04 Apr 1995, 9:00 am - 10:00 am

Role of In-Situ Testing in Geotechnical Earthquake Engineering

M. Jamiolkowski

D.C. F. LoPresti

O. Pallara

Proceedings: Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
April 2-7, 1995; Volume III, St. Louis, Missouri

STATE OF THE ART (SOA7)
Role of In-Situ Testing in Geotechnical Earthquake Engineering

M. Jamiolkowski, D.C.F. LoPresti and O. Pallara

Abstract: The available in situ testing techniques of special relevance in Geotechnical Earthquake Engineering are subject to a synthetic review in the light of the general framework of soil stress-strain behaviour. Especial attention is devoted to the recent innovations and current capabilities of in situ testing methods to assess the shear modulus G and damping ratio D. The determination of the undrained steady state shear strength via penetration and seismic tests is also discussed.

1. INTRODUCTION

This paper attempts to summarise the role and validity of in situ testing techniques in the characterisation of natural soil deposits with special emphasis on the problems related to the Geotechnical Earthquake Engineering (GEE).

This task involves a broad spectrum of situations in which a soil deposit is subject to rapid monotonic or cyclic stress variations that can occur in undrained, partially drained or drained conditions depending on the hydraulic conductivity of the soil, the distance from the drainage boundaries and loading history.

In order to analyse the response of the soil deposit and that of the interacting geotechnical engineered construction in a rational manner the following information is necessary:

- Geometry of soil strata and their spatial variability
- Ground water conditions
- Geostatic stresses and related stress history
- Characteristics of hydraulic conductivity
- Deformation and damping characteristics assessed in the strain range of interest
- Undrained monotonic and cyclic shear strength of both cohesionless and cohesive strata.

This information can be obtained by means of in situ and laboratory tests assisted by geological studies. The relative merits of in situ versus laboratory tests have recently been examined by Jamiolkowski and Lo Presti (1994) who have concluded that these experimental techniques are complementary rather than competing methodologies. Moreover, it is important to recognise that the progress that has been made since the early eighties in the area of laboratory testing and undisturbed sampling has greatly influenced the framework of the site characterisation thus allowing:

- A deeper insight into the prefailure mechanical behaviour of soils which in turn has had an important impact on the understanding and interpretation of in situ tests.
- A better focusing on the most relevant applications of in situ tests to fulfil the needs of practising engineers.

As far this latter point is concerned, the following applications of the in situ techniques appear of prioritary interest in the field of the GEE:

- Soil profiling and characterisation including spatial variability
- Evaluation of the initial horizontal stress
- Assessment of hydraulic conductivity
- Assessment of the small strain elastic shear (G_0) and constrained (M_0) moduli
- Assessment of the undrained shear strength especially of cohesionless deposits
- Correlations between in situ test results and the response of boundary value problems of practical interest, e.g. susceptibility of sand deposits to cyclic liquefaction.

In addition to the above applications, there have been attempts to obtain the following information by means of in situ tests: the material damping at small strains (D_0), the state parameter (ψ) of cohesionless deposits (Been et al. 1986, Yu et al. 1994), the stiffness anisotropy at small strains (Stokoe et al. 1985, Jamiolkowski and Lo Presti 1994, Stokoe et al. 1994a, Bellotti et al. 1995).

With this in mind, a brief examination is made of the different in situ techniques which are of special interest in GEE, with special reference to the frame of the mechanical behaviour of soils which emerges from the most recent results of advanced laboratory tests (Jardine 1985, 1994, Atkinson and Sallfors 1991, Burghignoli et al. 1991, Jardine et al. 1991, Tatsuoka and Shibuya 1992, Jamiolkowski et al. 1991,1994, Tatsuoka and Kohata 1994, Tatsuoka et al. 1995).

An appropriate understanding of the basic mechanical behaviour of soils is, in fact, judged to be essential in any rational interpretation of in situ test results no matter what degree of empiricism is used in the analysis.

It is therefore worthwhile to recall the requirements that, according to Wroth (1988), any attempted correlation between soil properties and in situ test results should ideally have:

- Based on a physical appreciation of why the properties can be expected to be related.
- Set against a theoretical background however idealised this may be.
- Expressed in terms of dimensionless variables so that advantage can be taken of the scaling laws of continuum mechanics.

2. QUALITATIVE PICTURE OF SOIL BEHAVIOUR

The last decade was characterised by major advances for a better understanding of the mechanical behaviour of soils. This progress is mostly related to the improvement of the quality and reliability of laboratory testing which has benefited from the following:

- Extensive use of microcomputer based systems permitting to perform feed-back controlled experiments.

- Disclosure of new techniques to locally measure on the sample the strains of up to $5 \cdot 10^{-6}$ with a high degree of confidence.

- A better accuracy in the measurements of the vertical load imposed on the sample, thanks to the load cells located inside the cell.

- Development of new generation of apparatuses that allow one to investigate the mechanical behaviour of soils over a wide range of imposed stress states.

For a deeper insight into these problems see the keynote lecture presented by Tatsuoka et al. (1995) at this conference.

Mechanical soil behaviour is hereafter discussed, making reference to the scheme proposed by Jardine (1985, 1992a, 1994) which is shown in Fig. 1.

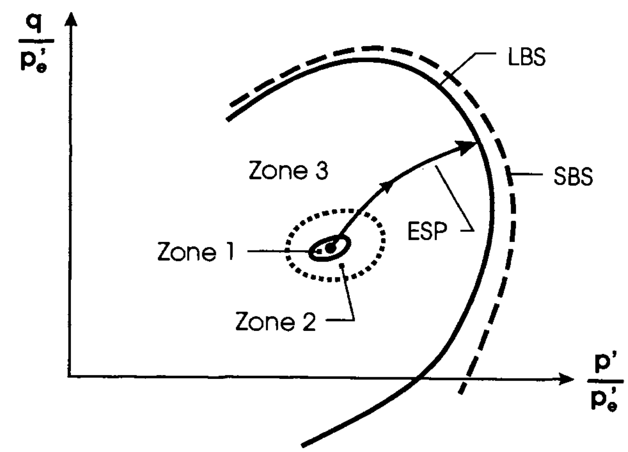

Figure 1 Stress-strain behaviour of soils: a simplified picture (Jardine, 1985, 1992a, 1994)

This scheme, basically following the ideas of Mroz (1967), adds two kinematic yield surfaces inside the State Boundary Surface (SBS). The SBS is the large scale yield envelope, typical of the family of Cam Clay Models. Multiple yield surface models give a more realistic picture of soil behaviour. A similar scheme was worked out by Stallebrass (1990).

The essential features of this model are shown in the normalised stress plane in Fig. 1 where referring to an axisymmetric stress state, such as that encountered in the Triaxial (TX) and Torsional Shear (TS) tests, one obtains:

$p' = (\sigma'_a + 2 \cdot \sigma'_r)/3$

$q = \sigma'_a - \sigma'_r$

p'_e = equivalent pressure on the isotropic virgin compression line (VCL) (Hvorslev 1937)

σ'_a, σ'_r = axial and radial effective stresses respectively

The normalised stress plane p'/p'_e vs. q/p'_e can be divided in three distinct zone in order to match the behaviour of large variety of soils and soft rocks, mainly observed in monotonic and cyclic TX and TS tests.

Linear Elastic Zone (Zone 1)

Within this zone, for all practical purposes, the soil exhibits a linear elastic response. Consequently, the shear modulus G_o and the Young's modulus E_o result as a first approximation to be invariant of the applied shear strain level, number of loading cycles N and the stress/strain history. They infact only depend on the current soil state asserted via the void ratio e, effective stress state p' and q and the material fabric originated by the depositional and post depositional processes. The E_o and G_o can therefore be regarded as the initial stiffness of the relevant stress-strain curves of a given soil. Both these moduli, if properly normalised with respect to the void ratio and effective stresses, result to be equal or similar in magnitude independently of the type of loading (monotonic or cyclic). Within Zone 1 the soils exhibit a rheological response related to the viscous characteristics of their skeleton and, at very high strain rates, also due to the viscosity of the pore fluid (Shibuya et al. 1994, Tatsuoka et al. 1995). From a practical point of view this affects the elastic stiffness in a twofold manner:

- Under constant consolidation stresses, both G_o and E_o increase with time when subject to drained creep. This increase is much more pronounced in fine grained soils. In particular for these soils the rate of stiffness raise with time increases with an increasing plasticity index (PI). The increase of G_o with time after the end of primary consolidation (EOP) is often quantified by means of the following empirical formula (Anderson and Stokoe 1978):

$$G_o(t) = G_o(t = t_p) \cdot \left[1 + N_G \cdot \log \frac{t}{t_p}\right] \quad (1)$$

where

$G_o(t)$ = small strain shear modulus at $t > t_p$

$G_o(t = t_p)$ = small strain shear modulus at $t = t_p$

t = any generic time larger than t_p

t_p = time to complete the primary consolidation

$N_G = \frac{G_o(t) - G_o(t = t_p)}{G_o(t = t_p)} \cdot \frac{1}{\log(t/t_p)}$ normalised rate of increment of the small strain shear modulus per log cycle of time.

TABLE 1 Increase of G_o with time in the laboratory

Soil	d_{50} [mm]	PI [%]	N_G [%]	Notes
Ticino sand	0.54	-	1.2	Predominantly silica
Hokksund sand	0.45	-	1.1	Predominantly silica
Messina sand and gravel	2.10	-	2.2 to 3.5	Predominantly silica
Messina sandy gravel	4.00	-	2.2 to 3.5	Predominantly silica
Glauconite sand	0.22	-	3.9	50 % Quartz 50 % Glauconite
Quiou sand	0.71	-	5.3	Carbonatic
Kenya sand	0.13	-	12	Carbonatic
Pisa clay		23-46	13 to 19	
Avezzano silty clay		10-30	7 to 11	
Taranto clay		35-40	16	

The N_G values range between 0.05 and 0.20 in clays, increasing with the PI (Kokusho 1987, Mesri 1987). In coarse grained soils the values of N_G are smaller, falling into the range of 0.01 to 0.03 except with special soils such as glauconitic and carbonatic sands where it can attain much higher values, comparable to those of clays (see data reported in Table 1). The limited experimental data so far available suggests that what is above stated about the influence of drained creep on G_o also holds in a first approximation for E_o.

- Much less is known about the shear strain rate $\dot{\varepsilon}_s$ and/or the loading frequency (f) affect G_o and E_o. It is generally recognised that over the wide range of $1 < \dot{\varepsilon}_s < 1000\% / \text{day}$ the strain rate does not influence the small strain stiffness, especially when soil is subject to monotonic loading (Tatsuoka and Shibuya 1992, Tatsuoka and Kohata 1994, Tatsuoka et al. 1995). The data recently presented by Stokoe et al. (1994a) for the range of f between 0.1 and 100 Hz indicated that while the increase of the loading frequency in sands leads to a very small increase of G_o (less than 5%), this factor can have much larger impact on the small strain shear stiffness in clays. The influence of f on G_o in clays increases with an increasing PI. In the case of Pisa clay, which is a soft lightly overconsolidated clay with $47\% < PI < 55\%$ %, undrained resonant column (RC) and drained monotonic loading torsional shear tests were performed (Lo Presti 1994). The strain rate for the monotonic test was about 0.014%/min while for the RC test it increased with the shear strain from about 15 to 1900%/min. At small strains a 1000 times larger strain rate was therefore experienced during the RC test resulting in an increase of G_o of less than 15 %.

Both G_o and E_o in Zone 1 result to be dependent on the direction of the applied effective stress path, in the case of the volume element tests, and on the direction of the propagation and polarisation in the case of the seismic body waves (Lee and Stokoe 1986, Stokoe et al. 1994a, Jamiolkowski et al. 1994, Jardine 1994, Tatsuoka and Kohata 1994, Bellotti et al. 1995). This directional dependency of stiffness reflects the elastic transverse-isotropy of many natural soils this also being called elastic cross-anisotropy. The small strain anisotropy reflects two basic types of phoenomena:

- Inherent or fabric anisotropy which a given soil exhibits when subject to an isotropic state of stress.
- Stress induced anisotropy which reflects the directional variation of stiffness as a function of the applied anisotropic stress system.

TABLE 2 Initial anisotropy of some soils within zone 2.

Sands*	e	σ'_h/σ'_v	G_{hh}/G_{vh}	M_h/M_v	E_h/E_v	
Predominantly silica Ticino river sand	0.778	0.5	0.96	0.83	0.81	Medium dense
		1.0	1.20	1.20	1.22	
		1.5	1.25	1.55	1.52	
		2.0	1.44	1.88	1.86	
$D_{50} = 0.55$ mm *Bellotti et al (1995)*	0.617	0.5	1.13	1.05	NA	Very dense
		1.0	1.15	1.31	NA	
		1.5	1.25	1.40	NA	
Carbonatic Kenya sand	1.573	0.5	1.09	0.76	0.76	Medium dense
		1.0	1.13	1.28	1.23	
		2.0	1.29	1.98	1.85	
$D_{50} = 0.13$ mm	1.315	0.5	1.05	0.94	0.97	Very dense
		1.0	1.24	1.25	1.29	
		2.0	1.40	2.00	1.92	

(*) Seismic tests performed in Calibration Chamber; NA = not available.

	σ'_h/σ'_v	OCR	G_{hh}/G_{vh}		σ'_h/σ'_v	OCR	G_{hh}/G_{vh}
	0.55	1	1.43		0.55	1	1.27
Panigaglia	0.55	1	1.36	***Upper***	0.55	1	1.34
clay**	0.71	1.7	1.43	***Pisa***	1.49	1.5	1.29
	0.87	2.5	1.59	***clay***	2.62	2.6	1.40
PI = 44 %	1.24	5.1	1.55		4.01	4.0	1.48
$e_o = 1.6$	1.97	12.8	1.79	PI = 41 %	7.96	8.0	1.56
	2.85	26.9	2.08	$e_o = 1.843$	16.39	16.4	1.77

(**) Seismic tests performed in laboratory on high quality undisturbed samples, Jamiolkowski et al (1994).

The relatively limited experimental data collected so far for sands and clays allow one to make the following preliminary comments:

- The fabric anisotropy is alone responsible for a relatively modest directional variation of G_o and E_o. Usually, in soils, the ratio of G_{hh}/G_{vh} and E_h/E_v in Zone 1 falls within the range of 1.2 to 1.5 where:

  $G_{hh} = G_o$ on the horizontal plane
  $G_{vh} = G_o$ on the vertical plane
  $E_h = E_o$ in the horizontal direction
  $E_v = E_o$ in the vertical direction

- The stress induced anisotropy in soils plays an important role in the directional variation of the small strain stiffness. In the presence of the anisotropic stresses the G_{hh}/G_{vh} and E_h/E_v ratios, as well as the ν_{vh} Poisson's ratio, depend on the applied effective stress ratio $K = \sigma'_a/\sigma'_r$ (Bellotti et al 1995, Tatsuoka and Kohata 1994, Pallara 1995).

Table 2 shows some data concerning the initial anisotropy (inherent+stress induced anisotropy) of two reconstituted sands and one natural clay.

The small strain viscous damping (D_o) in Zone 1, when determined via reliable laboratory experiments (Tatsuoka et al. 1995, Stokoe et al. 1994a), generally results to be less than 1 %. The D_o, depends on the same factors already mentioned in connection with the small strain stiffness, although their quantitative impact on D_o is not necessarily the same as that observed in the case of G_o and E_o. In particular, it is worth mentioning, the sensitivity of D_o to the loading frequency f and hence to the strain rate as recently pointed out by (Shibuya et al. 1994, Tatsuoka et al. 1995, Stokoe et al. 1994a) to a larger extent than in the case of stiffness.

The border between Zones 1 and 2, defined in terms of strain, is usually called elastic threshold strain. Depending on the type of laboratory test in question this term can be applied either to the shear strain ($\varepsilon^e_{st}, \gamma^e_t$) or to the axial strain (ε^e_{at}). Hereafter the term elastic threshold strain will be used making reference to the symbol ε^e_t and without specifying from what kind of test it has been inferred.

Generally the values of ε^e_t are inferred from TX, TS and RC tests. From the large set of experimental data available at present the following can be concluded as far as the magnitude of ε^e_t is concerned:

- In uncemented and unaged sands and clays the values of ε^e_t usually fall within the narrow range of $7 \cdot 10^{-6}$ to $2 \cdot 10^{-5}$.
- Aging, cementation and other diagenetic processes tend to increase the value of ε^e_t to up one order of magnitude.
- The influence of mechanical overconsolidation on the ε^e_t is still not clear. Some experimental data suggests that, in a given soil especially in clay, the ε^e_t can increase with increasing overconsolidation ratio OCR.
- Apparently ε^e_t seems to increase in clays with increasing PI. However, owing to the fact that ε^e_t also increases with increasing strain rate (Isenhower and Stokoe 1981, Tatsuoka and Shibuya 1992) and considering that the sensitivity of clays to the strain rate becomes more pronounced in high plasticity clays, this aspect may require further experimental validation. However, considering that in a RC test the strain rate can increase from about 10 to 2000 %/min with the strain level, this kind of test should not be used to assess ε^e_t.
- What has been stated above has mostly been established with reference to G_o. As the elastic threshold strain concept applied to the small strain material damping is concerned, the available information is relatively scarce. The data presented by Stokoe et al. (1994a) and

Tatsuoka et al. (1995) seems to suggest that $\varepsilon_t^e(D_o)$ is similar to $\varepsilon_t^e(G_o)$.

