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Experience with Seismic Dilatometer in Various Soil Types

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ABSTRACT: The seismic dilatometer (SDMT) is a combination of the standard DMT with a seismic module for measurement of the shear wave velocity V_S. This paper summarizes the experience gained from a large number of tests performed with the SDMT at several sites in the recent years and illustrates the main lessons learned from the use of the tool. In particular, the paper presents an overview of the SDMT equipment and test layout, comparisons of V_S measured by SDMT and by other methods and a selection of significant SDMT results and related comments. The paper also illustrates the major issues of present research on use and applications of the SDMT, mostly focused on the development of methods for deriving in situ decay curves of soil stiffness with strain level and for evaluating the liquefaction potential of sands based on SDMT results.

1. INTRODUCTION

The seismic dilatometer (SDMT) combines the traditional features of the flat dilatometer (DMT), introduced by Marchetti (1980), with the ability of measuring the shear wave velocity V_S. Initially conceived for research, the SDMT is gradually entering into use in current site investigation practice. The motivation of the development / diffusion of the SDMT stems from the following reasons:

– Increasing demand for seismic analyses, that require V_S as a basic and "multipurpose" input parameter. E.g. the technical seismic regulations newly introduced in Italy, according to the Eurocode 8, prescribe the measurement (or determination) of V_S – needed to classify the foundation soils and to identify the design seismic actions on structures – at all construction sites located in the seismic zones of the country (the majority).
– Recognition by researchers and designers of the importance of investigating the soil behaviour at small strains (V_S provides the small strain shear modulus G_0) and the non-linearity of stiffness with strain level (G-γ curves). Such assessment is today regarded as key point of the site characterization, required for many engineering applications, such as prediction of the site seismic response or analysis of the behaviour of complex geotechnical constructions (e.g. earth dams).
– Increasing demand for liquefaction analyses (e.g. re-evaluation of liquefaction hazard for nuclear power plants, tailing dams, etc.).
– Accuracy of settlement predictions (normal operative range of the flat dilatometer DMT).
– Availability of usual DMT results (e.g. constrained modulus M, undrained shear strength c_u, stress history OCR) for current design applications (monitoring soil improvement, design of laterally loaded piles, detecting slip surfaces, etc.).

In the period 2004-2006 over 30 construction sites were investigated using the SDMT, resulting in the accumulation of a large number of results. This paper presents comments on such results and illustrates the main lessons learned from the use of the tool. Also shown are validations of V_S measurements obtained by SDMT by comparisons with V_S measured by other methods at well documented research sites.

Use and current applications of the traditional "non-seismic" flat dilatometer (DMT), illustrated in several papers available in the literature, are just mentioned in this paper, limited to the most important design application (settlement prediction). A general overview of the DMT equipment, testing procedure, interpretation and design applications can be found in the comprehensive report by the ISSMGE Technical Committee TC16 (2001).

2. THE SEISMIC DILATOMETER (SDMT)

The seismic dilatometer (SDMT) is a combination of the standard DMT equipment with a seismic module for the down-hole measurement of the shear wave velocity V_S. The test is conceptually similar to the seismic cone SCPT (Robertson et al. 1985).

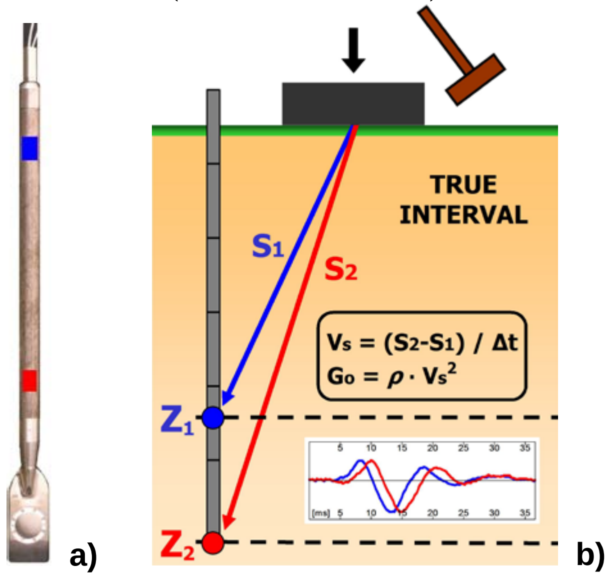


Fig 1. (a) DMT blade and seismic module. (b) Schematic layout of the seismic dilatometer test.

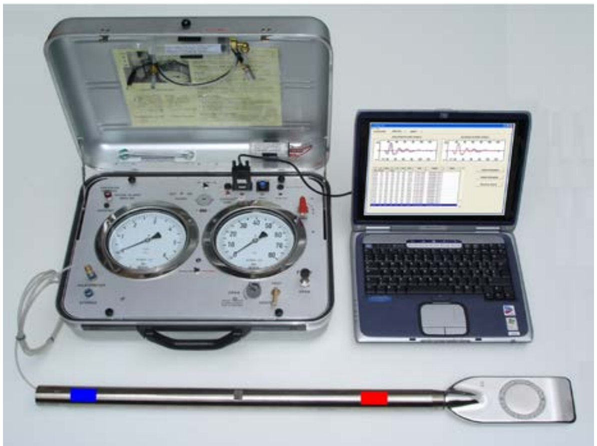


Fig. 2. Seismic dilatometer equipment

Fig. 3. Shear wave source at the surface

First introduced by Hepton (1988), the SDMT was subsequently improved at Georgia Tech, Atlanta, USA (Martin & Mayne 1997, 1998; Mayne et al. 1999).

A new SDMT system has been recently developed in Italy. The basic choices guiding in the development of this tool were:

– Two-receiver "true-interval" system.
– Signal amplified and digitized at depth.
– No hole required (independent from operator and interpreter).

Fig. 1 shows a schematic layout of the SDMT equipment (see also Fig. 2). The seismic module (Fig. 1a) is a cylindrical element placed above the DMT blade, equipped with two receivers located at 0.5 m distance. The signal is amplified and digitized at depth. The "true-interval" test configuration with two receivers avoids possible inaccuracy in the determination of the "zero time" at the hammer impact, sometimes observed in the "pseudo-interval" one-receiver configuration. Moreover, the couple of seismograms recorded by the two receivers at a given test depth (Fig. 1b) corresponds to the same hammer blow and not to different blows in sequence, not necessarily identical. Hence the repeatability of V_S measurements is considerably improved (observed V_S repeatability ≈ 1 m/s). The shear wave velocity V_S (Fig. 1b) is obtained as the ratio between the difference in distance between the source and the two receivers (S_2 - S_1) and the delay of the arrival of the impulse from the first to the second receiver (Δt). V_S measurements are obtained every 0.5 m of depth.

The shear wave source at the surface is a pendulum hammer (Fig. 3), of ≈ 10 kg weight, which hits horizontally a steel rectangular base pressed vertically against the soil and oriented with its long axis (≈ 0.8 m) parallel to the axis of the receivers, so that they can offer the highest sensitivity to the generated shear wave. The proper hammer orientation should be such that a line from the rods to the centre of the source is perpendicular to the longitudinal axis of the source. However, experience has shown that a slightly different hammer orientation has a small influence on the test results. The weight of the truck is transmitted to the anvil, ensuring good contact with the soil. The horizontal hammer blow is not transmitted up to the truck (no energy is wasted to "accelerate" the truck).

The determination of the delay from the seismograms obtained by SDMT is generally well-conditioned. Fig. 4 shows an example of seismograms obtained by SDMT at various test depths at the site of Fucino (Italy). In the authors' experience, it is a good practice to plot side-by-side the seismograms as recorded and re-phased according to the calculated delay, as shown in Fig. 4. Even in cases where the seismograms obtained by SDMT are less regular, as in the example shown in Fig. 5, the re-phasing is always well-conditioned.

Fig. 6 (SDMT results at the Fucino site) is an example of the typical graphical format of the SDMT output currently used today. Such output displays the profiles of four basic DMT parameters – the material index I_D (soil type), the constrained modulus M, the undrained shear strength c_u and the horizontal stress index K_D (related to stress history), obtained using current DMT correlations (Marchetti 1980) – and the profile of the shear wave velocity V_S measured by SDMT.

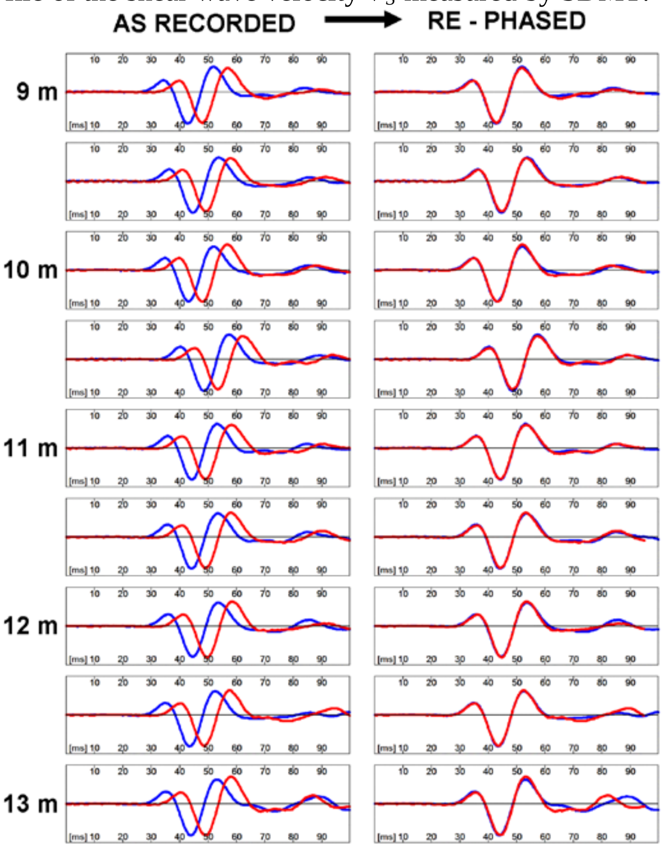


Fig. 4. Example of seismograms obtained by SDMT at various test depths at the site of Fucino (Italy) – as recorded and re-phased according to the calculated delay

