors the simplified hypothesis adopted, the fact that $E'_s(G_o)$ reflects implicitly soil stiffness after a large number of loading cycles, and the contribution of irreversible strains to the assessment of E'_s(TX) ($\epsilon_a=0.1\%$) particularly at low values of OCR.
- In NC sand, as expected, the suggested approach to obtain E'_s from G_o does not work due to the reasons explained before.
- The comparison of E'_s(TX) with $E'_s(G_o)$ along the broad range of axial strains (0.05 to 5%) indicates a good agreement at lower strains which becomes worse with increasing ϵ_a. This again confirms the usefulness of the suggested procedure only when the plastic strains are small (e.g. when the E'_s is measured within the current yield surface).

Table 2. E'_s(TX) vs. E'_s(RC) at $\gamma_{oct} \simeq 0.2\%$.

OCR	$E'_s(TX)/E'_s(G_o)$	1SD	Notes
1	0.28	0.08	Ticino sand
2 to 8	0.84	0.22	
1	0.20	0.04	Hokksund sand
2 to 8	0.70	0.12	



(*) MADE AT EQUAL γ_{oct}:

$$\gamma_{oct}\,(TX) = \frac{2\sqrt{2}}{3}\,(1+\nu')\epsilon_a \qquad \gamma_{oct}\,(RC) = \frac{2\sqrt{2}}{3}\,\gamma$$

Figure 9. Comparison between E'_s(TX) and $E'_s(G_o)$ for Ticino sand.

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