Non-Linear Elastic Zone (Zone 2)

When soil is strained beyond ε_t^e the ESP penetrates into Zone 2 (Fig. 1). In this zone the stress-strain response starts to be non linear. Consequently the deformation moduli G and E depend not only on the current state of the soil but also on the imposed shear stress or strain level. Moreover G and E are influenced by many other factors such as strain rate, aging, OCR, recent stress history or direction of perturbing ESP, etc. The terms "recent stress history" and "perturbing stress path" were introduced and used by Richardson (1988), Stallebrass (1990), Smith (1992) and Jardine (1994). These terms simply indicate the type and sequence of the ESP's which can be reproduced during laboratory tests.

The stress-strain-time behaviour of soils in this zone is still not fully understood and is to some extent controversial and therefore at present not easy to summarise.

According to Jardine (1985, 1992a, 1994) a small load-unload loop, during which the effective stress state remains confined inside Zone 2, produces an insignificant plastic shear strain, assuming that enough time is allowed for the recovery of the viscous deformation. This kind of behaviour implicitly implies a non linear but elastic behaviour.

The decay of the deformation moduli E and G with an increasing level of the relevant strain generally does not exceed 20-30 % of their initial value. Moreover, in cyclic tests, the stiffness is only moderately affected by the number of loading cycles.

Also in Zone 2 the deformation moduli appears relatively little sensitive to the strain rate effect.

Very little is known about the stiffness anisotropy in Zone 2. As a first approximation, considering the relatively small impact of the plastic phenomena on the observed stress-strain response, which implies only a secondary modification of the soil fabric, it can be postulated that the anisotropy of G and E preserves the features of that of Zone 1.

What has been stated above for the deformation moduli, applies in first approximation to D which appears however, much more affected by strain rate than E and G.

The limit between Zones 2 and 3 can again be defined in terms of the strain level, called volumetric threshold strain ε_t^v. The concept, which stays behind the ε_t^v, refers to the onset of important permanent volumetric strain (ε_v^p) and either positive or negative residual excess pore pressure (Δu) in drained and undrained tests respectively. Certainly, when exceeding ε_t^v during cyclic tests, N starts to influence the degradation of stiffness in a definitive manner and the material never reaches the stable configuration in the sense that with an increasing number of loading cycles an accumulation of Δu or ε_v^p continue in undrained and drained tests respectively, see Dobry et al. (1982), Chung et al. (1984), Lo Presti (1987), Vucetic (1994), Stokoe et al (1994a).

The values of ε_t^v are generally one order of magnitude higher than those of the ε_t^e. The available experimental data suggests that usually range between 0.007 and 0.01 %

The lower limit applies to the uncemented, unaged, normally consolidated sands and gravels while the upper limit refers to high plasticity clays and cemented soils (Vucetic 1994, Stokoe et al. 1994a). The RC tests, involving very high and variable strain rates, should be avoided when assessing the ε_t^v values, as for the determination of the elastic threshold strain.

The assessment of the ε_t^v can be attempted with reference to both the decay of the G/G_o and the raise of the D/D_o ratios. Experimental data shows that some times $\varepsilon_t^v(G) > \varepsilon_t^v(D)$ (Stokoe et al. 1994a).

Elasto-Plastic Zone (Zone 3)

When the deformation process engages Y_2, the soil starts to yield and the plastic deformation becomes important. As the ESP proceeds towards the Limit Boundary Surface (LBS) which coincides with Y_3 (Fig. 1), the ratio of the plastic shear strain to the total shear strain increases approaching values close to one at Y_3 (Fig. 2). This figure implicitly assumes that the soil response in Zone 2 is non-linear elastic.

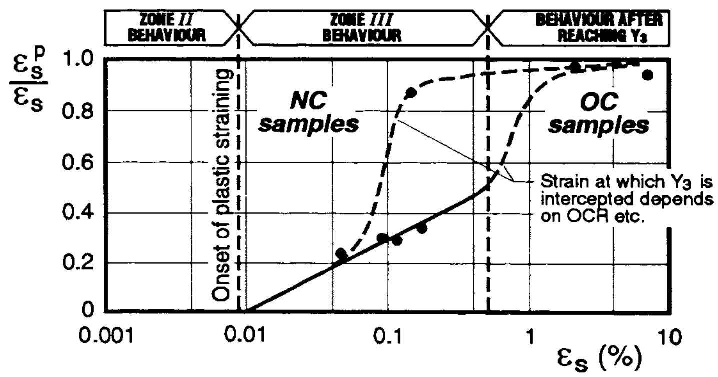

Figure 2 Relationship between permanent and total shear strain in Magnus till (Jardine, 1992a)

In case of clays, the Y_3 is matched by the undrained ESP of a truly normally consolidated (NC) material in extension and compression respectively (Gens 1982, 1985). The SBS, located outside of the LBS, retains only the role of a boundary separating the possible states from the impossible ones. As pointed out by Gens (1982,1985), the separation of Y_3 from SBS is typical for materials not obeying the Rendulic's principle which is a common condition verified for many natural soils.

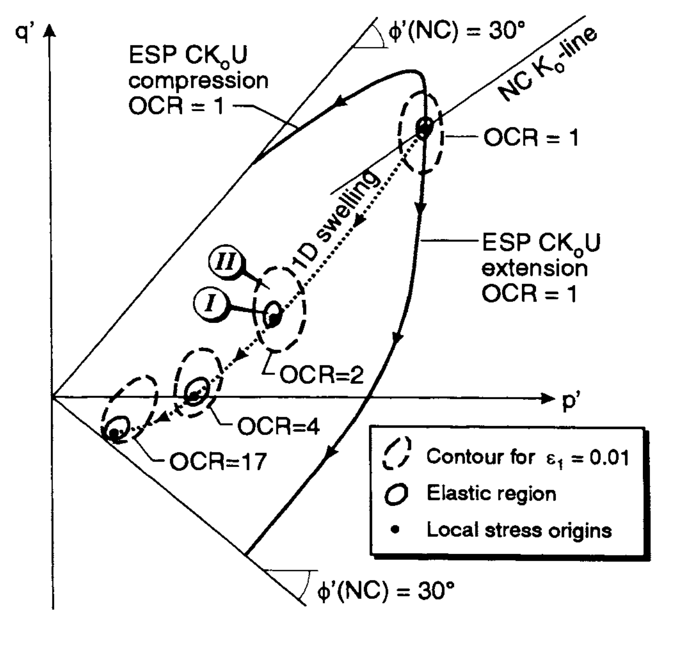

Figure 3 Kinematic nature of small strain regions. Qualitative trend (Adapted from Jardine, 1991 and Jardine et al, 1991).

The SBS on the "wet side" of the Critical State (CS) can be reached by drained ESP performed on NC clay under constant consolidation stress ratio $K_c = \sigma'_{rc} / \sigma'_{ac}$ in which large volumetric strains are attained (Gens 1982, Smith 1992, Jardine 1994). On the "dry side" of the CS the SBS is represented by the Hvorslev (1937) surface.

In the Zone 3 the stress-strain response of soils becomes highly non-linear. The G, E and D are strongly depending on shear stress and strain level. The factors like strain rate, creep, OCR influence in relevant manner the magnitudes of these parameters.

Factors like the recent stress history and the direction of perturbing ESP continue to influence the stress-strain response within Zone 3. However, their relevance decreases with increasing distance from Y_2 (Richardson 1988, Stallebrass 1990, Jardine 1994).

The above qualitative scheme of soil behaviour is characterised by the kinematic nature of the surfaces Y_1 and Y_2 which are always dragged with current stress point. On contrary the LBS and SBS are relatively immobile so that any sharp change of the ESP from the Y_3 inwards leaves their position unchanged except in soils with highly developed fabric in which the collapse of the structure can determine their contraction.

The above mentioned feature of the Y_1, Y_2 and Y_3 yield surfaces, during one dimensional unloading, are illustrated in Fig. 3. An example of the yield surfaces, as obtained on high quality undisturbed samples of the Bothkennar clay, is shown in Fig. 4.

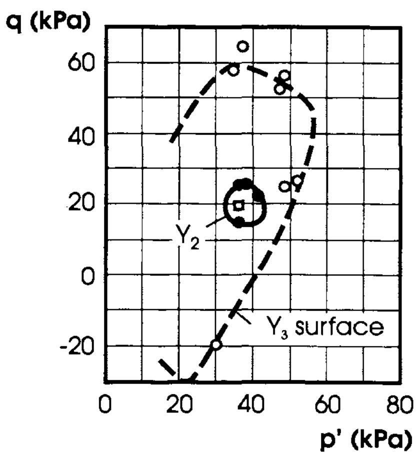

Figure 4 Yield surface of the Bothkennar clay, Sherbrooke samples (Smith, 1991).

Even if the previously discussed scheme has been developed for clays, as a first approximation, it holds also for sands and gravels especially as the role of the Y_1 and Y_2 is concerned.

3. USE OF IN SITU TESTING IN PRACTICE

Various approaches are used in the interpretation of in situ tests. Basically, they can be grouped as follows:

A. Direct approaches: observed behaviour of large scale models or full scale prototypes is directly correlated to in situ test results; e.g. use of shear wave velocity or penetration test results in the evaluation of susceptibility to liquefaction of the cohesionless deposits or evaluation of the ultimate bearing capacity of a single pile.

B. Indirect approaches: relevant soil properties, obtained from volume element tests, are correlated with in situ test results. The obtained parameters are then used in the design.

Within group B it is worthwhile to distinguish the following situations:

B1: During the test all strained soil elements essentially follow the same effective stress path. These kinds of tests can be interpreted for basic material parameters depicting the response of a representative volume element within a framework of an appropriate constitutive model. Typical examples are; seismic tests which in virtue of the very small strain level involved are interpreted within the framework of the theory of elasticity and the Self Boring Pressuremeter (SBP) tests (Clark 1994) from which, at least in principle, a complete stress-strain curve can be inferred within the framework of a properly formulated elasto-plastic soil model.

B2: During the test, the strained soil elements are subjected to very different ESP's. In this case it is not possible to obtain, from the tests, the soil properties that reflect the behaviour of a representative volume element. Typical examples are; Plate Load Test (PLT) and model tests (e.g. centrifuge, large shaking table, calibration chamber, etc.) which when interpreted within the framework of simplified theories allow one to determine the "operational" value of a soil parameter that reflects the average soil response linked to specific load magnitude and test geometry.

B3: The tests belonging to categories B1 and B2 have in common the fact that their implementation does not cause large straining of the surrounding ground. On the contrary all penetration tests [Cone Penetration Test (CPT), Piezocone Test (CPTU), Standard Penetration Test (SPT), Dilatometer Test (DMT), Pressiocone Test (PCPT)] are closely linked to large or even very large strains involved with their insertion into the ground. In this case the results of the penetration tests are correlated to the selected soil properties. Typical examples of this approach are; correlations between deformation moduli and CPT, SPT and DMT results, e.g.:, or charts for evaluation of the cyclic strength making reference to the cone resistance q_c of CPT or blow-count N_{SPT} of SPT (Seed et al. 1983, Tokimatsu and Yoshimi 1983, Seed and De Alba 1986, Seed et al. 1984). Because of the empirical nature of these correlations, they are subjected to many limitations that are not always fully recognised by the potential users. For example, it should be recognised that these correlations are implicitly formulated for either fully undrained or fully drained conditions, while in the field the penetration tests frequently occur in partially drained conditions.

The information gained from the monitored behaviour of prototypes can be used following one between the two approaches outlined here below:

- The gathered data is used to verify a specific constitutive model coupled with the computational procedure used. In this case all input parameters representing the stress-strain-time and strength characteristics of the soil must be determined via appropriate laboratory and in situ tests.

- A set of collected data, e.g. excess pore pressure, horizontal and vertical displacements, ground acceleration, etc. is interpreted with the aim of assessing the relevant design parameters within the framework of the adopted constitutive model.

Of these two approaches, only the second is considered to be pertinent to the scope of the present work.

Many experimental testing sites have been established over the last fifteen years in active earthquake zones to monitor free field and prototype (or large scale model) response to earthquake motion.

4. INNOVATIONS AND CAPABILITIES OF IN SITU TESTING

Detailed reviews of in situ testing techniques can be found in Mitchell (1978) Woods (1978), Wroth (1984), Woods and Stokoe

(1985), Jamiolkowski et al. (1985 and 1988), Woods (1991 and 1994). The present section focuses primary on the recent applications of the geophysical methods in geotechnical engineering hence they are especially pertinent to the theme of this conference.

4.1 Small strain tests (Seismic tests)

Seismic tests are conventionally classified into borehole and surface methods. Both of them belong to the category B1 of in situ techniques and have undergone significant advances as testing and interpretation methods are concerned. Usually, these methods enable one to determine the velocity of body waves [compressional (P) and/or shear (S)] and surface waves [Rayleigh] which induce very small strain levels into the soil, i.e. ε_{ij} < 0.001 % (Woods 1978). It is therefore possible, on the basis of the measured wave velocities, to obtain the small strain elastic deformation characteristics according to the well known relationships:

$$G_o = \rho V_s^2 \qquad (2)$$

$$M_o = \rho V_p^2 \qquad (3)$$

$$V_R / V_s \cong (0.862 + 1.14\nu)/(1+\nu) \qquad (4)$$

$$\nu = (V_p^2 - 2V_s^2)/2(V_p^2 - V_s^2) \qquad (5)$$

where:

G_o, M_o = small strain shear and constrained moduli respectively

ρ = mass density

V_s, V_p, V_R = velocities of shear, compressional and Rayleigh waves respectively

ν = Poisson's ratio

The above relationships are based on the hypotheses of elasticity and isotropy.

The P wave velocity measured below the water table is more or less coincident with that of sound propagation in water ($V_{water} \cong$ 1500-1600 m/s, see as an example Mitchell et al. 1994) and does not represent a propagation velocity in the soil skeleton which is usually less than V_{water}. Under these conditions the assessment of both constrained modulus and Poisson's Ratio by means of the above listed equations is no longer possible (Woods and Stokoe 1985).

Recently researchers have paid much more attention to the possibility of inferring the small strain damping ratio, D_o from seismic tests and various examples can be found in literature, as mentioned later on.

4.1.1 Borehole Methods

Current practice and recent innovations of borehole methods for seismic exploration are covered by many comprehensive works (Auld 1977, Stokoe and Hoar 1978, Woods 1978, Stokoe 1980, Woods and Stokoe 1985, Woods 1991 and 1994). In the following, the most recent advances concerning the basic aspects of the borehole seismic testing is briefly summarised.

Boreholes

A good coupling between boreholes and the surrounding soil is the key point to obtain useful measurements, especially during crosshole tests. PVC casing grouted into place with a cement-bentonite grout is recommended in order to avoid poor coupling. Grout should not shrink and should possibly reproduce the in situ unit weight. An excellent coupling is obtained by using the Seismic Cone (SCPT) both with a downhole configuration (Campanella and Robertson 1984) and a crosshole configuration (Baldi et al 1988). A very good coupling is also offered by the suspension P-S velocity logging method (Nigbor and Imai 1994). The scheme of this test is shown in figure 5. Pressure wave is generated by the source, a horizontal solenoid located at the end of the suspended probe, inside an uncased borehole which is filled with drilling mud or water. A good coupling is assured by the presence of the fluid. Pressure wave generates P or S body waves when approaching the borehole walls. Body waves travel upward in the soil and, after conversion back to pressure waves in the borehole fluid, they are detected by a pair of geophones placed one meter from each other and located in the upper part of the suspended probe, five meters above the source. Compressional and shear wave velocities can be determined by means of the true interval method. The suspension P-S velocity logging method, mainly used in Japan, has been developed for deep seismic explorations (up to 2 km); the shear wave velocity obtained with this method compare well with those determined with other borehole techniques (Nigbor and Imai 1994).

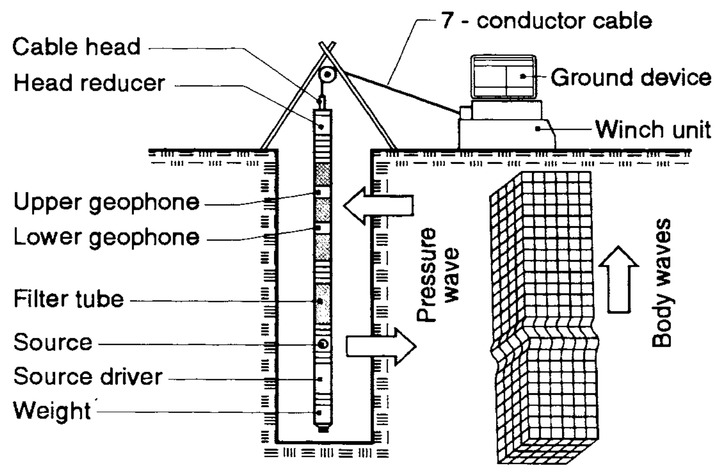

Figure 5 Suspension PS logging method schematic layout (Nigbor and Imai 1994).