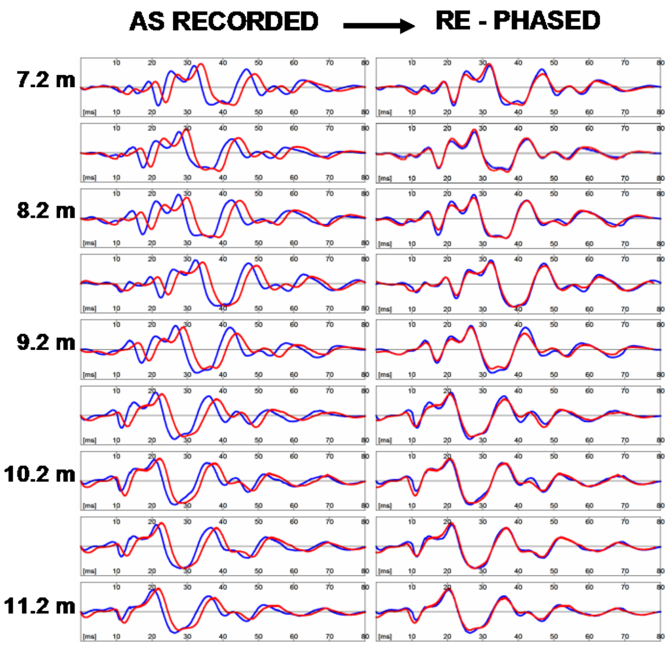


Fig. 5. Example of seismograms obtained by SDMT at various test depths at the site of Avezzano – Castello Orsini (Italy) – as recorded and re-phased according to the calculated delay

Table 1. Example of repeatability of V_S measurements by SDMT (Zelazny Most tailing dam, Poland)

Z [m]	V_S [m/s]	V_S values [m/s] corresponding to different hammer blows at each depth Z	Coefficient of variation [%]
7.00	179	178,178,180,180,180,179,179,180,180,180	0.50
7.50	231	234,232,232,230,229,231,232,229,230	0.68
8.00	225	227,225,224,225,225,225,226,226,225,224,224	0.40
8.50	276	276,276,280,273,275,273,271,273,287,281	1.68
9.00	296	291,286,301,292,296,288,301,300,304,303	2.09
9.50	248	244,251,250,247,250,249,250,249,242,248	1.11
10.00	292	292,289,290,293,289,292,289,292,296,295,293	0.79
10.50	320	321,323,320,325,323,325,316,314,308,321	1.61
11.00	291	293,291,293,291,291,290,290,291,290,290	0.38
11.50	321	324,320,320,322,320,322,319,319,320,320	0.48
12.00	309	311,307,311,309,309,311,309,309,307,311	0.50
12.50	286	287,285,285,285,287,285,285,287,287,287	0.35
13.00	265	264,265,265,265,264,265,265,265,266,265,266,264	0.24
13.50	280	287,276,279,276,276,276,294,275,278,279	2.08
14.00	312	313,312,312,322,310,312,310,310,310,312	1.10
14.50	298	301,298,299,299,298,296,299,298,299,298	0.44
15.00	309	307,309,307,309,309,309,309,309,309,309	0.29

3. REPEATABILITY OF V_S BY SDMT

Table 1 shows an example of repeatability of V_S measurements obtained by SDMT at the site of the Zelazny Most tailing dam (Poland). Each V_S value at a given test depth Z corresponds to a different hammer blow.

Today state-of-the-art in V_S repeatability is a few m/s scatter.

4. COMPARISONS OF V_S BY SDMT AND V_S BY OTHER TESTS

V_S measurements obtained by SDMT have been validated by comparison with V_S obtained by other in situ seismic tests (SCPT, cross-hole, SASW) at various research sites.

The first comparison (Fig. 7) was presented by Hepton (1988), who found good agreement between V_S profiles obtained by SDMT, SCPT and seismic refraction tests at the well-known clay research test site of Bothkennar (UK).

Seismic dilatometer tests ("true-interval" and "pseudo-interval" SDMT) and seismic piezocone tests (SCPTU) were performed in 2002 by the Georgia Tech research group (McGillivray & Mayne 2004) at the research site of Treporti, Venice (Italy). The profiles of V_S obtained by SDMT and by SCPTU (Fig. 8) were found in good agreement.

Seismic dilatometer tests were performed in 2004 at the site of Fucino (Italy), a well-documented NC clay research test site, extensively investigated at the end of the '80s by means of several in situ and laboratory tests carried out by various research groups. The comparison in Fig. 9 shows that the profile of V_S obtained by SDMT at the Fucino site is in good agreement with V_S obtained by SCPT, cross-hole and SASW in previous investigations (AGI 1991).

Fig. 10 (Młynarek et al. 2006) shows good agreement of the profiles of G_0 obtained from V_S measurements by SDMT and by SCPTU at the site of the Zelazny Most tailing dam (Poland).

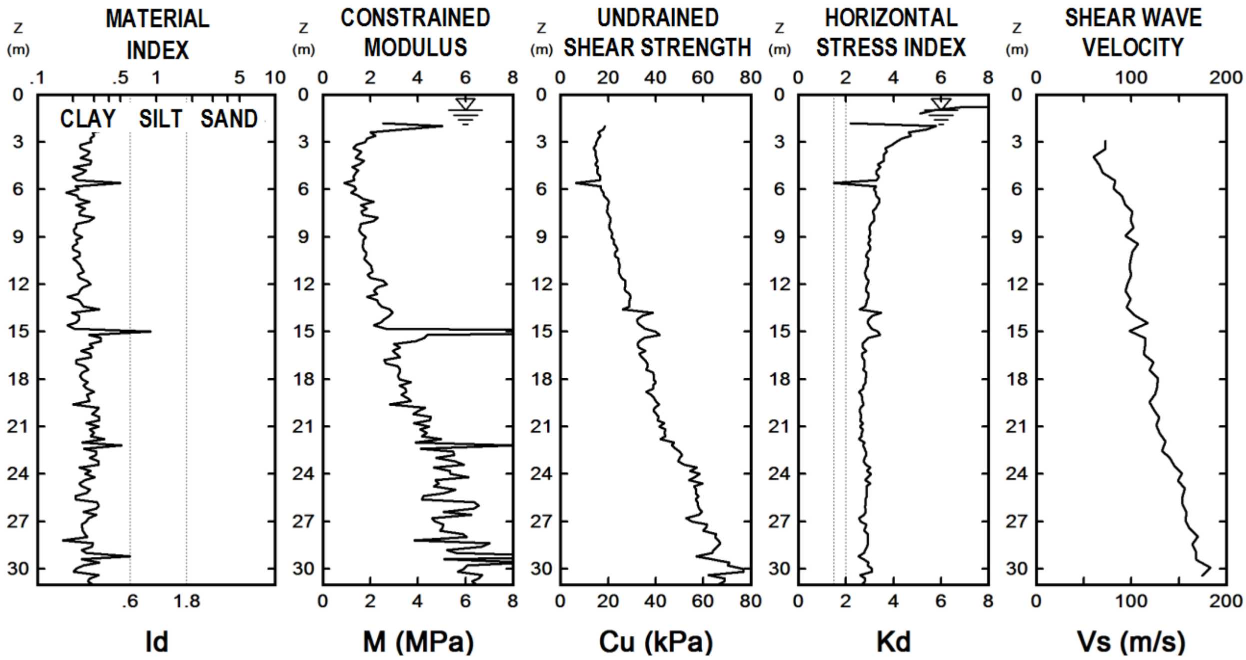


Fig. 6. SDMT profiles at the site of Fucino (Italy)

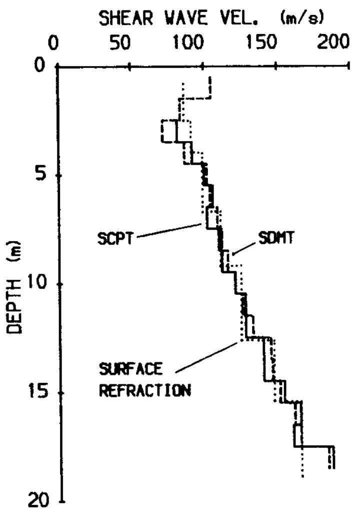


Fig. 7. Comparison of V_S profiles from SDMT, SCPT and seismic refraction tests at the research site of Bothkennar, UK (Hepton 1988)

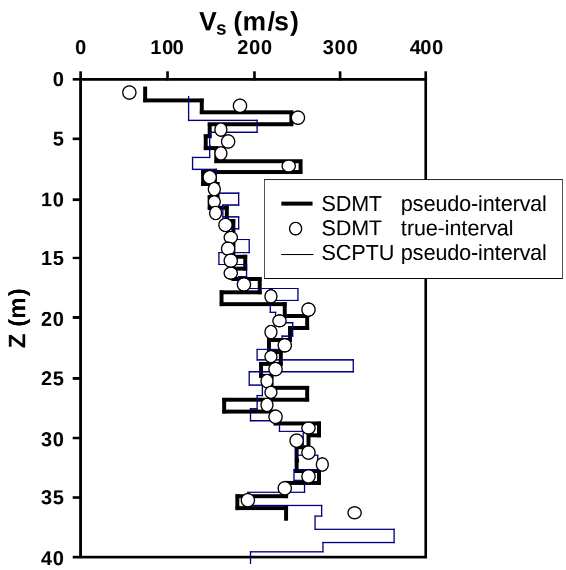


Fig. 8. Comparison of V_S profiles obtained by SDMT and by SCPTU at the research site of Treporti (Venice), Italy (McGillivray & Mayne 2004)

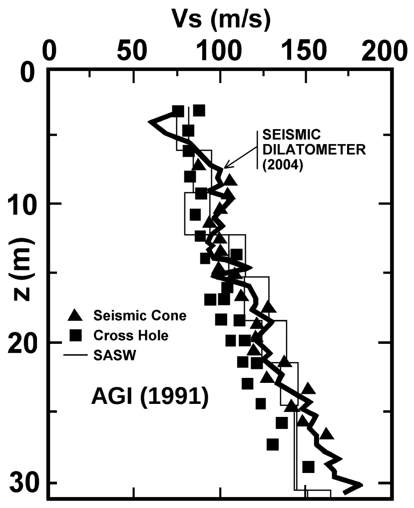


Fig. 9. Comparison of V_S profiles obtained by SDMT and by SCPT, cross-hole and SASW (AGI 1991) at the research site of Fucino, Italy