A check of the borehole verticality by means of inclinometer measurements is also highly recommended in order to accurately determine the travel path of the waves. These measurements are automatically performed during SCPT tests.

Sources

The repeatability of generated waveforms, the possibility of polarity inversion and generation of either compression or shear specialised body waves are desired features for any energy source. Various kinds of mechanical source that well conform to the above requirements are available in practise. Among these the "double action air piston" used by Ismes for downhole tests is worth mentioning (Piccoli and Smits 1991, Luke et al. 1994). This is a reversible mechanical source which produces mainly shear waves propagating along a vertical direction and vibrating in the horizontal direction. It consists of a hydraulic hammer mounted onto a wooden beam with a horizontally oriented piston. It has roved satisfactory in performing measurements at depth greater than 20 m and in offshore applications.

Mechanical sources for crosshole tests generating shear waves propagating horizontally and vibrating in the complementary horizontal direction have been available since the beginning of the eighties (see Woods and Stokoe 1985). Various kinds of sources for generating horizontally polarised shear waves have been developed: electromagnetic (Ismes 1993, Fuhriman 1993), piezoceramic (Roblee 1990, Nasir 1992). The piezoceramic source originally developed by Roblee (1990) for rock sites generates high frequency signals (up to 20 kHz). It has also been successfully used by Nasir (1992) in fractured rock or soil. A piezoceramic transducer has also been used as receiver by Roblee and Nasir. The great advantage of the Ismes source (Geos HV) is that it enables one to generate both vertically and horizontally polarised shear waves thus providing the possibility to evaluate the initial small strain anisotropy in situ with great accuracy (Jamiolkowski and Lo Presti 1994).

Receivers

In crosshole tests a single axis vertically oriented transducer is often used, whilst biaxial or triaxial transducers are necessary to identify both vertically and horizontally polarised shear waves. In downhole tests the use of a pair of geophones situated at a fixed distance (Patel 1981) has increased the accuracy and the resolution of the test allowing to use the true interval method of data interpretation. A pair of sensors placed 1 meter from each other along the cone rod is currently used in SCPT tests (Campanella and Stewart 1990, Piccoli and Smits 1991)

Two systems, pneumatic and leaf spring based systems, are currently used to couple the receiver to the casing and both have proved to be satisfactory (Woods and Stokoe 1985).

Data acquisition system

Dedicated portable waveform analysers or computer based data acquisition systems in addition to the conventional electronic equipment (oscilloscopes and seismographs) have greatly increased the capability of seismic tests especially for what concern the data interpretation (Hall 1985). Thanks to these enhancements, travel time measurement can be easily accomplished by means of more accurate methods in comparison to the so called "By Eye" method. In particular, for seismic tests performed with a pair of receivers located at a known distance d, the cross-correlation algorithm (Roesler 1978) can be used to determine travel time. Cross-correlation can be used both in the time and frequency domain and various examples and studies concerning the application of this method can be found in recent literature (Woods 1991 and 1994, Campanella and Stewart 1990, Baziw 1993, Piccoli and Smits 1991, Abiss and Viggiani 1994). Moreover the increased capability of data acquisition systems have also made possible to analyse waveforms in the frequency domain in order to in situ determine the small strain damping ratio (Mok 1987, Mancuso et al. 1989, Mancuso 1994, Stewart and Campanella 1991, Fuhriman 1993, Khwaja 1993, Boore 1993, Liu et al 1993.

Test interpretation

As far as travel time determination is concerned the cross-correlation algorithm and the analysis in the frequency domain of the signals are available for this purpose.

The cross-correlation function in the time domain is given by:

$$CC(\tau) = \int_{-\infty}^{+\infty} h(t)\cdot g(t-\tau)\cdot d\tau \qquad (6)$$

where h(t) and g(t) are the waves passing trough a pair of receivers and τ is the time delay.

In principle the signal detected by the receiver closest to the source, g(t) in eq. (6) is delayed for a short period τ and multiplied by the signal detected by the other receiver h(t) in eq.(6) The above steps are repeated for increasing values of τ and each term is added up. The cross-correlation function, constructed in such a way, indicates, for which time delay, the signals show the greatest similarity, which coincides with the maximum of $CC(\tau)$. This time corresponds to the travel time of the wave between the two receivers. In practice the time delay is chosen equal to the interval between two digitised points; the calculations result to be time consuming.

The computation of the cross-correlation function in the frequency domain is more efficient. It involves the determination of the cross spectrum:

$$CS(f) = G(f)\cdot \overline{H}(f) \qquad (7)$$

where:

G(f) and H(f) are Fourier Transforms of g(t) and h(t) respectively

$\overline{H}(f)$ is the Complex Conjugate of H(f)

The cross-correlation function is the Inverse Fourier Transform of the Cross Spectrum. However it is preferable to determine the travel time and consequently the wave velocity between the two receivers as a function of frequency.

$$t(f) = \Phi(f)/360\cdot f \qquad (8)$$

$$V(f) = d/t(f) \qquad (9)$$

where: t(f) and V(f) are respectively the travel time and the velocity for a given frequency, d is the distance between two receivers and $\Phi(f)$ is the phase angle in degree of the cross-spectrum.

Appropriate values of the wave velocity are obtained in the frequency interval within which the coherence of the cross spectrum function is very close to 1.

Examples of application of cross correlation to test interpretation performed in Pisa clay deposit (Italy) are given in figures 6 and 7. Experimental data refers to SCPT executed at a depth of 20 meters. For the same data the more conventional "by eye" method for travel time identification is shown in figures 8 and 9. Two waves with reversed polarity and detected by a single receiver were used in figure 8 to accomplish the so-called "cross-over" method. This method, of course, provide in downhole tests the average velocity of the encountered soil layers. In figure 9 the "true interval" method of interpretation is shown.

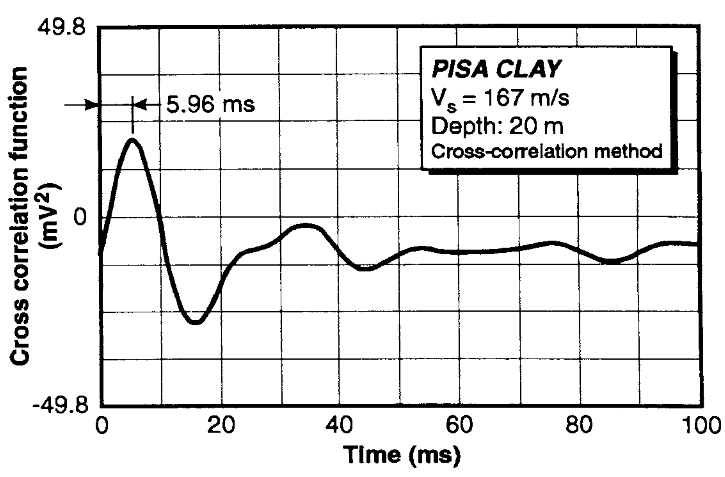

Figure 6 Down-hole seismic cone test in Pisa clay; cross correlation method (time domain).

Various methods are currently used to determine the damping ratio D_o from borehole seismic tests, among them the following are worth mentioning:

1) The spectral ratio method (Mok 1987, Fuhriman 1993) is based on the following assumptions which hold only in the far field:

- The amplitude of the body waves decreases in proportion to r^{-1}, where r is the distance from the input source, because of the so called geometric damping

- Body wave attenuation, due to material damping, is proportional to the frequency

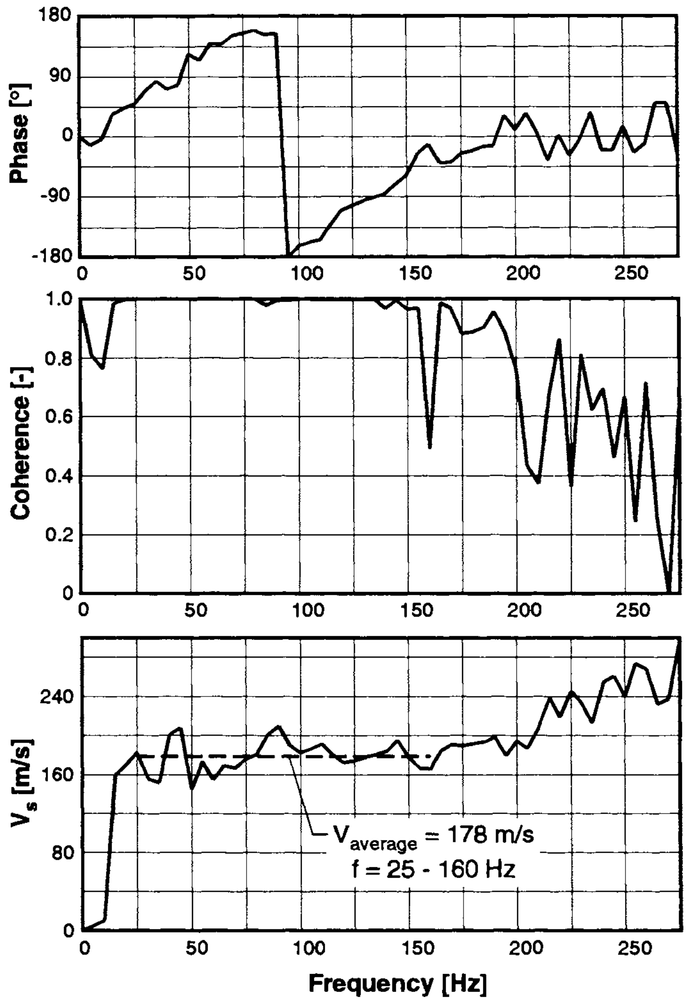

Figure 7 Down-hole seismic cone test in Pisa clay: cross correlation method (frequency domain).

- The soil-receiver transfer function can be considered identical for both receivers

Based on the above assumptions, the damping ratio can be computed by means of the following equation:

$$D(f) = \frac{\ln[A_1(f) \cdot r1 / A_2(f) \cdot r2]}{\Phi(f)} \tag{10}$$

where: r1 and r2 are the distances from the source of a pair of receivers, $A_1(f)$ and $A_2(f)$ are the amplitude spectra at the two receivers and $\Phi(f)$ is the phase of the wave velocity or phase difference between the two receivers.

2) The spectral slope method, originally developed for downhole measurements (Redpath 1982, Redpath and Lee 1986), differs from the previous one in that it assumes that material damping is frequency independent and that it is not necessary to define the law for geometric damping.

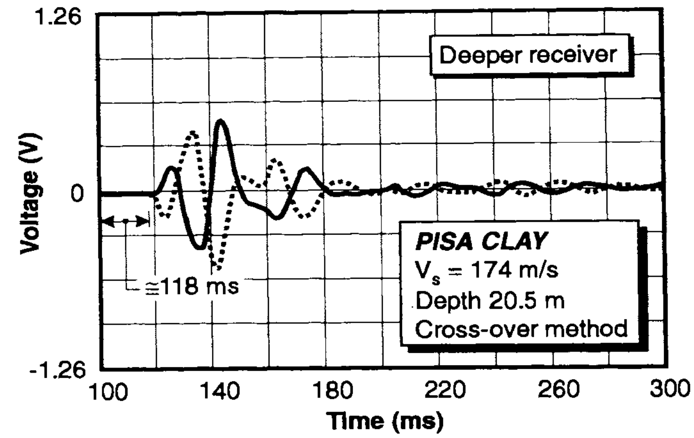

Figure 8 Down-hole seismic cone tests in Pisa clay; cross-over method.

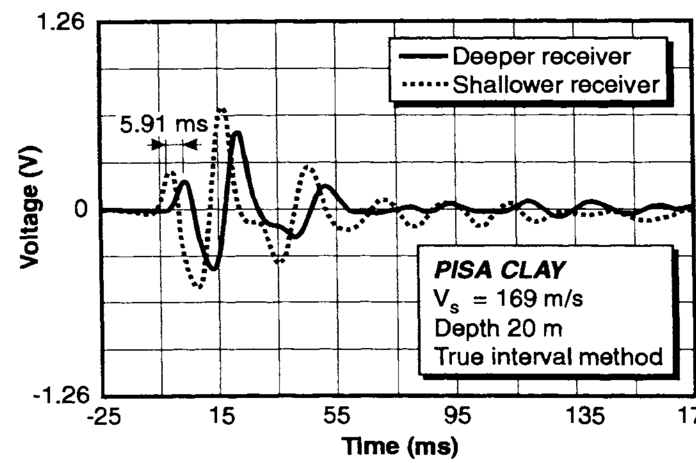

Figure 9 Down-hole seismic cone tests in Pisa clay; true interval method.

The attenuation constant, defined as the ratio of attenuation to frequency $k = \alpha / f$ represents the spectral slope, i.e. the slope of the spectral ratio against frequency:

$$k = \frac{-\Delta\{\ln[A_1(f) / A_2(f)]\}}{\Delta f (r2 - r1)} \tag{11}$$

therefore material damping can be computed by means of the following expression:

$$D(f) = \frac{-\Delta\{\ln[A_1(f)/A_2(f)]\}}{\Delta f \cdot 2\pi \cdot \Delta t(f)} \qquad (12)$$

Both previously described methods require signal filtering. They provide damping values against frequency, which are significant within the filter bandwidth. Khawaja (1993) and Fuhriman (1993) recommend to perform crosshole tests with four boreholes, in order to obtain stable values of damping by means of the spectral ratio method. They suggest using the two extreme boreholes to propagate waves in forward or reverse directions, keeping the receivers in the two central boreholes. The spectral ratio method with combined directions provides stable values of damping and avoids the extreme case of negative damping values also observed by other researchers (Campanella and Stewart 1990). Campanella and Stewart (1990) studied the applicability of the above described methods to the downhole SCPT's. They found that the spectral slope method provides more realistic values of material damping. However, in downhole tests, wave amplitudes are also attenuated for refraction/reflection phoenomena and due to the effect of the ray path curvature.

The influence of ray path curvature on the determination of the shear wave velocity in crosshole tests was studied by Hryciw (1989) who found that a correction, to account for this phoenomenon, is necessary at shallow depth in uncemented cohesionless soils.

Other approaches such as the rise time method, based on the experimental evidence that seismic waves broaden with distance from the source as a consequence of the material damping, are also used to assess D_o in the field. However none of available methods, based on borehole seismic tests, seems to be sufficiently consolidated and enough reliable in order to be recommendable in the every day practice.

Examples of damping computation, with the spectral ratio and spectral slope methods, are given in figure 10 (Fuhriman 1993).

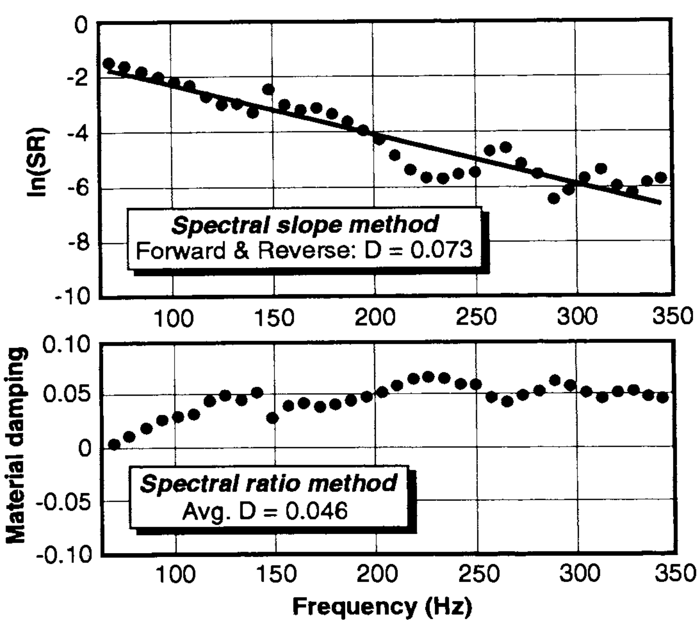

Figure 10 Material Damping at 4.5 m depth at Gilroy no. 2 (Fuhriman, 1993)

Interpretation of downhole seismic tests based on the so called "inversion method" (Mok et al. 1989), provides more accurate shear wave profiles which result to be comparable to those obtained by means of crosshole tests. The method is also capable of accounting for ray path curvature.

The use of tomographic techniques in geotechnical investigations has become possible thanks to the increased capability of electronic equipment and the consequent development of computerised procedures for the travel time determination. In geotomography, electromagnetic waves can be used besides mechanical waves. Cross hole, down hole and SCPT tests can be used to generate mechanical waves which are detected by a series of receivers according to the scheme of figure 11.

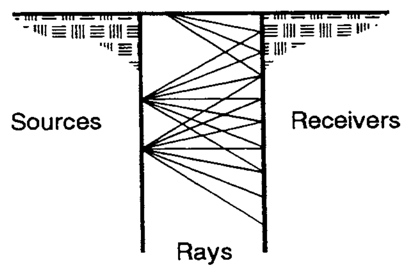

Figure 11 Schematic geotechnical tomography.

The basic equations of tomographic inversion can be written in matricial notation as:

$$\underline{T} = L\underline{S} \qquad (13)$$

where $\underline{T}$ is the vector of n measured travel times, $\underline{S}$ is the vector of the m unknown slownesses (slowness=1/velocity) and L is the nxm matrix of travel lengths.

If n>m the problem is overdetermined and can be solved, for example, by means of the least square method. Basic information and mathematical methods, available for tomographic inversion, can be found in Tallin and Santamarina (1990) and Santamarina (1994).

Geotomography is mainly suitable for detecting subsurface anomalies such as fracture zones in rock or such as very soft inclusions.

4.1.2 Surface methods

The Spectral-Analysis-of-Surface-Waves (SASW) (Nazarian and Stokoe 1983, 1984, Nazarian et al. 1983) technique is the most important innovation in the field of seismic surface methods. Stokoe et al (1994b) provided a very useful review of the method for both terrestrial and offshore applications.

The key aspects of the method are here briefly summarised:

- In terrestrial applications Rayleigh waves are propagated by means of impulsive mechanical sources and are detected by a pair of transducers located at different distances (r1 and r2) from the source (see Figs. 12 and 13). Waveforms with a high frequency content (short wavelength) propagate in the shallower strata; the deeper strata are investigated by generating waveforms with longer wavelengths. - the signals at the receivers are digitised and recorded by a dynamic signal analyser. The Fast Fourier Transform (FFT) is computed for each signal. Thus the cross power spectrum and coherence can be computed. The coherence function indicates, for which frequency interval, meaningful measurements have been obtained. On the other hand, the phase angle of the cross power spectrum enable ones to determine the travel time between the two receivers and consequently the Rayleigh wave velocity according to eqs 8 and 9 (d=r2-r1). The wavelengths corresponding to the frequency dependent Rayleigh wave velocities are computed as $L_R = V_R(f)/f$. It is thus possible to plot the surface wave phase velocity versus wavelength (field dispersion curve). The above mentioned data analysis is performed in real time.

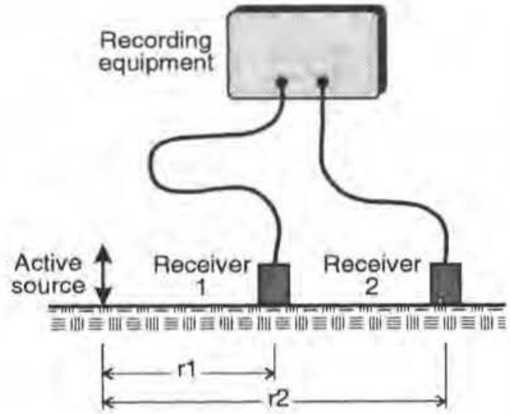

Figure 12 Basic configuration for SASW test.

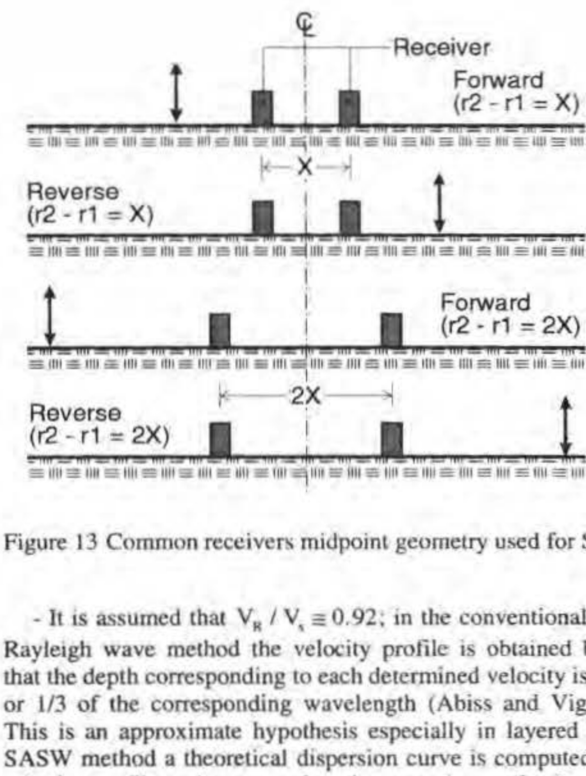

Figure 13 Common receivers midpoint geometry used for SASW test.

- It is assumed that $V_R / V_s \cong 0.92$; in the conventional steady state Rayleigh wave method the velocity profile is obtained by assuming that the depth corresponding to each determined velocity is equal to 1/2 or 1/3 of the corresponding wavelength (Abiss and Viggiani 1994). This is an approximate hypothesis especially in layered soils. In the SASW method a theoretical dispersion curve is computed for a given velocity profile and compared to that experimentally determined. The soil profile and the velocities assigned to each stratum are adjusted until the theoretical curve matches the experimental one (see Fig. 14).

Experimental (Gauer 1990) and theoretical (Manesh 1991) studies have shown that it is possible to use the SASW technique offshore. As already mentioned Luke et al (1994) have shown applications of SASW in offshore sites. Difficulties in test interpretation arise in the case of very stiff seafloors Stokoe et al (1994b).

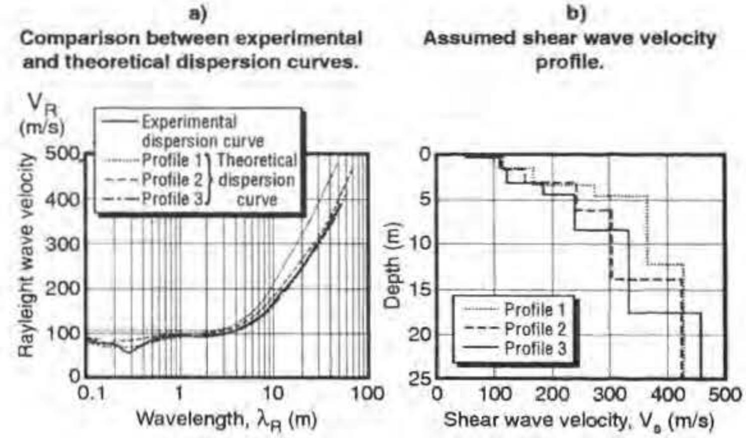

Figure 14 Illustration of forward modelling procedure used in SASW (Stokoe et al, 1994).

4.2 Large strain tests

Many kinds of pressuremeter probes are currently used in different countries (Amar et al. 1991, Clark 1994).

The common features of all these devices is that, at least in principle, the test should model the expansion of a cylindrical cavity. This implies that the length to diameter ratio of the pressuremeter probe should be sufficiently large, at least equal to 10, in order to fulfill the above mentioned postulate.

The differences among various probes are mostly related to the way in which they are inserted into the ground, i.e.: predrilled hole, push-in, self-bored etc. The information of major interest that can be obtained from the pressuremeter tests is the unload-reload shear modulus G_{ur} (Bellotti et al. 1989, Fahey 1991, Fahey and Carter 1993 Ghionna et al. 1994), the ultimate limit cavity stress p_u and, at least in principle, solely in the case of SBPT, the value of the horizontal total in situ stress σ_{ho}. In the field of GEE the special interest is devoted to the use of the SBPT in sands with the aim to assess G_{ur} from unload-reload loops performed under completely drained conditions. The value of G_{ur} is usually assessed in the range of the shear strain amplitude at the cavity wall $\Delta\gamma$ ranging from between 0.1 and 0.3 %. A rational use of the so determined G_{ur} must pass through the assessment of the average plane effective stress $s'_{AV} = \frac{1}{2}(\sigma'_r + \sigma'_\theta)$ and average plane shear strain $\gamma_{AV} = \varepsilon_r - \varepsilon_\theta$ existing around the expanding cavity during the execution of the loop. The θ and r subscripts indicate tangential and radial stress and strain components respectively. It is possible to locate the G_{ur} on the G vs. γ curves resulting from laboratory tests only in the case that s'_{AV} and γ_{AV} have been determined (Bellotti et al. 1989, Fahey 1991, Clark 1994). In doing so, it should be remembered that, in the frame of the theory of elasticity of a transverse-isotropic medium the G_{ur} corresponds to G_{hh}. Attempts have also been made to link the measured value of G_{ur} to the initial tangent shear stiffness G_o of the tested soils when subject to the geostatic effective stresses (Ghionna et al. 1994).

Cyclic pressuremeter tests have also been attempted with the aim of measuring the cyclic shear modulus G_{ur} and damping ratio D of various kinds of soils at large strains in situ (Mori and Tsuchiya 1981, Briaud et al 1983, Yoshida et al. 1984). The use of cyclic pressuremeter probes, generally installed in predrilled holes, does not enter into the current soil investigation practice. This probably can be explained by the already mentioned difficulties of linking the measured G and D values to the relevant s' and γ level as well as by the uncertainty related to the disturbance due to the fact that tests are carried out against the wall of the predrilled hole.

Research is underway in the USA with the aim of developing a tool which allows one to subject a soil element in situ to cyclic shear strain whose magnitude exceeds ε_t^e. A prototype for cylindrical shear tests is already available (Henke and Henke 1991, Henke and Henke 1992, Henke and Henke 1994). Tests are performed at the bottom of a borehole by pushing into the soil a dual concentric cylinder device (Henke and Henke 1991) or a single cylinder device (Henke and Henke 1992, 1994). In the first case anular samples are tested, while in the second case the soil surrounding the probe is subjected to test. In both cases the estimated vertical geostatic stress can be restored while the horizontal geostatic stress is affected by the disturbance induced during the probe penetration. Stress-controlled cyclic shear tests were conducted using both probes. Tests were mainly performed in the laboratory under controlled conditions in order to assess liquefaction susceptibility.

Impulsive shear tests were also performed with the single cylinder device (Henke and Henke 1993, 1994). The acceleration time history was recorded during these tests. Soil characteristics (G and D) were inferred by means of a simplified non-linear back analysis for a strain interval of 0.004 - 0.1 %.

A more advanced tool is under study at the California Department of Transportation (Li et al. 1993). The device is aimed at:

- Obtaining a free-standing cylindrical deep specimen by means of the self-boring method, which reduces soil disturbance.
- Restoring in situ geostatic stresses.
- Performing a low strain, controllable frequency, resonant column test, followed by large strain torsional shear test;
- Measuring the pore pressure during above tests.

The feasibility of the self-boring sample preparation method has already been experimentally investigated. The testing sequence and procedure, as well as test interpretation method, have also been studied.

The In-Situ Resonance tests and Torsional shear tests on large size (diameter 10 m, height 5 ro 9 m) columns of gravely soils which were recently performed by Konno et al (1991) satisfy the above listed requirements. In this case a large volume of soil is subjected to torsional shear, instead of a small volume soil element typical for laboratory experiments. The resonance frequencies, experienced in these tests, were in the range of 5 to 8 Hz much lower than those usually employed in RC tests.

4.3 Model tests

Plate Load Tests (PLT's) are performed in many countries using various standards. At the beginning of the test the model foundation is subject to a small load increment in order to obtain good contact with the soil. The plate settlement, which results from the application of this preload, is there after disregarded in the test interpretation. PLT's are usually performed on shallow model foundations of different shapes and sizes. The screw plate or flat plate tests are performed at the bottom of properly prepaired boreholes (Smith 1987) and are used to infer the operational stiffness of soils at depth. Cyclic PLT's are also performed both for shallow (Prakash and Puri 1981) and deep (Andreasson 1981) conditions.

The results of these tests are usually interpreted to obtain the operational Young's modulus by means of the following formula of the theory of elasticity which holds in the case of rigid circular plates laying on or within an isotropic, homogeneous elastic half-space:

$$E = \frac{\Delta p}{\Delta s} D_p \cdot (1 - \nu^2) \cdot I_w \cdot C_d \tag{14}$$

where E can be both a tangent or a secant Young's modulus, D_p is the diameter of the plate, ν is the Poisson's ratio, I_w is a shape factor and C_d is a non-dimensional factor which depends on the plate embeddement. The terms Δp and Δs are the measured load and displacement respectively. A more sophisticated back-analysis can be performed by means of a numerical analysis of the test data.

Resonant Footing tests are performed in various countries with different standards (Prakash and Puri 1981). The procedure proposed by Pang (1972), using only torsional vibrations, seems highly preferable because the torsional vibration is an uncoupled motion and may be treated independently. The assessment of equivalent shear modulus and damping is possible in this kind of test after the assumption of a soil model and a back analysis.

4.4 Penetration tests

During the eighties, penetration tests underwent significant innovations which may be summarised as follows:

- Recognition of the importance of the energy delivered to the rods during execution of the SPT (Schmertmann and Palacios 1979, Kovacs and Salomone 1982). of dynamic penetration tests with samplers larger than that employed for SPT (Yoshida 1988, Yoshida et al. 1988, Hatanaka et al. 1988 and 1989, Konno et al. 1991, Goto et al. 1992, Suzuki et al. 1992 and 1993, Crova et al. 1992, Harder and Seed 1986). This kind of test, named Large Penetration Test (LPT), is especially suited for coarse grained gravelly soils. Examples of LPT's performed in medium to coarse Po river sand and in Messina gravelly sand are compared with the results of the SPT's in figure 15. For both SPT and LPT the energy ratio ER expressed as the energy effectively delivered to the driving rod divided by the nominal energy is also displayed (Crova et al. 1992).
- Incorporation of the porous stone into the standard electric CPT tip which allows continous measurements of the pore pressure during penetration at preselected elevations (Baligh et al. 1981, Campanella and Robertson 1981, Muromachi 1981, Tumay et al. 1981, De Ruiter 1981, Smits 1982).
- Development of Marchetti's flat dilatometer, followed later on by implementation of the device with a sensor that allows one to monitor the process of expansion of the dilatometer blade in a continous manner (Marchetti 1980, Motan and Khan 1988, Fretti et al. 1993).
- Creation of a number of multipurpose CPT and CPTU probes which, in addition to the cone resistance q_c, penetration pore pressure u and local shaft friction f_s, are able to assess additional parameters of the penetrated soil thus making the test interpretation easier or/and more comprehensive. As examples of this new generation of the static cone penetration tests one can mention:
  - The Pressiocone (PCPT), which allows one to assess p_u and G_{ur} (Jezequel et al. 1982, Hughes and Robertson 1985, Whiters et al. 1986, Sully 1991)
  - The already mentioned Seismic Cone (SCPT), which allows one to obtain quasi-continous V_s profiles in sand and clay deposits down to the depth of 40-50 m (Campanella and Robertson 1984, Baldi et al. 1988). An example of SCPT results obtained at Garigliano site (Italy) is given in figure 16 where they are compared against the shear wave velocities measured during cross-hole tests.
  - The Resistivity Cone (RCPT), which allows one to measure the electrical resistivity (ρ_{soil}) of the penetrated soil (Vlasblom 1973, Campanella and Weemees 1990). The measured bulk resistivity of soils depends on both the pore fluid (ρ_{pf}) and soil skeleton (ρ_{ss})

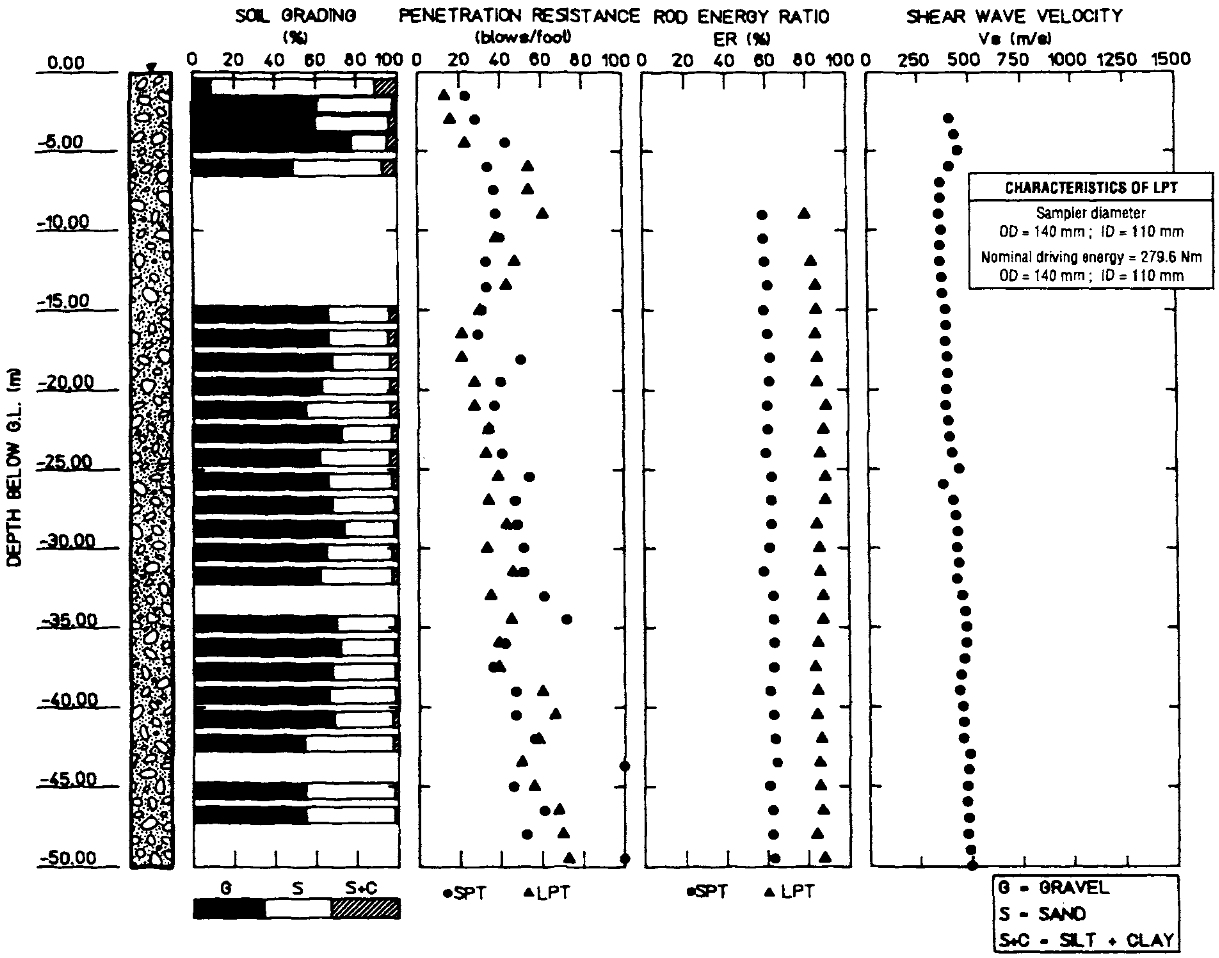

Figure 15 Messina straits crossing. SPT and LPT penetration resistance in Pleistocene sand and gravel.

resistivity. The formation factor ($F = \rho_{soil} / \rho_{pf}$) is a simplified way of isolating the resistivity of the soil skeleton and can be linked to the in situ porosity and soil structure thus offering the possibility of assessing the liquefaction susceptibility (Arulmoli et al. 1985). Other researchers have used the soil resistivity for groundwater contamination studies (Campanella and Weemees 1990). A resistivity probe, which is inserted into the soil by means of less destructive methods, such as the self-boring method, and which allows one to determine the formation

- Development, calibration and usefactor in both vertical and horizontal directions, has been developed by Bellotti et al. (1994) in order to assess in situ void ratio.