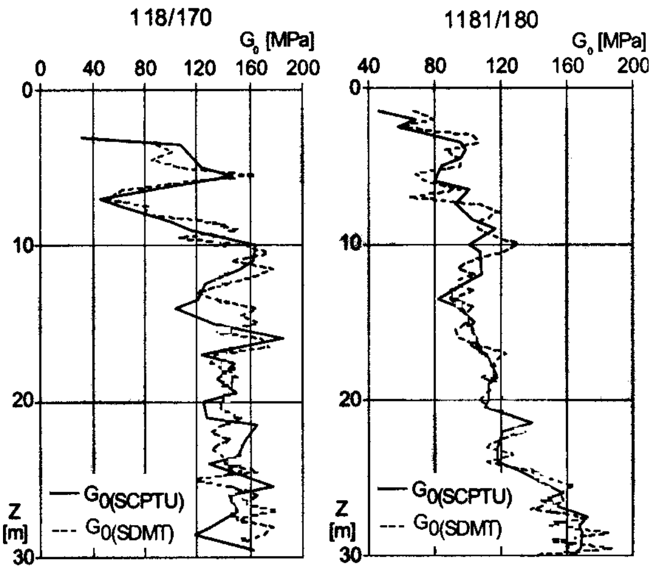


Fig. 10. Comparison of G_0 profiles obtained from V_S by SDMT and by SCPTU at the Zelazny Most tailing dam site, Poland (Młynarek et al. 2006)

5. IN SITU DECAY CURVES OF SOIL STIFFNESS WITH STRAIN LEVEL

One important peculiarity of the SDMT, compared to other tests which provide measurements of V_S, is that SDMT determines, besides the small strain shear modulus G_0 (from V_S), a modulus at "working strains" (relevant to settlements of foundations under working loads). This objective was a major stimulus in the development of the SDMT.

Research currently in progress investigates the possible use of the SDMT for deriving "in situ" decay curves of soil stiffness with strain level (G-γ curves or similar). Such curves could be tentatively constructed by fitting "reference typical-shape" laboratory curves through two points, both obtained by SDMT (Fig. 11): (1) the initial shear modulus G_0 from V_S, and (2) a "working strain" modulus, corresponding to the DMT constrained modulus M_{DMT}.

This approach is expected to provide more realistic estimates compared to other methods proposed for deriving G-γ curves from SDMT (Mayne et al. 1999), since the second point for the curve-fitting (given the first point G_0) is not located "at failure", but in the range of "working strains" (i.e. the strain range of "well designed foundations").

In order to use M_{DMT} for locating the second point of the G-γ curve, it is necessary to know, at least approximately, the shear strain – i.e. the abscissa – corresponding to M_{DMT}. The following indications have been advanced so far.

Mayne (2001) observed that correlations developed between some in situ tests (e.g. pressuremeter PMT, DMT) and performance monitored data of full-scale structures, or reference laboratory values, provide a modulus "somewhere along the stress-strain-strength curve" (Fig. 12), generally at an "intermediate" level of strain (≈ 0.05-0.1 % in Fig. 12). A similar indication is given in Fig. 13 (Ishihara 2001), where the DMT is classified within the group of methods of measurement of soil deformation characteristics involving an intermediate level of strain (0.01-1 %).

The above indications suggest that the shear strain range corresponding to M_{DMT} is ≈ 0.05-0.1 % to 1 %. This observation, supplemented by further investigations, could possibly help develop criteria for deriving in situ curves of decay of soil stiffness with strain level from SDMT.

6. BEST DESIGN APPLICATION OF THE "NON-SEISMIC" DMT: SETTLEMENT PREDICTION

Predicting settlements of shallow foundations is probably the No. 1 application of the DMT, especially in sands, where undisturbed sampling and estimating compressibility are particularly difficult.

Settlements of shallow foundations using DMT are generally calculated by means of the traditional linear elastic approach, e.g. by the classic 1-D method (Fig. 14), with stress increments $\Delta\sigma_v$ calculated by elasticity theory (Boussinesq) and soil moduli (constrained modulus M_{DMT}) determined from DMT. The calculated settlement is meant to be the settlement in "working conditions", i.e. for a safety factor Fs ≈ 2.5 to 3.5.

Details on the methods for settlement calculation by DMT and a review of several case histories including comparisons of DMT-predicted vs. measured settlements can be found in Monaco et al. (2006).

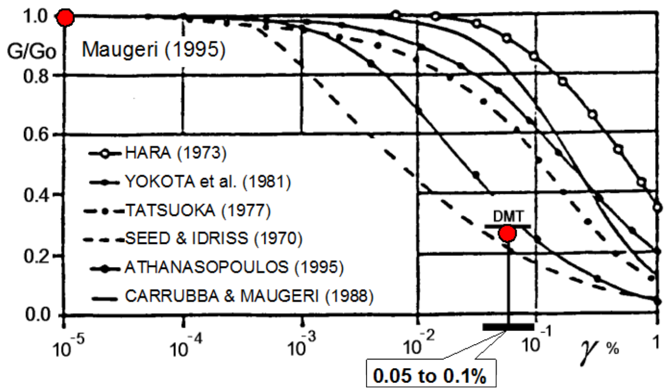


Fig. 11. Tentative method for deriving G-γ curves from two points obtained by SDMT

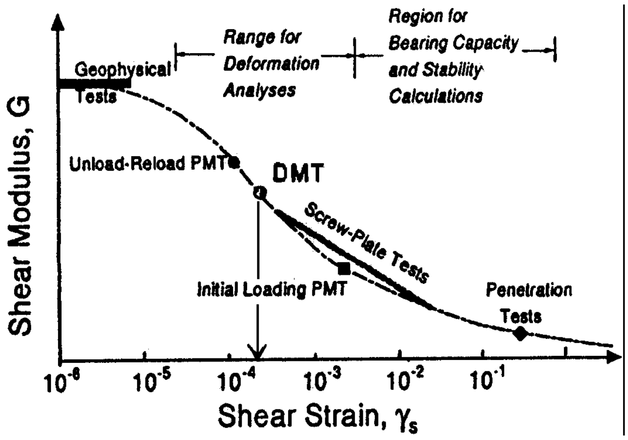


Fig. 12. Decay of shear modulus with strain level and possible strain range of moduli from various in situ tests (Mayne 2001)

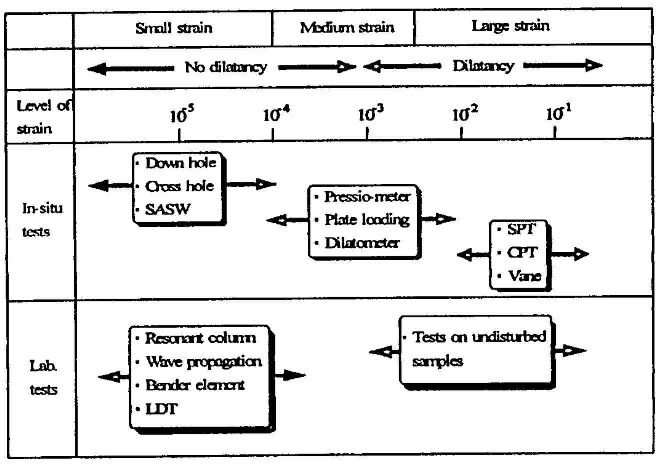


Fig. 13. Classification of methods of measurement of soil deformation characteristics according to the strain level involved (Ishihara 2001)

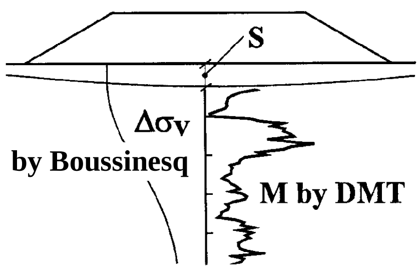
Settlements predicted by DMT have been generally found in good agreement with the observed settlements for a wide range of soil types (including sands, silts, clays and organic soils), settlements (from a few mm to over 300 mm) and footing sizes (from small footings to large rafts and embankments). The average ratio DMT-calculated/observed settlement for all the documented cases reviewed by Monaco et al. (2006), summarized in Fig. 15, is ≈ 1.3. The band amplitude (ratio between maximum and minimum) of the datapoints in Fig. 15 is less than 2, i.e. the observed settlement is within ± 50 % from the DMT-predicted settlement.

Fig. 16 shows a comparison of moduli M_{DMT} and M back-calculated from measurements of local vertical strains obtained by high-accuracy multiple extensometers, at 1 m depth intervals, under the centre of a full-scale instrumented test embankment (40 m diameter, 6.7 m height, applied load 104 kPa) constructed at the site of Treporti, Venice (Italy) on highly stratified, predominantly silty soils (Marchetti et al. 2006). The comparison shows an overall satisfactory agreement between M_{DMT} and moduli backfigured from the test embankment performance.

The available experience indicates that M_{DMT} can be considered a reasonable "working strain" modulus, i.e. introduced into the traditional elasticity theory formulae predicts settlements with reasonably good accuracy for foundations in "working conditions".