4.5 Seismic Arrays

Attempts at generating in situ large strain levels in order to determine in situ large strain shear modulus were performed in the seventies using the so called Cylindrical In Situ Test (CIST) or In Situ Impulse Test (see Wilson et al 1978, Bratton and Higgins 1978). These attempts were not very successful (Chang et al. 1991).

In the eighties many sites, located in active earthquake zones, were monitored to observe both free field and prototype (or large scale model) response to earthquake motion. The most famous experimentation sites are:

- SMART1, a near-source array of digital seismographs located in the town of Lotung in the Northeast corner of Taiwan (Bolt et al 1982, Loh et al. 1982);
- The Chiba seismometer array located at Chiba, 30 km east of Tokyo, Japan (Katayama et al. 1990a, 1990b)
- The Lotung surface and downhole accelerometer array (Tang 1987)
- The Tokyo Haneda airport site (Noda et al. 1988)
- The Suruga Bay-Izu region (Okubo et al. 1984)

Data from the downhole Lotung array was used by Chang et al. (1991) for back-calculation of shear moduli. Huerta et al. (1994) tried to use free field accelerometric records to back calculate material damping by means of the logarithm decrement method.

5. DEFORMATION AND DAMPING CHARACTERISTICS

The following section is focused on the discussion regarding the methods allowing to determine deformation and damping characteristics of soils in situ. The currently available methods have been discussed in detail by Tatsuoka and Shibuya 1992. These methods can be grouped as follows:

- Estimate of the in situ operational shear modulus G(F) on the basis of the small strain shear modulus determined in the field by means of seismic tests $[G_o(F)]$ and of the G/G_o vs. γ decay curve obtained from laboratory tests.
- Assessing G and E at different displacement levels from SBPT's and PLT's respectively and thereafter in combination with G_o from seismic in situ tests attempting to envisage the entire G/G_o vs. γ curve.

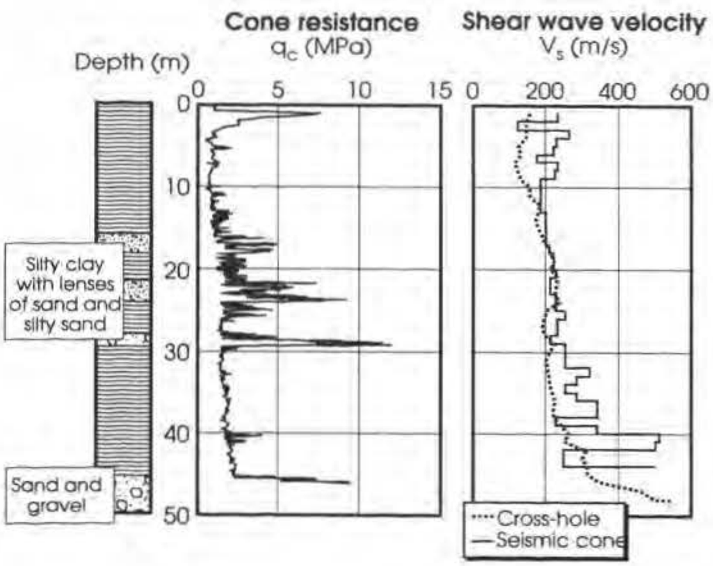

Figure 16 Seismic cone test at Garigliano site (ISMES, 1993).

- Relying on empirical correlations between G or E and results of various penetration tests.

As far as the assessment of D in situ is concerned, many researchers (see Table 4) have tried to determine the small strain damping ratio D_o from seismic tests according to one of the previously described methods.

The back analysis of in situ soil model tests (Konno et al. 1991) and the use of the logarithm decrement method to the transient response of free field earthquake records (Huerta et al. 1994) have been used to obtain D vs. γ curves.

5.1 Small strain stiffness

As already mentioned, various seismic tests make it possible to measure G_o. Data obtained by means of different tests are generally consistent among them if one takes properly into account the characteristics of the prapagated body waves and the effective stress state of the soil. A systematic comparison of the results obtained with the most commonly used techniques is here reported. Data was recently obtained in the USA in sites that liquefied during the 1983 Borah Peak (Andrus 1994) and the 1989 Loma Prieta Earthquakes (Mitchell et al. 1994). The ratio of V_s measured using SASW, SCPT and Up-hole techniques to that obtained from crosshole tests is plotted in figure 17. As known, the SASW method gives average values of the shear wave velocity which involves a certain underestimate of V_s in comparison to the crosshole measurements. Seismic cone results are in very good agreement to those of the crosshole, while the disparity between Up-hole and crosshole measurements is not very large.

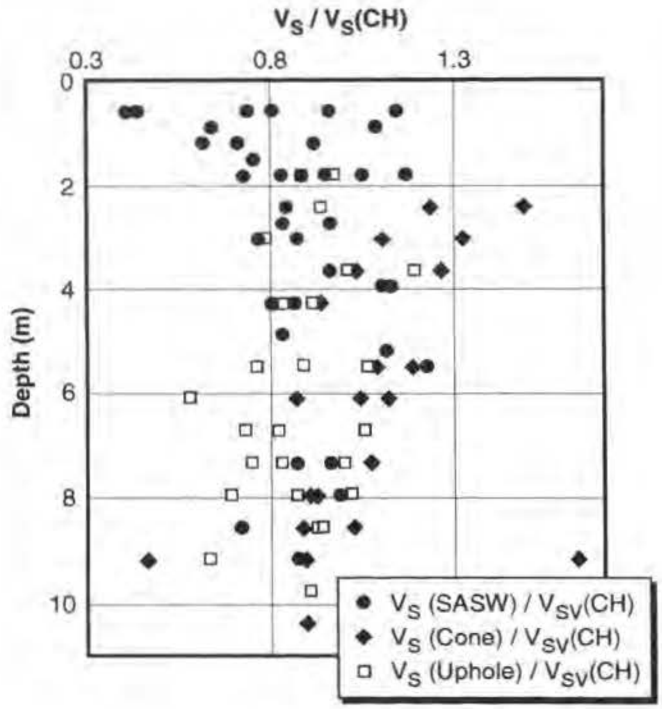

Figure 17 Seismic tests: comparison of field data obtained by different methods (Data from Mitchell et al, 1994; Andrus, 1994).

Investigations performed in CC on reconstituted sand specimens have shown a small degree of inherent anisotropy of soil stiffness at small strains. In particular, research undertaken at the Texas University at Austin (Stokoe et al 1985, 1991), and in Italy (Lo Presti and O'Neill 1991 and Bellotti et al 1995) have shown that under isotropic consolidation stresses the inherent anisotropy, quantified by means of the ratio of moduli referred to vertical and horizontal directions is quite small:

$$G_{hh} / G_{vh} \text{ and } M_h / M_v \text{ and } E_h / E_v \approx 1.2 \qquad (15)$$

The small influence of the inherent anisotropy on the small strain stiffness is confirmed by field data summarised in Table 3. The only exception is represented by the highly structured soils in Bolsom fills investigated by Stokoe et al (1992) and Nasir (1992). It should be considered that field data reflect both inherent and stress induced anisotropy. According to Roesler (1979), Stokoe et al (1985, 1991) Lewis (1990) Lee (1993) and Bellotti et al. (1995) the ratios of moduli previously mentioned depend on both inherent anisotropy and consolidation stress ratio, as shown for example in figure 18 Only for K=1 the ratios of moduli reflect only the inherent anisotropy. For data summarised in Table 3 K_o ranges from 0.4 to 0.85 (Bolsom fill is not considered); it is possible to plot them on figure 18 obtaining a good agreement with CC data also considering stress-induced anisotropy. The main conclusion is that even if for anisotropic soils Young's and shear moduli are independent from each other it seems reasonable to compute E_o from G_o by means of the well known relationship established in the case of elastic isotropic media:

$$E_o = 2 \cdot (1+\nu) \cdot \rho \cdot V_s^2 \qquad (16)$$

This formula can be used in combination with values of Poisson's ratio ranging between 0.1 and 0.2. Such a values have been determined at small strains in the laboratory for various kind of soils (Tatsuoka and Kohata 1994, Bellotti et al. 1995)

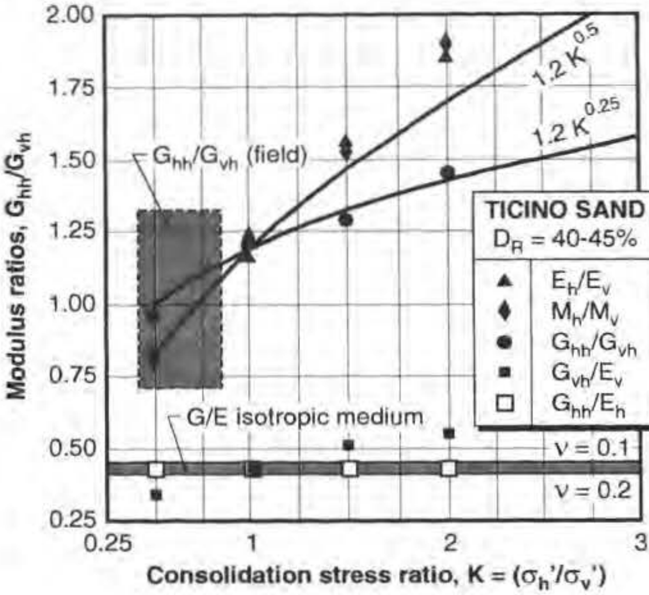

Figure 18 Small strain stiffness anisotropy: field versus laboratory data (Adapted from Bellotti et al, 1994).

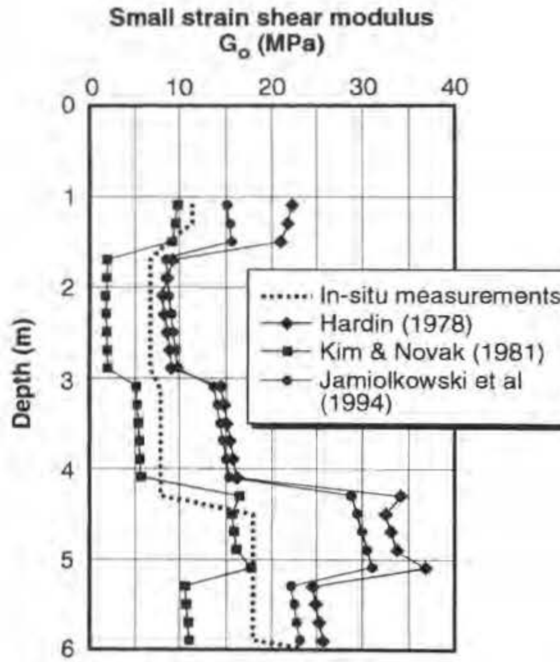

Figure 19 Champlain clay: values of G_o measured vs. those inferred from different empirical correlations.

A greater inherent anisotropy was shown by Jamiolkowski et al. (1994) in the case of some natural Italian clays tested in the laboratory, while the trend for G_{hh} / G_{vh} vs. K_o was similar to that observed for reconstituted sands.

Empirical correlations between the penetration resistance from SPT (Ohta and Goto 1978, Imai and Tonouchi 1982) or CPT (Sycora and Stokoe 1983, Robertson and Campanella 1983, Rix 1984, Baldi et al 1986, 1989a, 1989b, Bellotti et al. 1986, Lo Presti and Lai 1989, Rix and Stokoe 1991) and the small strain shear modulus G_o have been established using different database.

Due to their purely empirical nature, these correlations provide just an approximate estimate of G_o which in many cases can be quite poor as shown by Thomann and Hryciw (1991).

Recently Mayne and Rix (1993) have proposed the following empirical correlation between the shear wave velocity and the cone penetration resistance:

$$G_o = 99.5(p_a)^{0.305}(q_c)^{0.695} / e_o^{1.130} \tag{17}$$

This correlation is based on database from 31 clay sites and takes into account the in situ void ratio which significantly influences G_o, whilst has much smaller impact on q_c. The above correlation provide an estimate of G_o with a possible error of about ± 100 %.

Evaluation of G_o with the same degree of uncertainty can be attempted using correlations based on laboratory tests (e.g. Hardin and Blandford 1989). An example of such correlations is given by the following formula (Jamiolkowski et al. 1991,1994)

$$G_o = 600(\sigma_m')^{0.5} p_a^{0.5} / e^{1.3} \tag{18}$$

which requires the knowledge of σ_m' and e

A comparison between the values of G_o, evaluated using this type of laboratory correlations (Hardin 1978, Kim and Novak 1981, Jamiolkowski et al. 1991,1994), and those measured in situ in a structured Champlain clay deposit (Lefebvre et al. 1994) is shown in figure 19.

5.2 Large strain stiffness

Pressuremeter and plate load tests can provide an average stiffness linked to the load vs. displacement characteristics of the considered boundary value problem. Usually the values of G or E from these tests are evaluated using the formulae of the theory of elasticity of an isotropic medium. In order to refer so obtained values of stiffnesses to those of the volume element of soil the proper consideration to the average stress and strain levels involved, should be given (Tatsuoka and Shibuya 1992, Tatsuoka and Kohata 1994),

Only if this is done successfully and knowing G_o or E_o from in situ seismic tests, the assessment of the elementary soil stiffness can be attempted.

Various methods have been proposed to infer a complete elemental stress-strain relationship from pressuremeter tests for both cohesive (Palmer 1972, Muir Wood 1990, Jardine 1992b) and cohesionless deposits (Bellotti et al. 1986, 1989, Manassero 1989, Byrne et al. 1991, Ghionna et al. 1994, Fahey and Carter 1993).

The basic hypotheses commonly assumed when interpreting pressuremeter tests are:

- Plane strain condition confined to the horizontal plane ($\varepsilon_z = 0$) which is strictly verified in horizontally homogeneous soils and for a probe with a length to diameter ratio l_p / d_p approaching ∞.

- Drained conditions in cohesionless soils. In cohesive soils it is assumed that cavity expansion occurs under fully undrained conditions even though this hypothesis is never fully verified.

- Values of the initial stresses at the boundary are well known and correspond, in the case of SBP, to the in situ horizontal geostatic stress.
- A simplified elastic perfectly - plastic constitutive model is commonly considered.

Different methods are available to obtain the entire stress-strain curve from the SBPT:

- Direct application of closed - form solutions, which involves the numerical differentiation of raw pressuremeter data (Palmer 1972).
- Application of the available closed - form solutions to a curve fitting the raw pressuremeter data (Muir Wood 1990).
- Use of simple relationships which correlate the cavity strain to an elemental strain (Bellotti et al. 1986, 1989, Jardine 1992b, Ghionna et al. 1994).
- Finite element analysis of the pressuremeter expansion curve (Byrne et al. 1991, Fahey and Carter 1993). This approach enables one to assume more realistic constitutive model for soils.

The small unload-reload loops performed during pressuremeter tests provide values of G_{ur} that are less influenced by the disturbance of the surrounding soil caused by the probe insertion and are more easily interpreted within a consistent theoretical framework. However, also in this case it is very difficult to measure G_{ur} at cavity strains of less than 0.05 or 0.1 % because of the inadequate accuracy and compliance of the strain measuring system. Due to these limitations it seems preferable to use pressuremeter data in conjunction with field seismic test results and/or laboratory stress-strain curves, as suggested by Tatsuoka and Shibuya (1992) and Jardine (1992b). The validity of this approach is supported by the following observations:

- The small strain stiffness, determined in the laboratory on high quality undisturbed samples using appropriate apparatuses and procedures, is very close to that obtained in situ from seismic tests (Tatsuoka and Shibuya 1992, Jamiolkowski et al. 1994, Tatsuoka and Kohata 1994, Tatsuoka et al. 1995).
- The normalised shear modulus reduction curve (G/G_o vs. shear strain or shear stress level) obtained in the laboratory on high quality samples by applying strain rates consistent with those experienced in situ are comparable with those curves inferred by means of back analysis of field data or from in situ tests (Seismic tests plus SBPT or PLT) (Chang et al. 1991, Tatsuoka and Shibuya 1992, Tatsuoka and Kohata 1994, Henke and Henke 1993). With this respect, data by Chang et. al (1991), reported in figure 20, clearly indicate that the reduction of G in silty sand which can be obtained from back-analysis of down-hole seismic array data is much more pronounced than than observed in the laboratory during RC tests. This different behaviour might be attributed to high strain rates and large N experienced by the soil samples in the laboratory.

On the other hand, the main problems which have to be overcome when trying to link G_{ur} to G_o and to the G/G_o vs. γ laboratory curve are the following:

- The influence of the disturbance induced by the probe insertion.
- The fact that G_{ur} reflects the soil stiffness during the first unload-reload loop and not that related to the soil response to virgin loading (backbone curve).

TABLE 3 Anisotropy of shear wave velocity from in situ tests

Reference	Soil	V^s_{hh} / V^s_{vh}	Site
Jamiolkowski and Lo Presti (1994)	Silty sand and silty clay strata	1.00 to 1.10 (*)	Montalto di Castro (Italy)
Mitchell et al. (1994)	Sand and Gravel	0.88 to 1.10 (*)	S. Francisco - Oakland Bay Bridge Toll Plaza (SFOBB 1)
Mitchell et al. (1994)	Sand with fines	0.85 to 1.04 (*)	Alameda Bay Farm Island (Dike)
Mitchell et al. (1994)	Sand and clay strata	0.86 to 1.16 (*)	Alameda Bay Farm Island (South Loop Road)
Mitchell et al. (1994)	Sandy clay, silty clay and clay strata	0.93 to 1.12 (*)	Port of Richmond (Hall Avenue)
Mitchell et al. (1994)	Sandy clay, silt and clay strata	0.93 to 1.08 (*)	Port of Richmond (POR 2)
Mitchell et al. (1994)	Poorly graded sand	0.82 to 1.00 (*)	Port of Oakland (P007)
Andrus (1994)	Silty sand and gravel to sandy gravel	0.85 to 1.03 (***)	Pence Ranch Idaho
Andrus (1994)	Sandy gravel from loose to medium dense	0.85-1.15 (*)	Andersen Bar Idaho
Andrus (1994)	Silry sand to sandy gravel	0.85-1.2 (*)	Larter Ranch Idaho
Fuhriman (1993)	Quaternary alluvium Bay Mud	0.91 to 1.14 (*) 0.90 to 1.11 (*)	Gilroy No. 2 Treasure Island
Stokoe et al. (1992) Nasir (1992)	Bolson fill	0.75 to 1.41 (**) 0.57 to 1.08 (**)	Site A Site B Fort Hankock (Texas)

(*) Typical values
(**) Range
(***) A unit between 1.5 and 3 meter depth of sandy gravel and gravelly sand has typical values between 0.6 and 0.8.

- The influence of anisotropy on the soil stiffness at large strains. It should be remembered that the pressuremeter modulus is a G_{hh} shear modulus while that usually obtained from seismic tests or laboratory tests is a G_{vh} shear modulus.

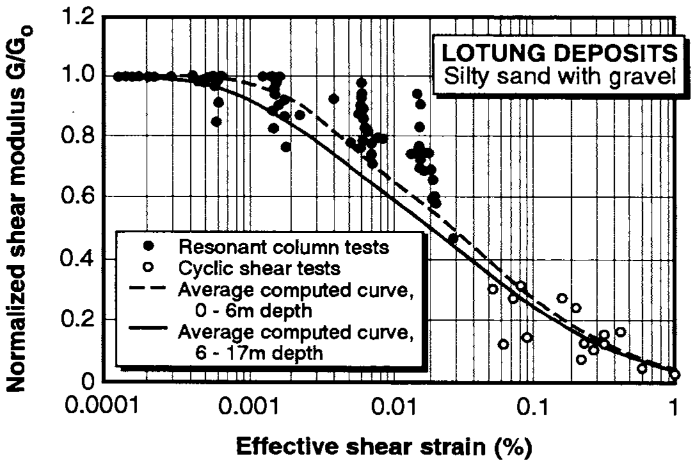

Figure 20 Comparison of back-calculated normalized shear modulus with data from laboratory tests (Chang et al, 191).

Due to the difficulties in obtaining undisturbed samples of cohesionless soils, many researchers have focused their attention on the possibility of estimating the field shear stiffness G(F) at $\gamma > \gamma_t^e$ on the basis of the knowledge $G_o(F)$ measured in situ and the G vs. γ decay curve obtained from laboratory tests performed on samples reconstituted at the best estimate of the in situ soil density.. The results obtained so far are contradictory. Ishihara (1993a), examining the results of hollow cylinder cyclic torsional shear tests carried out by Katayama et al. (1986) on both undisturbed samples of a dense sand deposit which were retrieved by means of in situ freezing and specimens of the same soil reconstituted in the laboratory with the in situ void ratio, proposed the procedure to evaluate G(F) which can be approximated by the following formula:

$$\frac{G_s(F)}{G_o(F)} = \omega(\gamma)\cdot\frac{G_s(L)}{G_o(L)} \tag{19}$$

where:

$G_o(F)$ = maximum shear modulus from in situ seismic tests

$G_o(L)$ = maximum shear modulus measured in the laboratory on reconstituted specimens

$G_s(F)$ = expected field secant shear modulus

$G_s(L)$ = secant shear modulus measured in the laboratory on reconstituted specimens

$\omega(\gamma)$ =	2.00	1.67	1.20	1.00
γ =	10^{-5}	10^{-4}	10^{-3}	10^{-2}

This formula attempts to take into account, by means of the coefficient ω, the fact that the shear modulus is much more sensitive to the disturbance or destructuration at small and intermediate strains than at large strains.

Cyclic triaxial test results provided by Hatanaka et al 1988, 1989 and by Suzuki et al. 1992 for gravelly soils confirm the above indications whilst the experimental data by Goto et al 1987, 1992 and Stokoe et al. 1994b seems to postulate the uniqueness of the G/G_o vs. γ curves for both undisturbed and reconstituted specimens. Experimental data provided by Stokoe et al. (1994a) concerns cemented sand samples and was obtained from torsional shear and resonant column tests. The authors believe that, at present, it is not possible to draw a definitive conclusion regarding the above discussed point. The uncertainties mostly arises from the fact that:

- It is difficult to compare the quality of the undisturbed samples used by different researchers.
- Differences in laboratory equipment used by the above mentioned researchers. This aspect might result especially relevant as far as type and location of the strain measuring sensors are concerned.

The authors believe that in order to obtain a reliable comparison of the stiffness of undisturbed and reconstituted specimens from TX tests, the use of local gauges for local axial strain measurements is imperative (Tatsuoka et al. 1995). The use of local gauges infact eliminates any eventual differences between reconstituted and undisturbed specimens which could arise from different degrees of asperities of the end surfaces due to different preparation methods.

In the case when laboratory G/G_o vs. γ curves have been obtained from tests on high quality undisturbed samples, reconsolidated to the best estimate of geostatic stresses, i.e. when $G_o(F) \cong G_o(L)$, Tatsuoka et al. (1995) suggest the following relation in order to infer G(F):

$$G_s(F) = G_s(L)\cdot\frac{G_o(F)}{G_o(L)} \tag{20}$$

This formula coincides with eq. (19) assuming that $\omega(\gamma)=1$ over the entire strain range.

5.3 Damping

Figure 21 (Pallara 1995) shows the damping ratio of a crushable carbonatic sand determined in the laboratory, by means of cyclic TS and RC tests performed on hollow specimens under similar test conditions, using the same TS/RC apparatus (Lo Presti et al. 1993). The cyclic torsional shear test was performed by using a triangle wave with a frequency of 0.1 Hz. The measurements of the applied torque and of the top cap rotation were contemporary achieved by means of three digital voltmeters, receiving the same trigger signal to start data acquisition. Contemporary acquisition of torque and rotation is considered imperative by Toki et al. (1994) to avoid underestimate or overestimate of damping values (Kohata et al 1993).

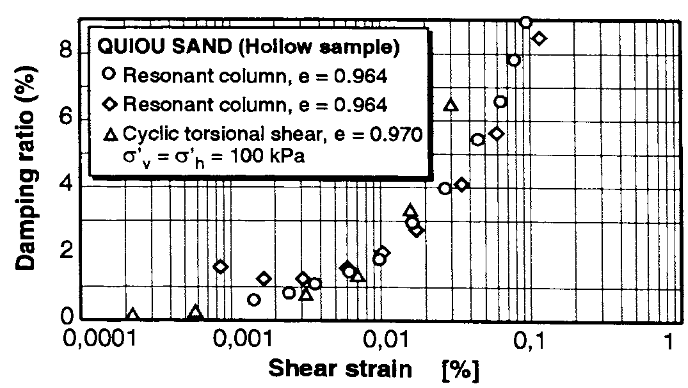

Figure 21 Damping ratio of Quiou sand from laboratory tests (Pallara, 1995).

In the above figure the damping from the RC test is larger than that from the TS test at small strains, while the opposite occurs at larger strains. Similar results were obtained by Papa et al. (1988), Teachavorasinskun et al. (1991), Stokoe et al. (1994a) for various kind of geomaterials. Larger values of D from RC tests at small strains are mainly due to the following factors:

- TS/RC equipment generates an apparent damping ratio due to the electromagnetic force induced by the rotating magnets. This apparent damping is almost zero in TS tests when f≤0.1 Hz, while in RC tests it can range between 0.5 and 3.5 % depending on the loading frequency (Stokoe et al. 1994a, 1994c) A calibration of each specific apparatus is required to assess the appropriate correction factors (Stokoe et al. 1994c).
- Frequency (strain rate) dependence of the damping ratio, especially at small strains and in the case of cohesive soils (Stokoe et al. 1994a)

At large strains the influence of N on D becomes the dominant factor. The number of cycles produces a cyclic strain hardening in drained tests while in undrained tests it produces a strain softening due to both pore pressure build-up and structure alteration.

The above data clearly shows that the assumption of a pure hysteretic (frequency independent) nature of damping is just an approximation and that the use of very high frequencies can provide damping values which are overestimated for practical purposes.

A more detailed study on the frequency (strain rate) effects on damping of cohesive soils was performed by Shibuya et al. (1994) (see also Toki et al. 1994). A qualitative picture of the findings of Shibuya et al. (1994) is reported in figure 22 where the damping values are plotted against frequency for various strain levels. The increase in damping for decreasing frequency below 0.1 Hz is considered a consequence of creep. Constant values of damping for f=0.1-10 Hz indicate that soil non-linearity is the main cause of energy dissipation in this case. Damping increases linearly with frequency according to the Kelvin-Voigt model for frequencies greater than 10 Hz.

The influence of strain rate on damping of various geomaterials is reported by Tatsuoka et al. (1995). These findings are consistent with those shown by Shibuya et al. (1994) and confirm that damping, in the case of soft rocks and clays, depends on the following factors even at small or very small strain levels:

- Creep, which leads to increasing values of damping ratio for decreasing strain rate
- Drainage conditions. The occurrence of a partial drainage produces energy dissipation due to the friction between the pore fluid and soil skeleton. Energy dissipation increases with an increase of the strain rate.
- Pore fluid viscosity including, in case of fine grained soils, the absorbed water layers, which becomes relevant only for strain rates larger than a certain value. This of course involves increasing damping values for an increase in the strain rate.

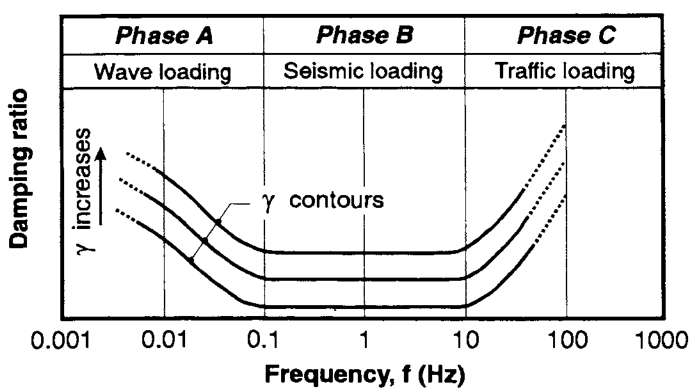

Figure 22 Effects of loading frequency on damping ratio for cohesive soils (Shibuya et al, 1994).

In conclusion the nature of soil damping in soils can be linked to the following phoenomena (fig. 2):

- Non-linearity which governs the so called hysteretic damping controlled by the current shear strain level. This kind of material damping is absent in Zone 1 and has a negligible importance in Zone 2 (see Fig. 1).
- Viscosity of the soil skeleton which is relevant at very small strain rates.
- Viscosity of the pore fluid which is relevant at very high frequencies.

Soil damping within Zone 1 and 2 is mainly due to the viscosity of the soil skeleton or of the pore fluid depending on the strain rates or frequencies.

However, it is worthwhile to remember that, in all cases, the computation of the damping and its use in practice is based on the implicit assumption of visco-elastic linear behaviour (Roesset 1989).

As already mentioned, many attempts to measure small strain damping in situ have been recently made by various researchers. Table 4 summarises the data obtained by means of different methods of interpretation and different kinds of tests. It is possible to observe that the small strain damping reported in Table 4 is usually quite large. It is believed that this is a consequence of the large strain rate (frequency) that usually occurs in seismic tests. Small strain damping obtained by Konno et al. (1991) for gravel deposits are quite small and in good agreement to those determined in the laboratory on high quality samples by means of cyclic quasi static tests. The agreement between in-situ and laboratory damping values is also good at larger strain levels. The above examples confirm the necessity of using an appropriate shearing rate (frequency) when assessing D. Infact, the values of D reported in Table 4, with exception of data due to Konno et al. (1991), appear too high and not suitable for seismic response analysis.

The random decrement technique was used by Yang et al. (1989) and Huerta et al. (1994), among others, to obtain damping values from earthquake records. The method tries to eliminate the random component from the records maintaining only the free vibration curve which can be interpreted in terms of logarithmic decrement, thus providing the damping ratio and the corresponding frequency of the predominant mode. This kind of approach, together with the back-analysis of seismic array records, represent the most promising way of obtaining large strain soil parameters (G, D) directly in situ.

6. UNDRAINED SHEAR STRENGTH OF COARSE GRAINED SOILS

The undrained shear strength controls the stability of liquefiable sand deposits and earth structures after the triggering mechanisms, i.e. vibrations, earthquakes, impact loading, etc. have been activated. In the presence of these kinds of short duration dynamic loading, a saturated soil is sheared under undrained conditions (e=constant) reaching the critical or steady state at large deformations. At the critical or steady

TABLE 4 Small Strain Damping from In-Situ Tests

Reference	Soil	D [%]	Test	Method	Site	f [Hz]
Boore (1993)	Quaternary alluvium	4 to 6	DH	Spectral Slope	Gilroy No. 2 (California)	10 to 60
Liu et al. (1993)	Clayey silt	3	DH	Rise Time	S. Francisco Bay Mud (California)	N.A.
Stewart and Campanella (1991)	Sand Silt	2 to 3 0.3 to 0.5	Seismic cone (DH)	Spectral Slope		40-100
Mancuso (1994)	medium plasticity clay	3 to 9	CH	Spectral Ratio	Bilancino Dam (Italy)	> 100
Abiss and Viggiani (1994)	London clay	4	Stationary Surface wave	Logaritm decrement	Canons Park (UK)	N.A.
Khwaja (1993)	Reconstituted mortar sand	1 to 2.5	CC seismic arrays	Spectral ratio	Large Triaxial Calibration Chamber (Texas)	1000 to 4000
Fuhriman (1993)	Quaternary alluvium	2 to 7	CH	Spectral ratio Spectral slope	Gilroy No. 2 (California)	70 to 350
Konno et al. (1991)	Diluvial gravel	0 to 2	In situ soil column	Back analysis	Tadotsu Engineering Laboratory (Japan)	0.1 to 8
Mok (1987)	Mortar Sand	3 to 5	CH (P waves)	Spectral ratio	Large Triaxial Calibration Chamber (Texas)	2000 to 3000
Mok (1987)	clay	4 to 7	CH	Spectral ratio	O'Neill Forebey Dam	10 to 300

state, the soil deforms at constant volume, effective confining stress, shear stress and strain rate (Casagrande 1936, 1976, Schofield and Wroth 1968, Poulos et al. 1985). The undrained strength that can be mobilised at the critical state can be very low or even zero, depending on the initial state of the deposit, i.e its geological age, void ratio and mean effective stress. In geotechnical literature this strength is usually indicated as s_{us} and is named "residual" or "steady state" shear strength. It is especially relevant for the estimate of the stability of loose coarse grained soils under undrained loading conditions.