Possible reasons of the superior accuracy of settlement predictions by DMT compared to other penetration tests, documented by several studies (e.g. DMT vs. SPT, Bullock & Failmezger 2004, Fig. 17), are believed to be:

– M_{DMT} routinely takes into account overconsolidation and possible high lateral stresses (incorporated via the stress history parameter K_D), that reduce considerably soil compressibility. The necessity of stress history for a realistic assessment of settlements has been emphasized by many researchers (e.g. Leonards & Frost 1988, Massarsch 1994).
– The wedge-shaped tip deforms the soil considerably less than conical tips (Fig. 18, Baligh & Scott 1975).
– The modulus obtained by expanding a membrane (a "mini load test") is more closely correlated to in situ soil modulus than a penetration resistance.



$$S_{1-DMT} = \sum \frac{\Delta\sigma_v}{M_{DMT}} \Delta z$$

Fig. 14. Recommended method for settlement calculation using DMT

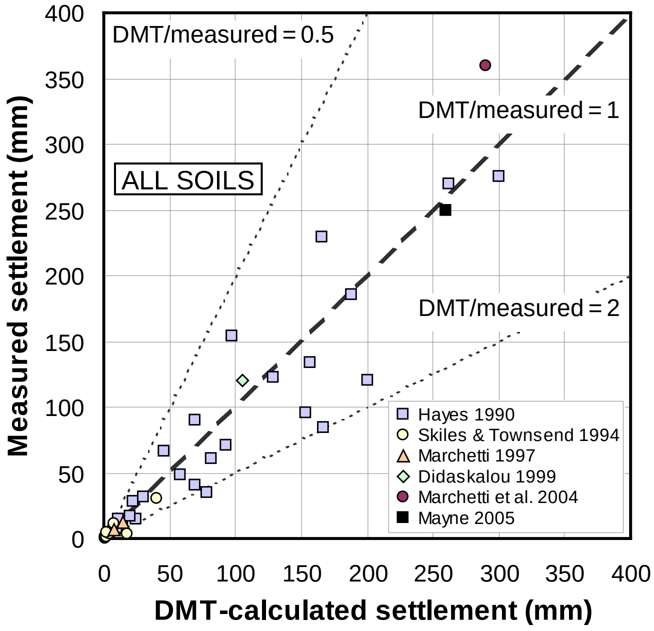


Fig. 15. Summary of comparisons of DMT-predicted vs. observed settlements (Monaco et al. 2006)

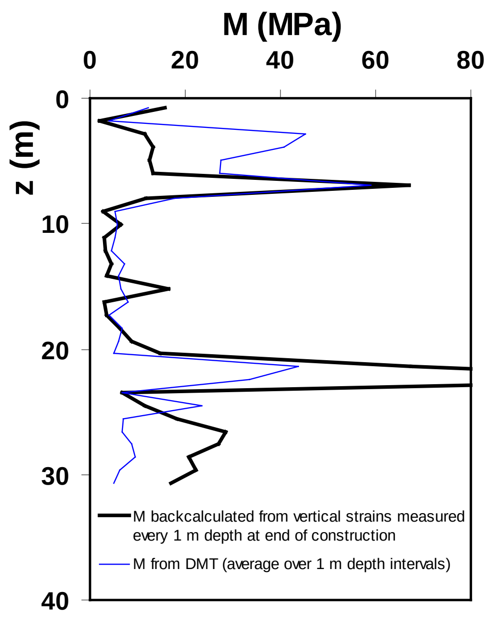


Fig. 16. Comparison of M_{DMT} vs. M backcalculated from from local vertical strains measured at 1 m depth intervals under the centre of the Treporti test embankment at the end of construction (Marchetti et al. 2006)

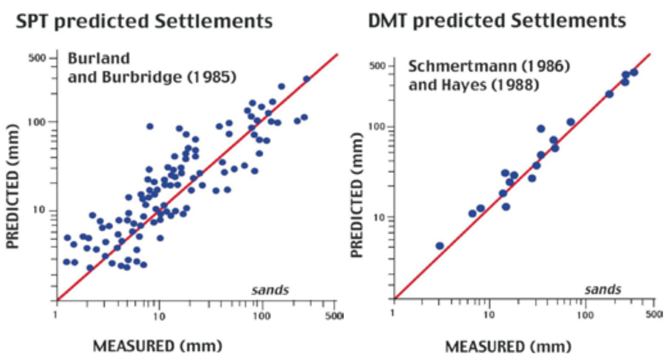


Fig. 17. Comparisons of settlements observed vs. predicted by SPT and by DMT (Bullock & Failmezger 2004)

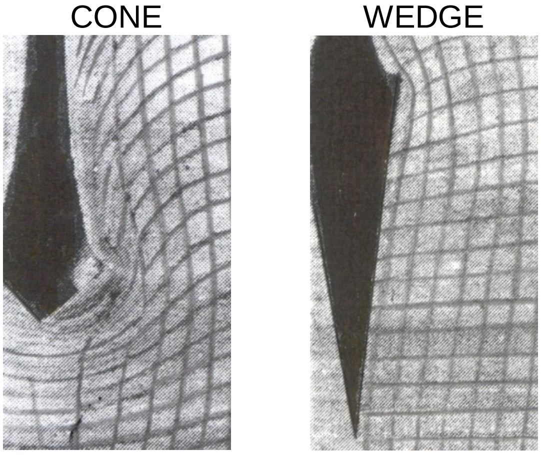


Fig. 18. Deformed grids comparing the distortions caused by conical tips and by wedges in clay (Baligh & Scott 1975)

7. RELATIONS G_0/E_D

Each SDMT sounding provides, at each test depth, pairs of values of the small strain shear modulus G_0 and the dilatometer modulus E_D. The large amount of data G_0 -E_D collected by SDMT in the recent years in various soil types permits to check if G_0 and E_D are correlated and to investigate the possible use of the ratio G_0/E_D.

Correlations E_D to G_0 have been proposed by many researchers. E.g. Tanaka & Tanaka (1998) found in four NC clay sites (where $K_D \approx 2$) $G_0/E_D \approx 7.5$. They also found in three sand sites that G_0/E_D decreases as K_D increases (from $G_0/E_D \approx 7.5$ for K_D = 1.5-2 to $G_0/E_D \approx 2$ for $K_D > 5$). Similar trends in sands had been observed by Sully & Campanella (1989) and Baldi et al. (1989). Indications on G_0/E_D were also given by Lunne et al. (1989), Hryciw (1990), Baldi et al. (1991), Cavallaro et al. (1999), Ricceri et al. (2001).

Such correlations G_0 -E_D were generally aimed at estimating G_0 from E_D, in absence of V_S measurements. This purpose appears today less important, given the increasing diffusion of the SDMT, which measures V_S directly. Today it could be possibly of greater interest to investigate the ratio of the small strain modulus G_0 to the "working strain" modulus M_{DMT}, in view of the possible use of both for deriving in situ G-γ curves from SDMT (see Section 5). However, since M_{DMT} is obtained by applying to E_D the correction factor R_M, depending primarily on K_D, M_{DMT} incorporates the effects of stress history (OCR). On the other hand, the uncorrected modulus E_D lacks information on stress history. Since G_0 is scarcely sensitive to OCR, as demonstrated by various studies (e.g. Yamashita et al. 2000), it appears appropriate to investigate the possible use of the ratio G_0/E_D.

Fig. 19 shows the variation of the ratio G_0/E_D as a function of the material index I_D (soil type) for various ranges of the horizontal stress index K_D (stress history). Fig. 19 indicates a large dispersion of the G_0/E_D datapoints in clay. By contrast in sand the ratio G_0/E_D ($\approx$ 2-3) is nearly constant, lower than in clay and independent on K_D (OCR).

Fig. 20 shows the variation of the ratio G_0/E_D as a function of K_D for various soil types. Similarly to Fig. 19, Fig. 20 indicates that the ratio G_0/E_D in sand is not influenced by stress history (K_D), while in clay the ratio G_0/E_D decreases as K_D (OCR) increases.

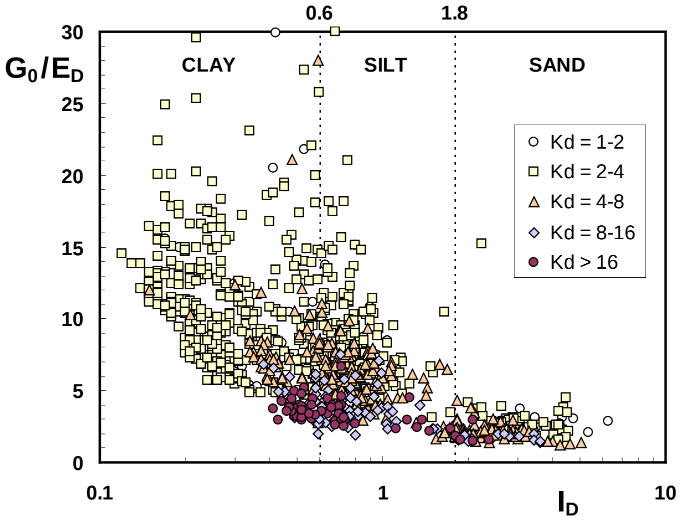


Fig. 19. Ratio G_0/E_D vs. I_D (soil type) for various ranges of K_D (OCR)

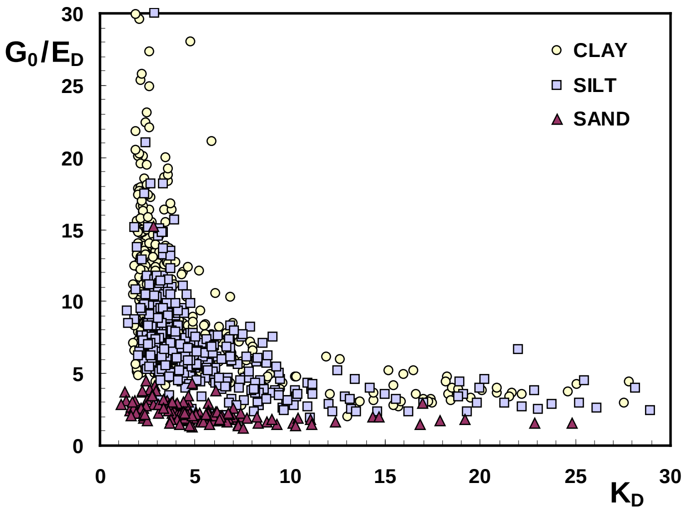


Fig. 20. Ratio G_0/E_D vs. K_D (OCR) for various soil types

8. USE OF SDMT FOR LIQUEFACTION

SDMT routinely provides, among other measurements, pairs of profiles of two parameters – the horizontal stress index K_D and the shear wave velocity V_S – that previous experience has indicated as bearing a significant relationship with the liquefaction resistance of sands. Hence SDMT permits to obtain two parallel independent estimates of liquefaction resistance CRR, one from K_D and one from V_S, using CRR-K_D and CRR-V_S correlations, where CRR is the cyclic resistance ratio – a basic input in the commonly used Seed & Idriss (1971) simplified procedure.

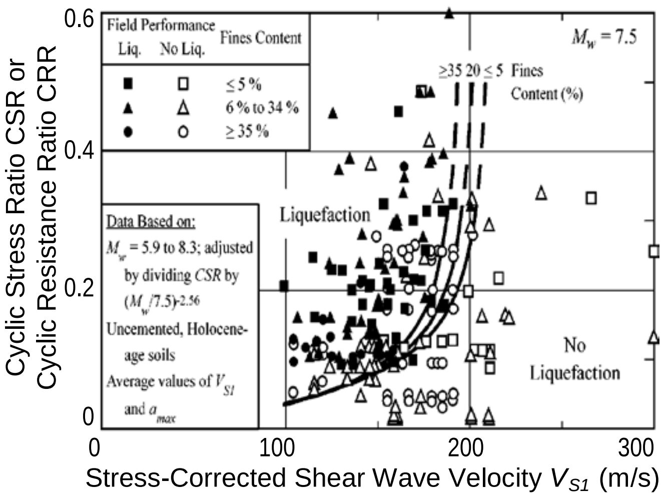


Fig. 21. Curves for evaluating CRR from V_S for clean uncemented soils with liquefaction data from compiled case histories (Andrus & Stokoe 2000)

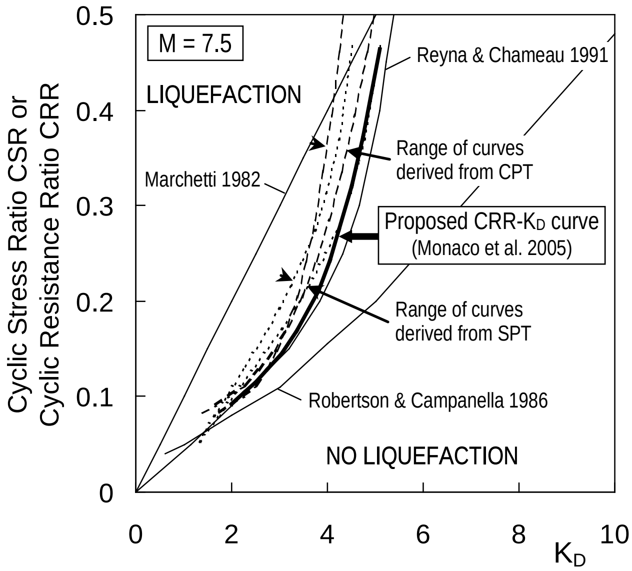


Fig. 22. CRR-K_D curves for evaluating liquefaction resistance from DMT (Monaco et al. 2005)

The use of V_S for evaluating CRR is well known. The most popular CRR-V_S correlation (Fig. 21) was proposed by Andrus & Stokoe (2000) for uncemented Holocene-age soils, and modified by Andrus et al. (2004) with the introduction of age correction factors for older soils. CRR is obtained as a function of $V_{S1} = V_S\,(p_a/\sigma'_{v0})^{0.25}$, shear wave velocity corrected for the overburden stress σ'_{v0} (p_a = atmospheric pressure). The CRR-V_{S1} curves in Fig. 21, for various fines contents, are for magnitude M_w = 7.5 earthquakes.

Correlations CRR-K_D have been developed in the last two decades, stimulated by the recognized sensitivity of K_D to a number of factors which are known to increase liquefaction resistance – difficult to sense by other tests – such as stress history, prestraining/aging, cementation, structure, and by the relationship of K_D to relative density and state parameter. A summary of the available knowledge on the subject and the latest version of the CRR-K_D correlation, based on all previous data, can be found in Monaco et al. (2005). Fig. 22 summarizes the various correlations developed to estimate CRR from K_D (for magnitude M = 7.5 and clean sand), to be used according to the Seed & Idriss (1971) simplified procedure. The convergence in a narrow band of the more recent CRR-K_D curves, compared to earlier curves, in Fig. 22 encourages the use of K_D to estimate CRR. However, since the CRR-K_D correlation is based on a limited real liquefaction case history database, considerable additional verification is needed.

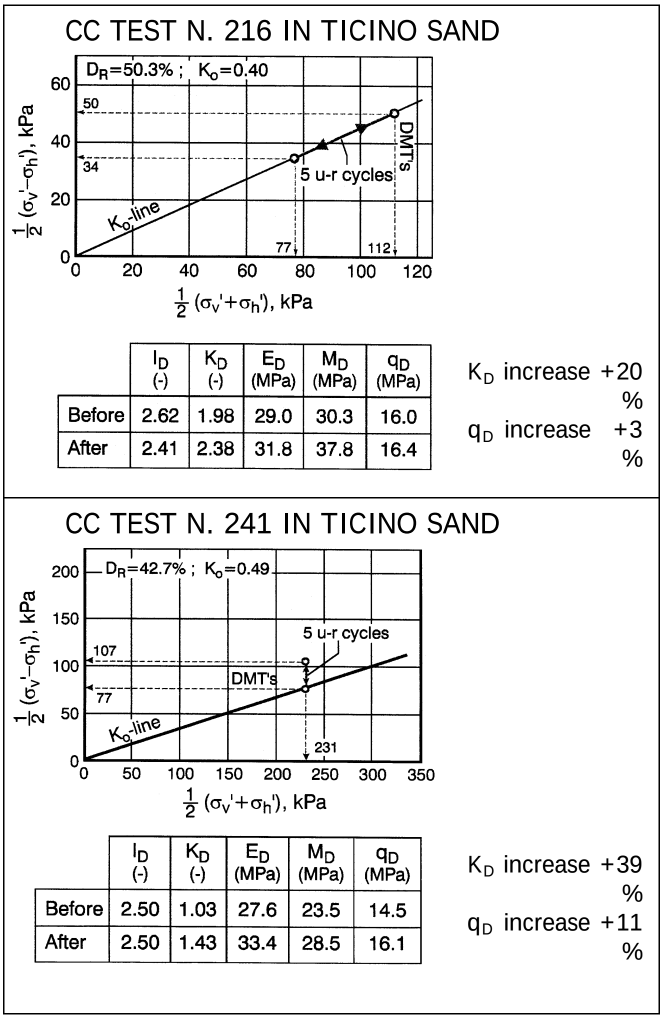
Maugeri & Monaco (2006) reported comparisons of CRR values predicted by CRR-K_D and CRR-V_S correlations, based on a large amount of parallel measurements of K_D and V_S obtained by SDMT at several sandy sites. They found that current methods based on K_D and V_S would provide, in general, substantially different estimates of CRR (generally CRR from V_S was found less conservative or "more optimistic" than CRR from K_D). This finding opens the question "which CRR should be given greater weight" when parallel analyses by K_D and V_S produce contradictory results. This point will be further discussed in the next Section.

Latest studies (Monaco & Schmertmann 2007, Monaco & Marchetti 2007) provide further insight into the ability of K_D to reflect *aging* in sands, a factor that recent research has indicated as having a first order of magnitude influence on liquefaction behaviour.

Leon et al. (2006) pointed out, as many did before (e.g. Pyke 2003), that commonly used correlations for estimating CRR (based on SPT, CPT, V_S) were derived mostly for young or freshly deposited sands – where the aging effect is negligible or small, anyway smaller than in older soils – and are not strictly valid in older sands. Leon et al. (2006) also remarked the poor ability of SPT and CPT to capture the effects of aging (disturbance during these tests may destroy the microstructure resulting from aging, that increases liquefaction resistance). Ignoring aging effects and using current CRR correlations, developed for young sands and based on in situ tests insensitive to aging, would result in many cases in a large underestimation of CRR (60 % less in the sand deposits studied by Leon et al. 2006) and in overconservative design.

As noted by Monaco & Schmertmann (2007), giving insufficient weight to aging, or disregarding aging, is equivalent to omitting a primary parameter in a CRR correlation. The omission of the parameter *aging* may possibly lead to overconservative CRR predictions and also largely contribute to the frequently observed dispersion of CRR values, ultimately leading to the generally accepted recommendation "evaluate CRR by as many methods as possible" (e.g. Youd & Idriss 2001).

A desirable alternative, seemingly better than relying on an "average" from correlations missing the aging factor, would be to use a testing tool significantly more sensitive to aging – in addition to other factors that are known to increase CRR.



	I_D (-)	K_D (-)	E_D (MPa)	M_D (MPa)	q_D (MPa)
Before	2.62	1.98	29.0	30.3	16.0
After	2.41	2.38	31.8	37.8	16.4

	I_D (-)	K_D (-)	E_D (MPa)	M_D (MPa)	q_D (MPa)
Before	2.50	1.03	27.6	23.5	14.5
After	2.50	1.43	33.4	28.5	16.1

Fig. 23. Calibration chamber test results (prestraining cycles) showing the higher sensitivity of K_D to prestraining than penetration resistance q_D (Jamiolkowski & Lo Presti 1998)

It is of interest to note that Jamiolkowski et al. (1985) had already pointed out, many years ago, that "reliable predictions of liquefaction resistance of sand deposits having complex stress-strain history would require the development of some new in situ device [other than CPT or SPT], more sensitive to the effects of past stress-strain histories".

Calibration chamber research work by Jamiolkowski & Lo Presti (1998) has shown that K_D is much more sensitive to cyclic prestraining – a sort of "simulated aging" (see Monaco & Schmertmann 2007) – than penetration resistance (Fig. 23). The increase in K_D caused by prestraining was found ≈ 3 to 7 times the increase in the penetration resistance q_D of the DMT blade, and presumably also of the CPT cone. On the other hand, it is well known that cyclic prestrain, just as aging, increases the liquefaction resistance, due to the similarity of the mechanism (at least for the mechanical "non-chemical" mechanism responsible of aging, consisting in the grains gradually slipping into a more stable configuration). Therefore the results of the above CC research suggest that K_D is much more sensitive to aging than penetration resistance.

A commonly accepted way to take into account the effects of aging is to correct current CRR correlations, developed for young soils, by means of correction factors depending on the age of the deposit. However specific factors should in general be developed for different soil deposits, because the CRR gain due to aging can depend on many ambient factors and thus can vary widely from site to site. It is possible that current CRR correlations based on K_D, or future refined versions, will not need the introduction of "age correction factors", because part of the aging effects are already "incorporated" in K_D.