The theoretical essence and the experimental aspects of the undrained shear strength of coarse grained soils can be found in works by Poulos et al. (1985) and Ishihara (1993b). However, it is worthwhile to qualitatively summarise the behaviour of a sand during an undrained shear tests, defining the meaning of s_{us} (fig. 23).

The assessment of the s_{us} should be done, at least in principle, through a series of appropriately programmed laboratory tests on undisturbed samples obtained by means of freezing techniques. However, the cost which this kind of sampling involves and the extreme sensitivity of the laboratory test results to even small sample disturbance render such kind of approach still not feasible for every day practice.

Seed (1986) proposed a relationship between normalised SPT resistance $(N_1)_{60}$ and s_{us} for clean sands, based on 17 well documented case records. This relationship, revised by Seed and Harder (1990) is shown in figure 24. When sands contain fines, the relationship depends on the fine content according to the indication given by Seed (1986) and Seed and Harder (1990). Robertson (1990) and Ishihara (1993b) presented a similar correlation which allows to use the cone resistance normalised with respect to the effective overburden stress q_{c1}. In short, these correlations use the q_{c1} vs. $(N_1)_{60}$ data base such as that developed by Robertson and Campanella (1985) and are based on the same case records collected by Seed (1986).

A more elaborated approach to assess s_{us}, which combines the in situ measured values of V_s with the critical state parameters Γ and λ_{ss} obtained from laboratory tests on reconstituted samples of sands, have been recently proposed by Fear and Robertson (1994), where:

λ_{ss} = slope of the steady state (SSL) line in $\ln(\sigma'_m)$ vs. void ratio plane

Γ = intercept of the SSL on the void ratio axis at σ'_m=1 kPa.

Fear and Robertson (1994) assume that a unique ratio s_{us} / σ'_m or s_{us} / σ'_v exists for a given sand at a given state parameter ψ (Been and Jefferies 1985). The s_{us} / σ'_v ratio can be used when K_o=constant.

The method allows one to draw the curves relating s_{us} to the shear wave velocity V_{s1} normalised with respect to the effective in situ stresses. By thereafter utilising material specific correlations between V_{s1} and the penetration resistance $[q_{c1},(N_1)_{60}]$, see Yoshida et al. (1988) and Robertson et al. (1992), it is possible to work-out the charts linking the undrained shear strength to q_{c1} or $(N_1)_{60}$.

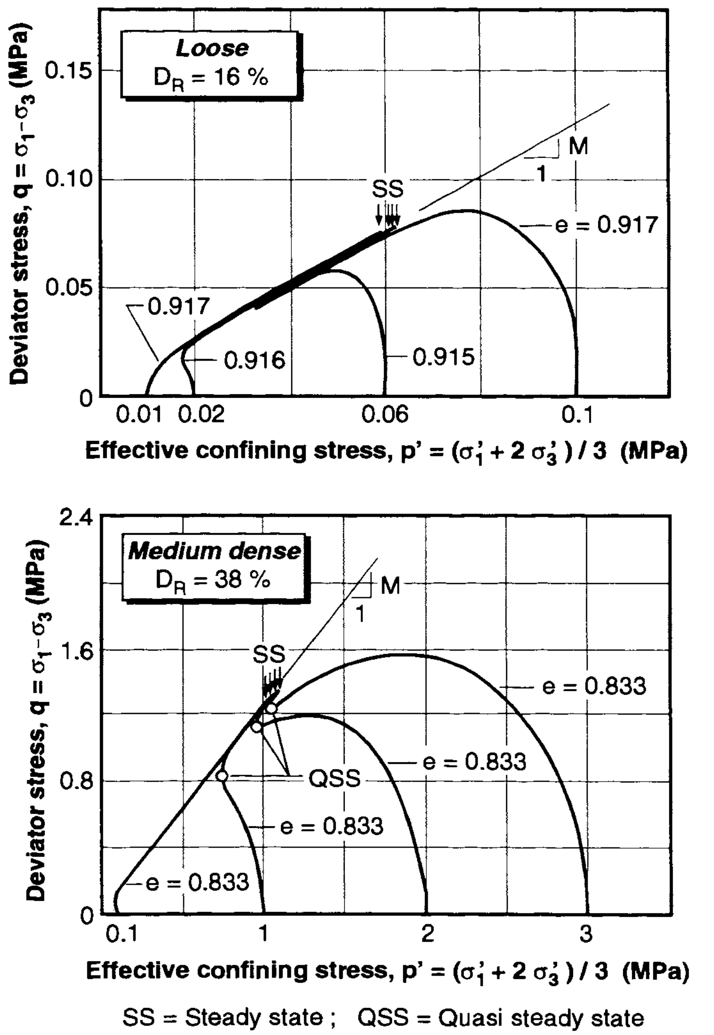

Figure 23 Behaviour of Toyoura sand in undrained triaxial compression tests (Ishihara, 1993).

Fear and Robertson (1994) showed that their method yields s_{us} vs. $(N_1)_{60}$ relationships which are less conservative than that of Seed (1986). This is especially true for more compressible sands characterised by larger values of λ_{ss}. The new approach, although theoretically more sound than those simply relating the s_{us} to $(N_1)_{60}$ or q_{c1} requires further research and first above all field validations. Its accuracy at present may be hampered by problems such as:

- The uniqueness, in a given sand, of the previously mentioned relations between the normalised undrained shear strength and the ψ parameter is questioned, in the case of very loose sand, by Ishihara (1993b).

- The uncertainty of the uniqueness of the SSL and its relevance for the assessment of the s_{us} (Konrad 1990a and 1990b, Ishihara 1993b)

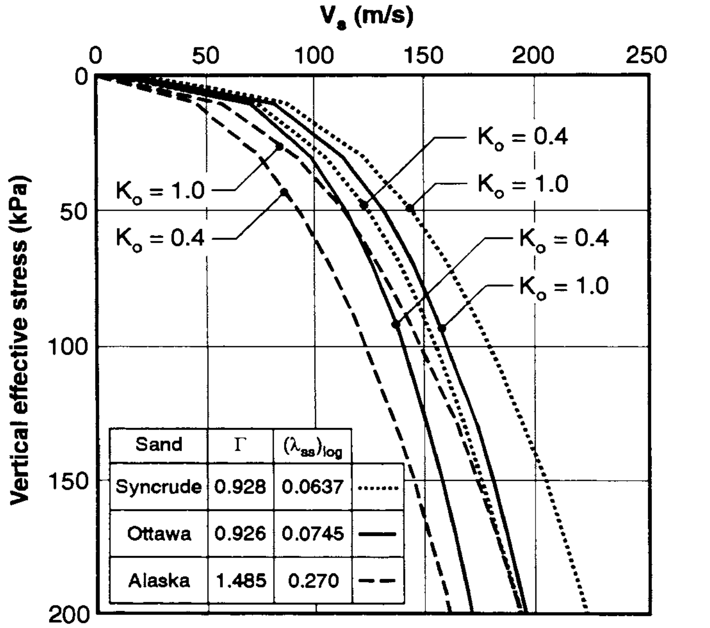

Figure 24 Contractive versus dilative boundary from seismic tests (Cunning et al, 1994).

REFERENCE

Abiss, C.P. and Viggiani, G., (1994), "Surface wave and damping measurements of the ground with a correlator," Proc. XIII ICSMFE, New Delhi, Vol. 3, pp. 1329-1332.

Amar, S., Clarke, B.G.F., Gambin, M.P. and Orr, T.L.L., (1991), "The Application of Pressuremeter Test Results to Foundation Design in Europe," ISSMFE Europian Regional Technical Committee No. 4 - Pressuremeters, Part 1.

Anderson, D.G. and Stokoe, K.H. II, (1978), "Shear Modulus: A Time Dependent Soil Property, Dynamic Geotechnical testing," ASTM STP 654, pp. 66-90.

Andreasson, B.A., (1981), "Dynamic Deformation Characteristics of a Soft Clay," Proc. of International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, Vol. I, pp. 65-70.

Andrus, R.D., (1994), "In Situ Characterization of Gravelly Soils that Liquefied in the 1983 Borah Peak Earthquake," Ph.D. Thesis, The University of Texas at Austin.

Arulmoli, K., Arulanandan, K. and Seed, H.B., (1985), "New Method for Evaluating Liquefaction Potential", Journal of Geotechnical Engineering, Vol. 111, No. 1, pp. 95-114.

Atkinson, J.H. and Sallfors, G., (1991), "Experimental Determination of Stress-Strain-Time Characteristics in Laboratory and in Situ Tests", Proc. of X ECSMFE, Florence, Italy, Balkema, Rotterdam, Vol. 3 pp. 915-956.

Auld, B., (1977), "Cross-Hole and Down-Hole V_s by Mechanical Impulse," Proc. of ASCE, Journal of the Geotechnical Engineering Division, Vol. 103, No. GT12, pp. 1381-1398.

Baldi, G., Bellotti, R., Ghionna, V., Jamiolkowski, M. and Pasqualini, E., (1986), "Interpretation of CPTs and CPTUs - Part II: Drained Penetration in Sands," Proc. of 4th International Geotechnical Seminar on Field Instrumentation and In Situ Measurements, Singapore.

Baldi G., Bruzzi, D., Superbo, S., Battaglio, M. and Jamiolkowski, M., (1988), "Seismic Cone in Po River Sand," Proc. of 1st IS on Penetration Testing/ISOPT-1, Orlando, Florida, Vol. 2, pp. 643-650.

Baldi, G., Bellotti, R., Ghionna, V.N., Jamiolkowski, M. and Lo Presti, D.C.F., (1989a), "Modulus of Sands from CPT's and DMT's," Proc. XII ICSMFE, Rio de Janeiro, Vol.1, pp. 165-170.

Baldi, G., Jamiolkowski, M., Lo Presti, D.C.F., Manfredini, G. and Rix, G.J., (1989b), "Italian Experience in Assessing Shear Wave Velocity from CPT and SPT," Earthquake Geotechnical Engineering, Proc. of Discussion Session on Influence of Local Conditions on Seismic Response, XII ICSMFE, Rio de Janeiro, Brasil. pp 157-168

Baligh, M.M., Azzouz, A.S., Wissa, A.Z.E., Martin, R.T. and Morrison, M.J. (1981), "The Piezocone Penetrometer," Proc. of Symposium on Cone Penetration Testing and Experience, ASCE National Convention, St Louis, Missouri.

Baziw, E.J., (1993), "Digital Filtering Techniques for Interpreting Seismic Cone Data," Journal of Geotechnical Engineering, ASCE, Vol. 119, No. 6, pp. 998-1018.

Been, K. and Jefferies, M.G., (1985), "A State Parameter for Sands," Géotechnique, No. 2.

Been, K., Crooks, J.H.A., Becker, D.A: and Jefferies, M.G., (1986), "The Cone Penetration Test in Sands: Part I, State Parameter and Interpretation," Géotechnique, No. 2.

Bellotti, R., Ghionna, V.N., Jamiolkowski, M., Lancellotta, R. and Manfredini, G., (1986), "Deformation Characteristics of Cohesionless Soils from in Situ Tests," Use of In Situ Tests in Geotechnical Engineering, Geotechnical Special Pubblication, No. 6, S.P. Clemence, Ed., ASCE, pp. 47-73.

Bellotti, R., Ghionna, V.N., Jamiolkowski, M., Robertson, P.K. and Peterson, R.W., (1989), "Interpretation of Moduli from Self-Boring Tests in Sand," Géotechnique 39, No. 2, pp. 269-292.

Bellotti, R., Benoit, J. and Morabito, P., (1994), "A Self-Boring Electrical Resistivity Probe for Sands," Proc. of XIII ICSMFE, New Delhi, India, pp. 313-316.

Bellotti, R., Jamiolkowski, M., Lo Presti, D.C.F. and O'Neill, D.A., (1995), "Anisotropy of Small Strain Stiffness in Ticino Sand," Géotechnique, (in press).

Bolt, B.A., Tsai, Y.B., Yeh, K. and Hsu, M.K., (1982), "Earthquake Strong Motions Recorded by a Large Near-Source Array of Digital Seismographs," Earthquake Engineering and Structural Dynamics, Vol. 10, pp. 561-573.

Boore, D.M., (1993), " Some Notes Concerning the Determination of Shear-Wave Velocity and Attenuation," Geophysical Techniques for Site and Material Characterization sponsored by the National Science Foundation, Atlanta, Georgia.

Bratton, J.L. and Higgins, C.J., (1978), "Measuring Dynamic In Situ Geotechnical Properties," Proc. of the ASCE Geotechnical Engineering Division Specialty Conference Earthquake Engineering and Soil Dynamics, Pasadena, CA, Vol. I, pp. 272-289.

Briaud, J.-L., Lytton, R.L. and Hung, J.T., (1983), "Obtaining Moduli from Cyclic Pressuremeter Tests," Journal of Geotechnical Engineering ASCE, Vol. 109, No. 5, pp. 657-665.

Burghignoli, A., Pane, V., Cavalera, L., Sagaseta, C., Cuellar, V. and Pastor, M., (1991), "Modelling Stress-Strain-Time Behaviour of Natural Soils," Proc. of X ECSMFE, Florence, Italy, Balkema, Rotterdam, Vol. 3, pp. 959-979.

Byrne, P.M., Salgado, F. and Howie, J.A., (1991), "G_{max} from Pressuremeter Tests: Theory; Chamber Tests; and Field Measurements," Proc. of Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, Vol. I, pp. 57-63.

Campanella, R.G. and Robertson, P.K., (1981), "Applied Cone Research," Dept. of Civil Engineering, University of British Columbia, Vancouver, Soil Mechanics Series, N. 46.

Campanella, R.G. and Robertson, P.K., (1984), "A Seismic Cone Penetrometer to Measure Engineering Properties of Soil," Society Of Exploration Geophisics, Atlanta, Ga.

Campanella, R.G. and Weemes, I., (1990), "Development and Use of an Electrical Resistivity Cone for Groundwater Contamination Studies," University of British Columbia, Vancouver, Soil Mechanics Series No. 140.

Campanella, R.G. and Stewart, W.P., (1990), "Seismic Cone Analysis Using Digital Signal Processing for Dynamic Site Characterization," 43rd Canadian Geotechnical Engineering Conference, Quebec City.

Casagrande, A., (1936), "Characteristcs of Cohesionless Soils Affecting the Stability of Slopes and Earth Fills," Journal of Boston Society of Civil Engineers, January, pp. 257-276.

Casagrande, A., (1976), "Liquefaction and Cyclic Deformation of Sands - A Critical Review," Harvad Soil Mechanics, Series No. 8, Harvard Univ., Cambridge, Mass.

Chang, C.Y., Mok, C.M., Power, M.S., Tang, Y.K., Tang, H.T. and Stepp, J.C., (1991), "Development of Shear Modulus Reduction Curves Based on Lotung Downhole Ground Motion Data," Proc. 2nd ICRAGEESD, S. Louis, Paper 1.44.

Chung, R.M., Yokel, F.Y. and Wechsler, H., (1984), "Pore Pressure Buildup in Resonant Column Tests," Journal of Geotechnical Engineering, ASCE, Vol. 110, No. 2, pp. 247-261.

Clark, B.G., (1994), "Pressuremeters in Geotechnical Design," Blackie Academic and Professional Edit., London.

Crova, R., Jamiolkowski, M., Lancellotta, R. and Lo Presti, D.C.F., (1992), "Geotechnical Characterization of Gravelly Soils at Messina Site," Selected Topics, Proceedings of the Wroth Memorial Symposium, Thomas Telford, London, pp 199-218.

Cunning, J.C., Robertson, P.K. and Sego, D.C., (1994), "Shear Wave Velocity to Evaluate In-Situ State of Cohesionless Soils," Canadian Geotechnical Journal, submitted.

De Ruiter, J., (1981), "Current Penetrometer Practice," Proc. Symposium on Cone Penetration Testing and Experience, ASCE National Convention, St Louis, Missouri.

Dobry, R., Ladd, R.S., Yokel, F.Y., Chung, R.M. and Powell, D., (1982), "Prediction of Pore Water Pressure Build-up and Liquefaction of Sands During Earthquake by the Cyclic Strain Method," National Bureau of Standards Building, Science Series 138, Washington, D.C.

Fahey, M., (1991), "Measuring Shear Modulus in Sand with the Self-Boring Pressumeter", Proc. of X ECSMFE, Florence, Italy, Balkema, Rotterdam, Vol. 1, pp. 73-76.

Fahey, M. and Carter, J.P., (1993), "A Finite Element Study of the Pressuremeter Test in Sand Using a Non-Linear Elastic Plastic Model," Canadian Geotechnical Journal, Vol. 30.