9. SDMT RESULTS AT VARIOUS TEST SITES

This Section presents a selection of "commented examples" (for any possible use) of SDMT results obtained at various test sites.

Repeatability of SDMT results

Fig. 24 (Rome – Fiumicino) and Fig. 25 (Rome – Casilino) are examples of the high repeatability of the SDMT results (both usual DMT parameters and V_S measurements).

OCR and K_D crusts in sand

"Crust-like" K_D profiles, very similar to the typical K_D profiles found in OC desiccation crusts in clay, have been found at the top of most of the investigated sand deposits. An example (Catania, Italy) is shown in Fig. 26. Many indications (see Maugeri & Monaco 2006) suggest that "K_D crusts" in sands are "Stress History crusts" (reflecting OCR, cementation, aging and/or other effects), rather than "Relative Density crusts". Note in Fig. 26 that, while the existence of a shallow crust is well highlighted by the K_D profile, the profile of V_S is much more uniform and does not appear to reflect the shallow crust at all. Such capability of K_D to reflect stress history is important for liquefaction. The fact that "Stress History crusts" – believed by far not liquefiable – are unequivocally depicted by the high K_Ds, but are almost unfelt by V_S, suggests a lesser ability of V_S to profile liquefiability.

Role of the interparticle bonding

Fig. 27 shows SDMT profiles obtained at the site of Cassino (Italy). The Cassino data are somehow anomalous, in that relatively high V_S coexist with very low values of K_D and soil moduli M. Many volcanic sands in that area (pozzolana) are known to be active in developing interparticle bonding. A possible explanation could be the following. The shear wave travels fast in those sands thanks to the interparticle bonding, that is preserved at small strains. By contrast K_D is "low" because it reflects a different material, where the interparticle bonding has been at least partly destroyed by the blade penetration. As noted by Andrus & Stokoe (2000), one concern when using V_S to evaluate liquefaction resistance is that V_S measurements are made at small strains, whereas pore-pressure build up and liquefaction are medium- to high-strain phenomena. This concern is significant for cemented/bonded soils, because small-strain measurements are highly sensitive to weak interparticle bonding that is eliminated at medium-high strains (range of K_D measurement). Weak interparticle bonding can increase V_S, while not necessarily increasing CRR. Thus, for liquefiability, the K_D indications could possibly be more relevant. Very light earthquakes, however, may not destroy bonding, then CRR evaluated by V_S may be appropriate in this case.

Limiting V_{S1} and K_D values for liquefaction occurrence
Fig. 28 shows SDMT profiles obtained at the site of the Zelazny Most tailing dam (Poland). Another difference in the correlations CRR-V_S and CRR-K_D may be noted in the limiting values of V_{S1} and K_D for which liquefaction occurrence can be definitely excluded, even in case of strong earthquakes (asymptotes of the CRR-V_{S1} curve in Fig. 21 and CRR-K_D curve in Fig. 22). Such values are respectively V^*_{S1} = 215 m/s and K^*_D = 5.5 (see Maugeri & Monaco 2006), for clean sands and magnitude M_w = 7.5. At Zelazny Most, while V_{S1} values (mostly > 215 m/s) suggest "no liquefaction" for any earthquake, K_D values (≈ 1.5-2) indicate that liquefaction may occur above a certain seismic stress level (high cyclic stress ratio CSR).

Variability of the ratio G_0 / M_{DMT}
Fig. 29 shows the results of SDMT carried out in June 2006 at a site near El Prat Airport, Barcelona (Spain), in cooperation with Universidad Politécnica de Cataluña (UPC) and IGeoTest. Note in Fig. 29 that, while the "working strain" modulus M_{DMT} exhibits a dramatic drop at ≈ 12 m depth, at the transition from an upper stiff sand layer to a lower very soft clay layer, the profile of V_S is much more uniform and V_S shows only a slight decrease in the lower soft clay. This evidence is particularly interesting in view of the possible development of G-γ curves from SDMT (see Section 5).

As noted e.g. by Fahey (1999), despite the considerable advance of knowledge on non-linearity of soil stiffness and measurement of the non-linear stiffness parameters (including the development of advanced soil models, incorporated into sophisticated FEM computer programs), yet this level of sophistication will probably be used only for complex or important projects. At the other end of the spectrum, many projects will continue to be designed using linear elastic soil models. This then leads to the problem of how to choose an appropriate linear elastic stiffness such that the predicted deformations will be as close as possible to the correct values. A rule-of-thumb statement (Simpson 1999) is that "… most engineering calculations for the working state could safely be carried out using linear elasticity with stiffness set to 50 % of the very small strain value".

In many cases, such as in the example in Fig. 29, SDMT results have indicated that the profile of the small strain shear modulus G_0, derived from V_S, is much more uniform than the profile of the "working strain" modulus M_{DMT}. Hence the corresponding reduction of the ratio G/G_0 at "working strains" (presumably ≈ 0.05-0.1 %) appears highly variable. This finding casts doubts on the reliability of some current practice rules based on simply reducing the small strain modulus by a fixed percent factor. Further research work on this topic is needed.

Offshore SDMT
SMDT investigations have also been carried out successfully in offshore conditions, obtaining good results. An example of offshore SDMT results (Vado Ligure, Italy) is shown in Fig. 30. The V_S measurements were carried out operating the hammer at the sea bottom (see details in the photographs in Fig. 30).

SDMT inside backfilled boreholes
In cases where the soil is too hard to penetrate, or even in rock, SDMT can be carried out inside a backfilled borehole, according to the following procedure: (1) Drill a hole. (2) Fill it with sand. (3) Do SDMT (only seismic, no DMT).

In order to check the reliability of V_S measurements obtained by this procedure, parallel V_S measurements by SDMT have been carried out at the same site in the natural soil and in a backfilled borehole. The comparison in Fig. 31 (Montescaglioso – Ginosa, Italy) shows very good agreement of V_S profiles obtained by the two methods. This information could be useful for current practice. Measurements of V_S by SDMT can be obtained practically in any type of soil or rock, even when penetration is impossible.

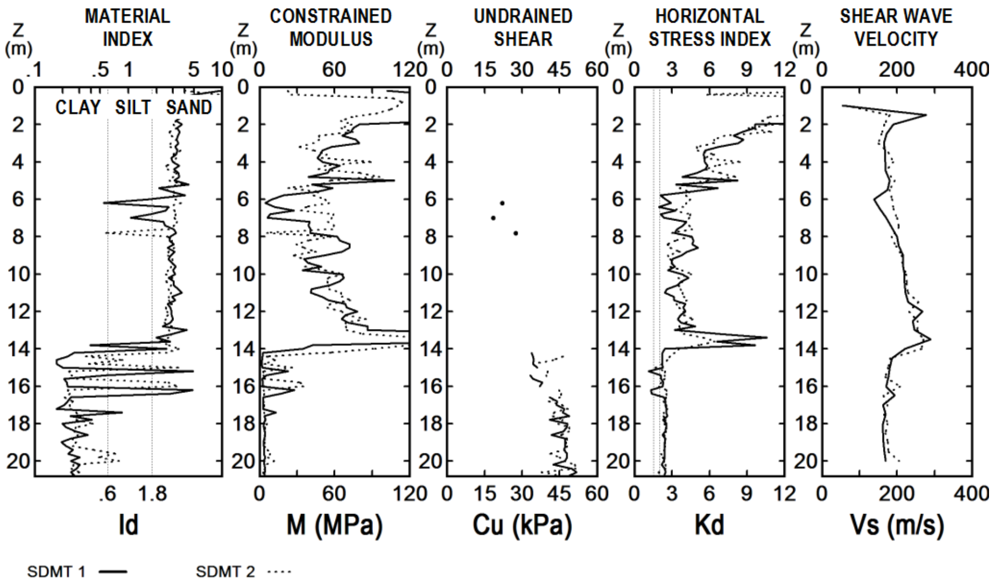


Fig. 24. Superimposed SDMT profiles at the site of Rome – Fiumicino (Italy)

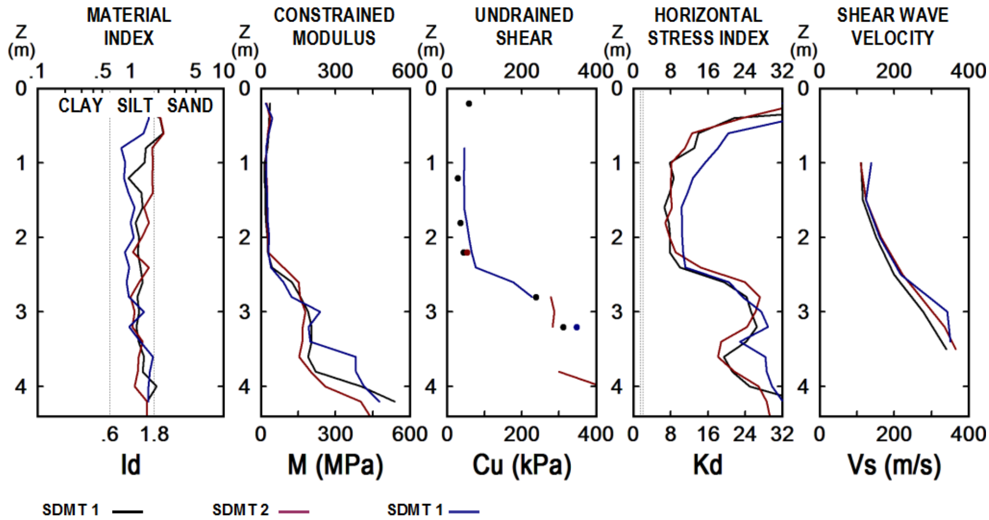


Fig. 25. Superimposed SDMT profiles at the site of Rome – Casilino (Italy)

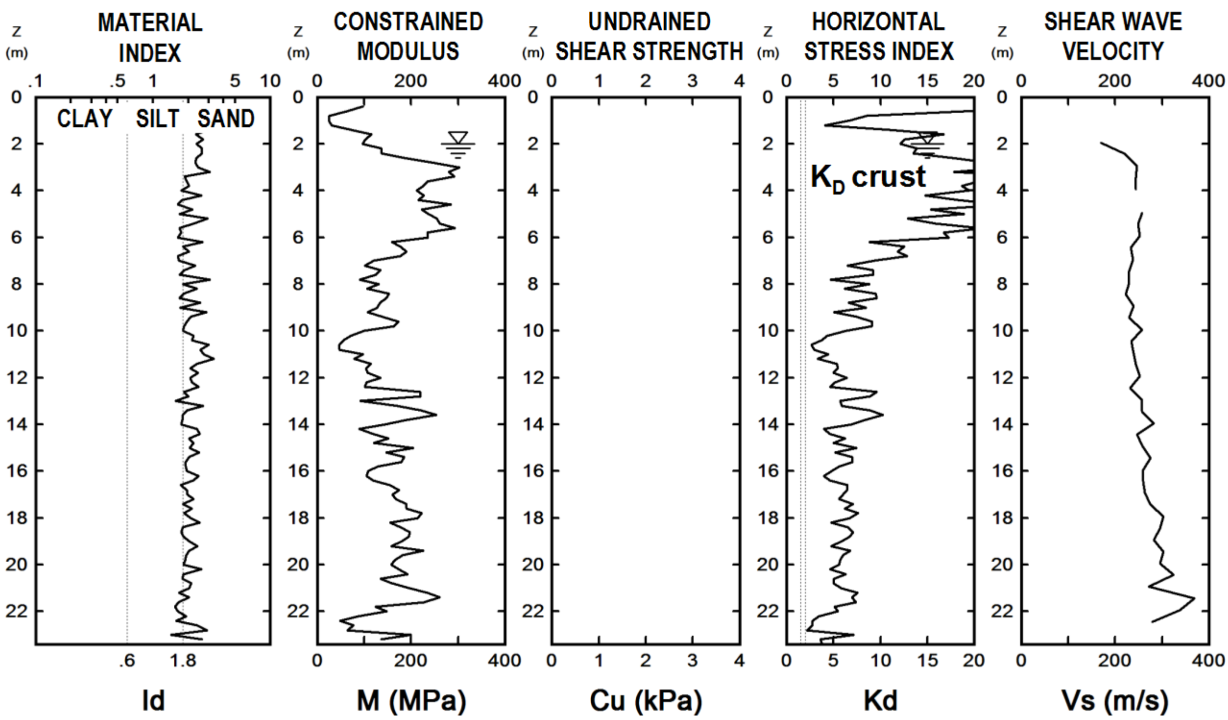


Fig. 26. SDMT profiles at the site of Catania – San Giuseppe La Rena (Italy)

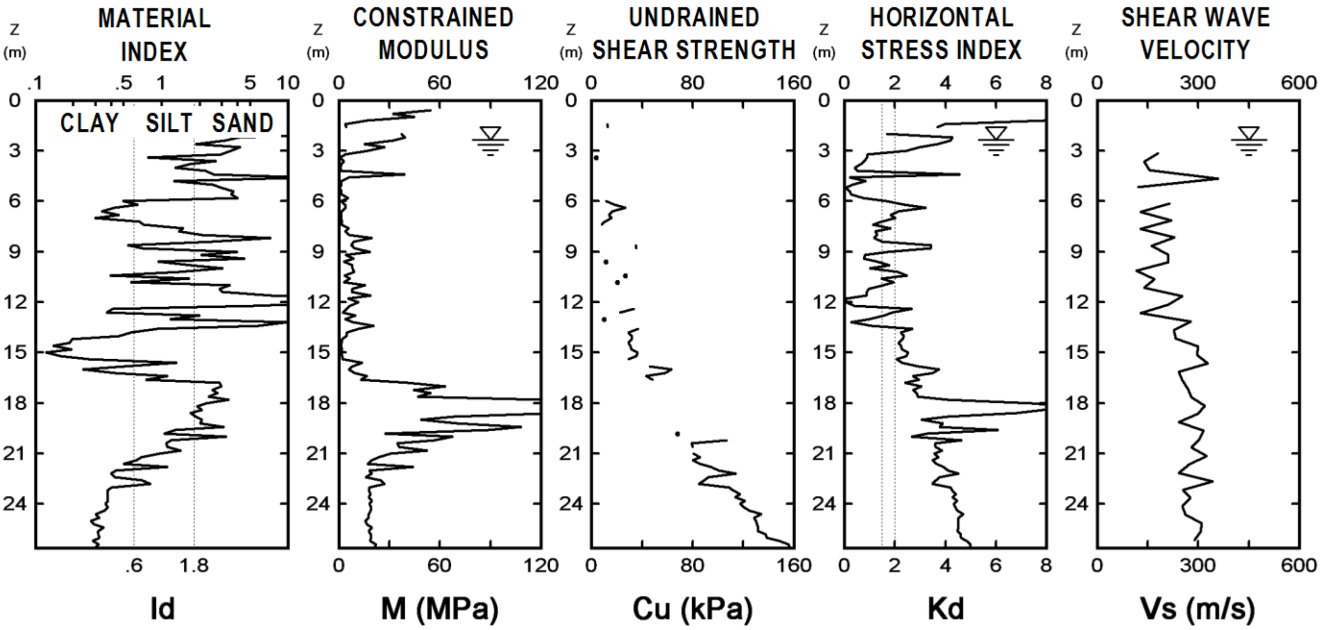


Fig. 27. SDMT profiles at the site of Cassino (Italy)

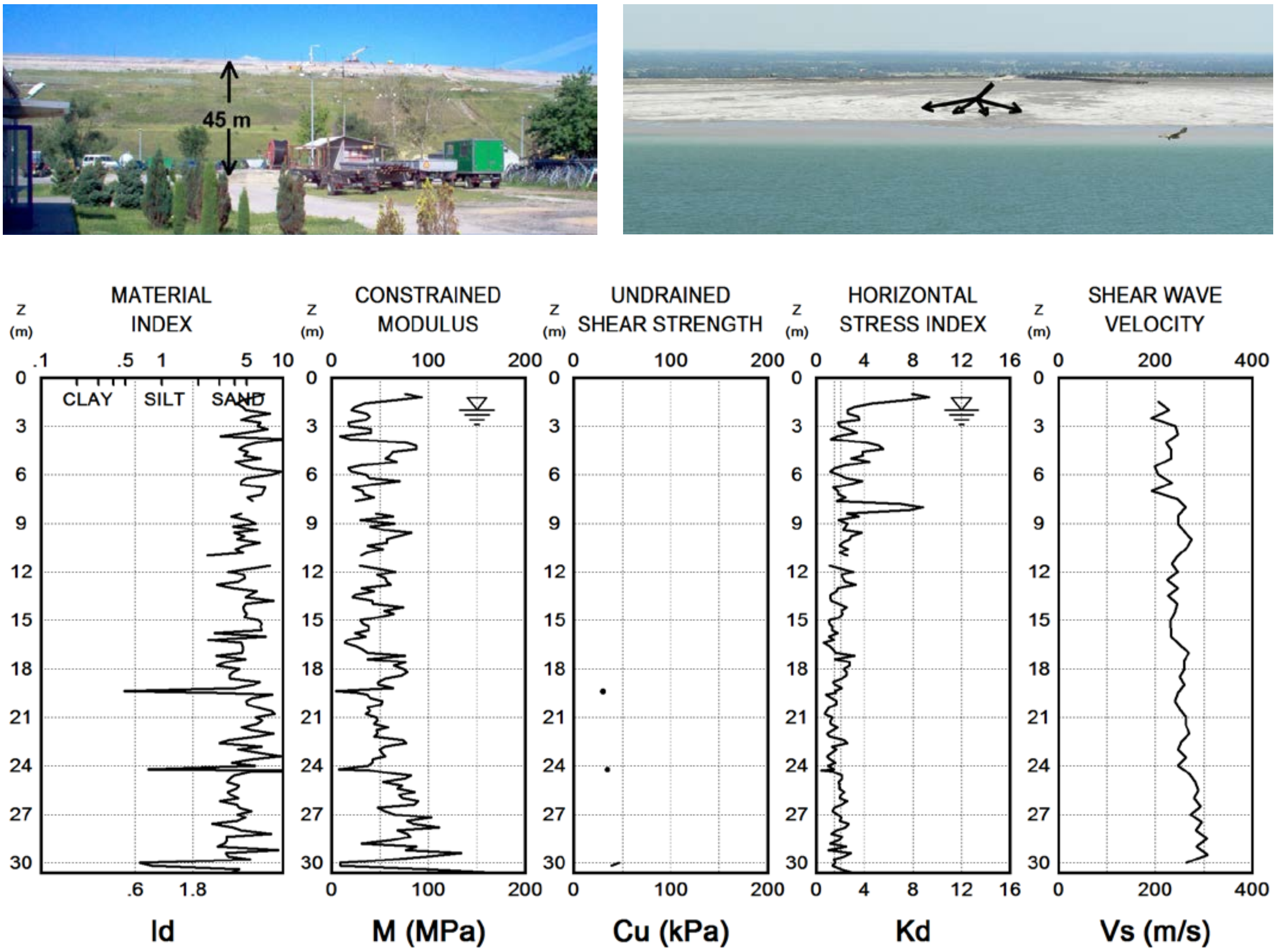


Fig. 28. Details of SDMT investigation (photographs) and SDMT profiles at the site of the Zelazny Most tailing dam, Poland

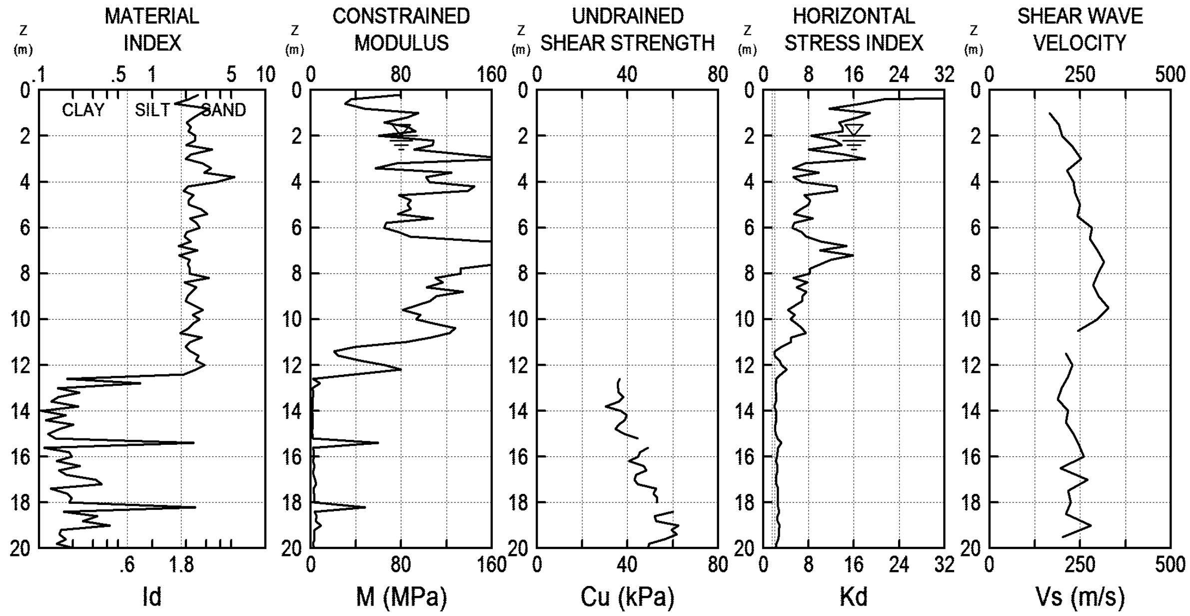


Fig. 29. SDMT profiles at the site of Barcelona – El Prat Airport (Spain)

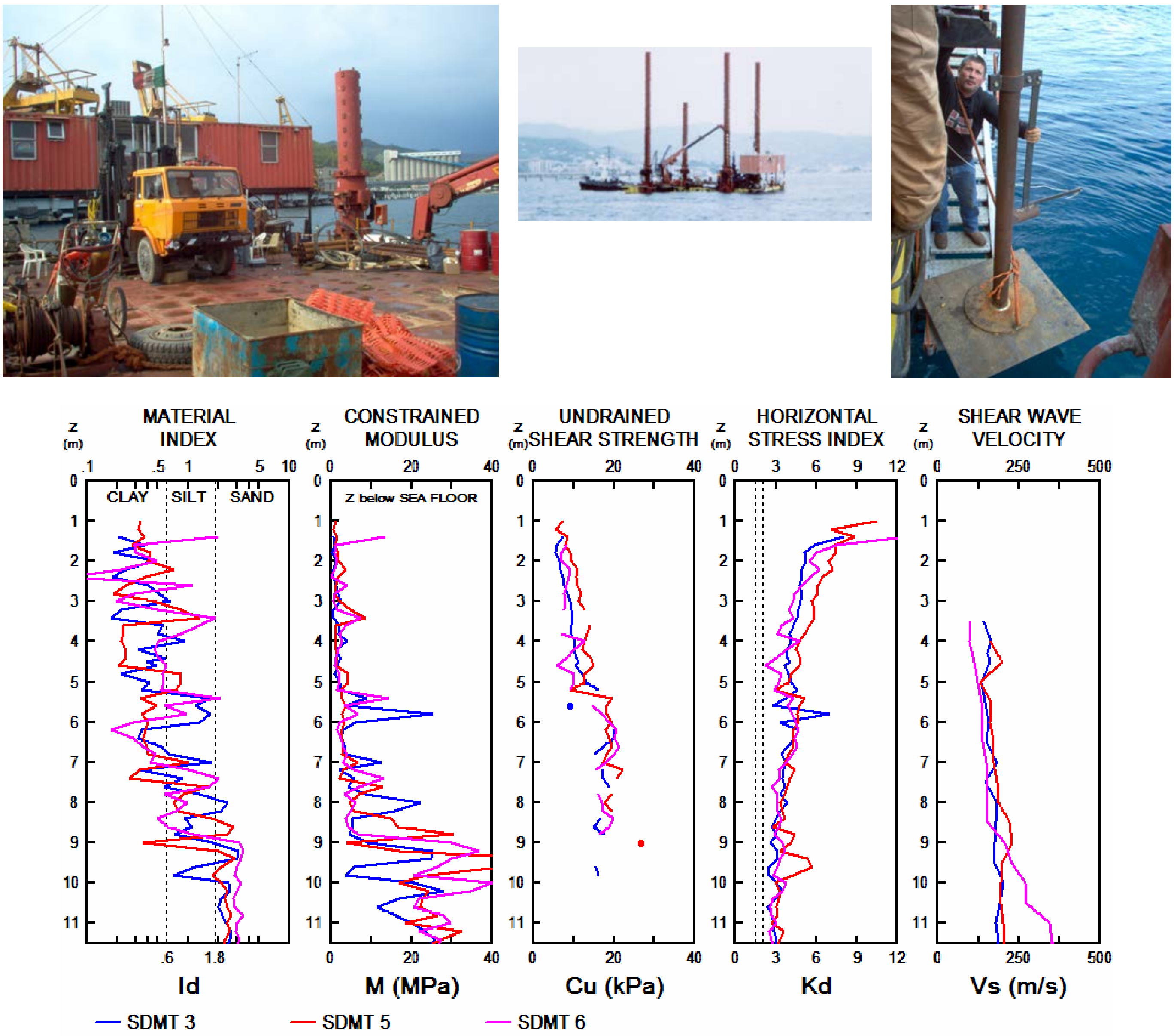


Fig. 30. Details of offshore SDMT investigation (photographs) and SDMT profiles at the site of Vado Ligure (Savona), Italy

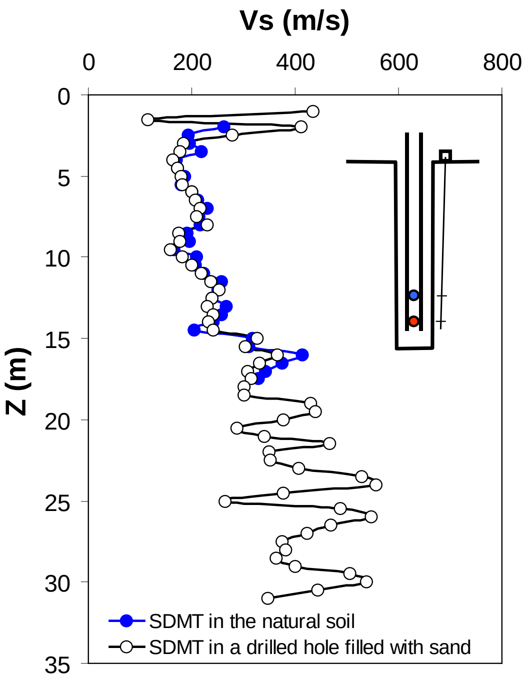


Fig. 31. Comparison of V_S profiles obtained by SDMT in the natural soil and in a backfilled borehole at the site of Montescaglioso – Ginosa (Matera), Italy

10. CONCLUSIONS

The seismic dilatometer (SDMT) appears to provide accurate and highly reproducible measurements of the shear wave velocity V_S – a basic input parameter for seismic analyses.

SDMT provides, besides V_S, usual DMT results (e.g. constrained modulus M_{DMT}, undrained shear strength c_u, stress history OCR) for current design applications, such as settlement prediction and many others (see TC16 2001).

Research currently in progress investigates the possible use of the SDMT for deriving "in situ" decay curves of soil stiffness with strain level (G-γ curves or similar). Such curves could be tentatively constructed by fitting "reference typical-shape" laboratory curves through two points – the initial shear modulus G_0 (from V_S) and a "working strain" modulus corresponding to M_{DMT} – both provided by SDMT.

One of the major issues of present SDMT research is the use of the SDMT for evaluating the liquefaction resistance of sands – an attractive alternative / integration to current methods based on CPT-SPT, since "redundancy" in evaluating liquefiability by more than one method is generally recommended.

SDMT permits to obtain two parallel independent evaluations of liquefaction resistance CRR from the shear wave velocity V_S and from the horizontal stress index K_D, by means of the CRR-V_S and CRR-K_D correlations shown in Fig. 21 and in Fig. 22, to be used in the framework of the Seed & Idriss (1971) simplified procedure. The use of V_S for evaluating CRR is well known. Correlations CRR-K_D have been developed in the last two decades, stimulated by the recognized sensitivity of K_D to a number of factors which are known to increase liquefaction resistance – stress history, prestraining/aging, cementation, structure – and the relationship of K_D to relative density and state parameter.

Latest studies have pointed out the ability of K_D to reflect aging in sands, a factor that recent research has indicated as having a first order of magnitude influence on liquefaction behaviour. In addition, recent SDMT experience has indicated the high sensitivity of K_D to "non-textbook" OCR crusts in NC sands. These findings lend additional support to a well-based CRR-K_D correlation.

Comparisons based on parallel measurements of K_D and V_S obtained by SDMT at several sandy sites have shown that V_S and K_D would provide, in general, substantially different estimates of CRR, leaving open the question "which CRR should be given greater weight". In principle, the authors would propend to give greater weight to CRR by K_D for the following reasons:

– Shallow OC crusts (very unlikely to liquefy), found at the top of most sand deposits, are unequivocally depicted by high K_D values but almost "unfelt" by V_S. This suggests a lesser ability of V_S to profile liquefiability.
– V_S is measured at small strains, whereas pore-pressure build up and liquefaction are medium- to high-strain phenomena. In cemented/bonded soils V_S can be "misleadingly" high due to interparticle bonding, largely destroyed at higher strains (range of K_D measurement). Thus the K_D indications could possibly be more relevant for liquefiability. Very light earthquakes, however, may not destroy bonding, then CRR evaluated by V_S may be appropriate in this case.
– Many indications suggest at least some link between K_D and state parameter, which is probably one of the closest proxy of liquefiability.
– K_D is more sensitive than V_S to relative density and to other factors that greatly increase liquefaction resistance, such as stress history, aging, cementation, structure (which, incidentally, are felt considerably more than by penetration resistance).

The above obviously deserves considerable additional verification, supported by well documented real-life liquefaction case histories.

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