Fear, C.E. and Robertson, P.K., (1994), "Estimation of Ultimate Undrained Steady State Shear Strength of Sand Using Shear Wave Velocity Measurements," Geotechnical Group, Department of Civil Engineering, Univ. of Alberta, Edmontore, Draft.

Fretti, C., Lo Presti, D.C.F. and Salgado, R., (1993), "The Research Dilatometer: in Situ and Calibration Chamber Test Results," Rivista Italiana di Geotecnica, Vol. XXVI, No. 4, pp. 237-243.

Fuhriman, M.D., (1993), "Cross-Hole Seismic Tests at two Northern California Sites Affected by the 1989 Loma Prieta Earthquake," M. Sc. Thesis, The University of Texas at Austin.

Gauer, R.C., (1990), "Experimental Study of Applying the Spectral-Analysis-of-Surface-Waves Method Offshore," M. Sc. Thesis, The University of Texas at Austin.

Gens, A., (1982), "Stress-Strain and Strength Characteristics of a Low Plasticity Clay," Ph.D. Thesis, Imperial College, London.

Gens, A., (1985), "A State Boundary Surface for Soils not Obeying Rendulic's Priciple," Proc. X ICSMFE, San Francisco.

Ghionna, V., Karim, M. and Pedroni, M., (1994), "Interpretation of Unload-Reload Modulus from Pressuremeter Tests in Sand," Proc. of XIII ICSMFE New Delhi, India, pp. 115-120.

Goto, S., Shamoto, Y. and Tamaoki, K., (1987), "Dynamic Properties of Undisturbed Gravel Sample by In-Situ Frozen," Proc. of 8th ARCSMFE, Kyoto, Vol. 1, pp. 233-236.
Goto, S., Suzuki, Y., Nishio, S. and Oh Oka, H., (1992), "Mechanical Properties of Undisturbed Tone-River Gravel obtained by In-Situ Freezing Method," Soils and Foundations No. 3, pp. 15-25.
Hall, J.R., (1985), "Limits on Dynamic Measurements and Instrumentation," Richart Commemorative Lectures, ASCE, pp 108-119.
Harder, L.F. and Seed, H.B., (1986), "Determination of Penetration Resistance for Coarse-Grained Soils Using the Becker Hammer Drill," Report No. UCB/EERC-86/06, University of California, Berkeley, Ca.
Hardin, B.O., (1978), "The Nature of Stress-Strain Behaviour of Soils," Earthquake Engineering and Soil Dynamics, Pasadena CA, Vol. 1, pp 3-90.
Hardin, B. O. and Blandford, G. E., (1989), "Elasticity of Particulate Materials," Journal of Geotechnical Engineering, ASCE, Vol. 115, No. 6.
Hatanaka, M., Suzuki, Y., Kawasaki, T. and Endo, M., (1988), "Cyclic Undrained Shear Properties of High Quality Undisturbed Tokyo Gravel," Soil and Foundations, Vol. 28, No. 4, pp. 57-68.
Hatanaka, M., Suzuki, Y., Kawasaki, T. and Endo, M., (1989), "Dynamic Properties of Undisturbed Tokyo Gravel," Proc. of XII ICSMFE, Rio de Janeiro.
Henke, W. and Henke, R., (1991), "In Situ Torsional Cylindrical Shear Test-Laboratory Results," Proc. 2nd ICRAGEESD, S. Louis.
Henke, W. and Henke, R., (1992), "Protoype Cyclic Torsional Cylindrical Shear Tests," Proc. 10th World Conference on Earthquake Engineering, Madrid.
Henke, W. and Henke, R., (1993), "Laboratory Evaluation of In-Situ Geotechnical Torsional Cylindrical Impulse Shear Test for Earthquake Resistant Design," Bulletin of the Seismological Society of America, Vol. 83, No. 1, pp 245-263.
Henke, W. and Henke, R., (1994), "Simplified in Situ Torsional Cylindrical Impulse Shear Test," Proc. of the Second International Conference on Earthquake Resistant Construction and Design, Berlin.
Hryciw, R.D., (1989), "Ray-Path Curvature in Shallow Seismic Investigations," Journal of Geotechnical Engineering, ASCE, Vol. 115, No. 9, pp. 1268-1284.
Huerta, C.I., Acosta, J., Roesset, J.M. and Stokoe, K.H. II, (1994), "In Situ Determination of Soil Damping from Earthquake Records," Proc. of ERCAD Berlin 1994, Balkema, Vol. 1, pp. 227-234
Hughes, J.M.O. and Robertson, P.K., (1985), "Full Displacement Pressuremeters Testing in Sands," Canadian Geotechnical Journal, No. 3.
Hvorslev, M.J., (1937), "On the Strength Properties of Remoulded Cohesive Soils," Thesis, Danmarks Naturvidenskabelige Samfund, Ingeniorvidenskabelige Skrifter, Copenhagen.
Imai, T. and Tonouchi, K., (1982), "Correlations of N-Values with S-Wave Velocity," Proc. 2nd European Symposium on Penetration Testing, Amsterdam, Vol. 2, pp. 67-72.
Isenhower, W.M. and Stokoe, K.H. II, (1981), "Strain Rate Dependent Shear Modulus of San Francisco Bay Mud," International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri.
Ishihara, K., (1993a), "Dinamic Properties of Soils and Gravels From Laboratory Tests," Proc. of the Seminar on Soil Dynamics and Geotechnical Earthquake Engineering, Lisboa.
Ishihara, K., (1993b), "Liquefaction and Flow Failure During Earthquakes," Géotechnique, Vol. 43, No. 3, pp. 351-415.
ISMES, (1993), "Sviluppo e Sperimentazione di Attrezzature per indagini Geofisiche e di Tecniche per Indagini non Distruttive e Applicazioni in Situ - Controlli Sonici non Distruttivi (Radar, Termografia, Onde di Taglio)," Report, Prog. DGF-6761, Doc. n. RAT-DGF-004.
Jamiolkowski, M., Ladd, C.C., Germaine, J.T. and Lancellotta, R., (1985), "New Developments in Field and Laboratory Testing of Soils," Theme Lecture, Proc. XI ICSMFE, San Francisco, CA.
Jamiolkowski, M., Ghionna, V.N., Lancellotta, R. and Pasqualini, E., (1988), "New Correlations of Penetration Test in Design Practice," Proc. of ISOPT-I, Orlando, Florida.
Jamiolkowski, M., Leroueil, S. and Lo Presti, D.C.F., (1991), "Design Parameters from Theory to Practice", Proc. Geo-Coast '91, Yokohama, Japan.
Jamiolkowski, M. and Lo Presti, D.C.F., (1994), "In Situ Testing and Real Soil Behaviour" text of the Panel Discussion to be presented at the Plenary Session A-Soil Properties, Proc. XIII ICSMFE, New Dehli.
Jamiolkowski, M., Lancellotta, R. and Lo Presti, D.C.F., (1994), "Remarks on the Stiffness at Small Strains of Six Italian Clays," Theme lecture session 1a, IS Hokkaido, Pre-print volume, pp. 95-114.
Jardine, R.J., (1985), "Investigations of Pile-Soil Behaviour with Special Reference to the Foundations of Offshore Structures," Ph.D. Thesis, University of London, London.
Jardine, R.J., St John, H.D., Hight, D.W. and Potts, D.M., (1991), "Some Pratical Applications of a Non-Linear Ground Model," Proc. of X ECSMFE, Florence, Vol. 1, pp. 223-228.
Jardine, R.J., (1992a), "Some Observations on the Kinematic Nature of Soil Stiffness," Soils and Foundations, Vol. 32, pp. 111-124
Jardine, R.J., (1992b), "Non Linear Stiffness Parameters from Undrained Pressuremeter Tests," Canadian Geotechnical Journal, No. 29, pp. 436-447.
Jardine, R.J., (1994), "One Perspective of the Pre-Failure Deformation Characteristics of Some Geomaterial," Keynote Lecture 5, IS Hokkaido, Pre-print volume, pp. 151-182.
Jezequel, J.F., Lamy, J.L. and Perrier, M., (1982), "The LPC-TLM Pressio-Penetrometer," Proc. of Symposium on Pressuremeter and its Marine Applications, Paris.
Katayama, T., Fukui, F., Goto, M., Makihara, Y. and Tokimatsu, K., (1986), "Comparison of Dynamic Deformation Characteristics of Dense Sand between Undisturbed Samples," Proc. of 21st Annual Conference of JSSMFE, pp. 583-584, (in Japanese).
Katayama, T., Yamazaki, F., Nagata, S., Lu, L. and Türker, T., (1990a), "Development of Strong Motion Database for the Chiba Seismometer Array," Earthquake Disaster Mitigation Engineering, Institute of Industrial Science, University of Tokyo.
Katayama, T., Yamazaki, F., Nagata, S., Lu, L. and Türker, T., (1990b), "A Strong Motion Database for the Chiba Seismometer Array and its Engineering Analysis," Earthquake Engineering and Structural Dynamics, Vol. 19, pp. 1089-1106.
Khawaja, A.S., (1993), "Damping Ratios from Compression and Shear Wave Measurements in the Large Scale Triaxial Chamber," M. Sc Thesis, The University of Texas at Austin.
Kim, T.C. and Novak, M., (1981), "Dynamic Properties of some Cohesive Soils of Ontario," Canadian Geotechnical Journal, Vol. 18 pp. 1743-1761.
Kohata, Y., Teachavorasinskun, S., Suzuki, T., Tatsuoka, F. and Sato. T., (1993), "Effects of Time Lag on Damping in Cyclic Tests," Proc. of 28th Japan National Conference on SMFE, JSSMFE, pp. 887-890, (in Japanese).
Konno, T., Suzuki, Y., Tateishi, A., Ishihara, K., Akino, K. and Iizuka, S., (1991), "Gravelly Soil Properties by Field and Laboratory Tests," Proc. of 3rd International Conference on Case Histories in Geotechnical Engineering, S. Louis, Paper 3.12.
Kovacs, W.D. and Salomone, L.A., (1982), "SPT Hammer Energy Measurements," Journal of Geotechnical Engineering, ASCE, No. GT4.

Lee, S.H. and Stokoe, K.H. II, (1986), "Investigation of Low Amplitude Shear Wave Velocity in Anisotropic Material," Geotechnical Engineering Report GR 86-6, University of Texas at Austin.

Lee, N-K.J., (1993), "Experimental Study of Body Wave Velocities in Sand Under Anisotropic Conditions," Ph.D. Thesis, University of Texas at Austin.

Lefebvre, G., Leboeuf, D., Rahhal, M.E., Lacroix, A., Warde, J. and Stokoe, K.H. II, (1994), "Laboratory and Field Determination of Small Strain Shear Modulus for a Structured Champlain Clay," Canadian Geotechnical Journal, No. 31, pp. 61-70

Lewis, M.D., (1990), "A Laboratory Study of the Effect of Stress State on the Elastic Moduli of Sand", Ph.D. Thesis, University of Texas at Austin.

Li, X.S., Roblee, C.J. and Wang, G., (1993), "Development and Evaluation of a Prototype Tool for In-Situ Determination of High-Strain Properties of Soft to Medium-Stiff Clays. Phase I," Interim Report # FHWA/CA/TL-94/05

Liu, H.-P., Warrick, R.E., Westerlund, R.E. and Kayen, R.E., (1993), "In-Situ Measurement of Seismic Shear-Wave Absorption in the San Farancisco Bay Mud," Geophysical Techniques for Site and Material Characterization sponsored by the National Science Foundation, Atlanta, Georgia.

Loh, C.H., Penzien, J. and Tsai, Y.B., (1982), "Engineering Analysis of SMART 1 Array Accelerograms," Earthquake Engineering and Structural Dynamics, Vol. 10, pp. 575-591.

Lo Presti, D.C.F., (1987), "Mechanical Behaviour of Ticino Sand from Resonant Column Tests," Ph.D. Thesis, Department of Structural Engineering, Politecnico di Torino.

Lo Presti, D.C.F. and Lai, C,. (1989), "Shear Wave Velocity in Soils from Penetration Tests," Atti del Dipartimento di Ingegneria Strutturale, Politecnico di Torino, No. 21, pp. 32.

Lo Presti, D.C.F. and O'Neill, D.A., (1991), "Laboratory Investigation of Small Strain Modulus Anisotropy in Sand," Proc. of ISOCCT1, Postdam, NY.

Lo Presti, D.C.F., Pallara, O., Lancellotta, R., Armandi, M. and Maniscalco, R., (1993), "Monotonic and Cyclic Loading Behaviour of Two Sands at Small Strains," Geotechnical Testing Journal, Vol. 16, No. 4, pp. 409-424.

Lo Presti, D.C.F., (1994), "Measurement of Shear Deformation of Geomaterials from Laboratory Tests," General report session 1a, IS Hokkaido, Pre-print volume, pp. 339-360.

Luke, B.A., Stokoe, K.H. II and Piccoli, S., (1994), "Surface Wave Measurements to Determine In Situ Shear Wave Velocity Profiles: On Land at Pontida and Offshore at Treporti, Italy," Geotechnical Engineering Report GR94-1, Geotechncal Engineering Center, Civil Engineering Department, University of Texas at Austin.

Manassero, M., (1989), "Stress-Strain Relationships from Drained Self-Boring Pressuremeter Tests in Sands," Géotechnique, Vol. 39. No. 2. pp. 293-307.

Mancuso, C., Simonelli, A. and Vinale, F., (1989), "Numerical Analysis of in Situ S-Wave Measurements," Proc. of XII ICSMFE, Rio de Janeiro.

Mancuso, C., (1994), "Damping of Soil by Cross-Hole Method," Proc. of XIII ICSMFE, New Delhi, India, Vol. 3, pp. 1337-1340.

Manesh, M.S., (1991), "Theoretical Investigation of the Spectral-Analysis-of-Surface-Waves (SASW) Thecnique for Application Offshore," Ph.D. Thesis, University of Texas at Austin.

Marchetti, S., (1980), "In Situ Tests by Flat Dilatometer," Journal of Geotechnical Engineering, ASCE, No. GT3.

Mayne, P.W. and Rix, G.J., (1993), "G_{max}-q_c Relationships for Clays," Geotechnical Testing Journal, Vol. 16, No. 1, pp. 54-60.

Mesri, G., (1987), "Fourth Law of Soil Mechanics," Proc. IS on Geotechnical Engineering of Soft Soils, Mexico City, Mexico. Also: Lecture Presented to Japanese Society of Soil Mechanics and Foundation Engineering, Tokyo, November 1988.

Mitchell, J.K., (1978), "In Situ Techniques for Site Characterization," Proc. of Specialty Workshop Site Charcterization and Exploration, ASCE, pp. 107-129.

Mitchell, J.K., Lodge, A.L., Coutinho, R.Q., Kayen, R.E., Seed, R.B., Nishio, S. and Stokoe, K.H. II, (1994), "In Situ Test Results from four Loma Prieta Earthquake Liquefaction Sites: SPT, CPT, DMT and Shear Wave Velocity," EERC Report No. UCB/EERC - 94/04.

Mok, Y.J., (1987), "Analytical and Experimental Studies of Borehole Seismic Methods," Ph.D. Thesis, Univ. of Texas at Austin, Austin, TX.

Mok Y.J., Stokoe, K.H. II and Wilson, C.R., (1989), "Analysis of Downhole Seismic Data Using Inverse Theory," Proc. of 9th World Conference on Earthquake Engineering, Tokyo 1989, Vol III, pp. 65-70.

Mori, H. and Tsuchiya, H., (1981), "In Situ Measurement on Dynamic Modulus and Damping of Pleistocene Soils," Proc. of International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, Vol. II, pp. 581-584.

Mroz, Z., (1967), "On the Description of Anisotropic Work Hardening," Jour. of Mech. Phys. Solids, No. 15, pp. 163-175.

Motan, S.E. and Khan, A.Q., (1988), "In Situ Shear Modulus of Sands by a Flat Plate Penetrometer: a Laboratory Study," Gerotechnical Testing Journal, Vol. XI No. 4, pp. 257-262.

Muir Wood, D., (1990), "Strain Dependent Soil Moduli and Pressuremeter Tests," Géotechnique, No. 40, pp. 509-512.

Muromachi, T., (1981), "Cone Penetration Testing in Japan," Proc. of Symposium on Cone Penetration Testing and Experience, ASCE National Convention, St Louis, Missouri.

Nazarian, S. and Stokoe, K.H. II, (1983), "Use of the Spectral Analysis of Surface Waves for Determination of Moduli and Thickness of Pavement Systems," Transortation Research Recod, No. 954, Washington, D.C.

Nazarian, S. and Stokoe, K.H. II, (1984), "In Situ Shear Wave Velocities from Spectral Analysis of Surface Waves," Proc. of the 8th World Conference on Earthquake Engineering, San Francisco, CA, Vol. III, pp. 31-38.

Nazarian, S. and Stokoe, K.H. II and Hudson, W.R., (1983), "Use of the Spectral Analysis of Surface Waves Method for Determination of Moduli and Thickness of Pavement Systems," Research Recod, No. 930, Transortation Research Board, pp. 38-45.

Nasir, S.H., (1992), "Field Measurements of Changes in Body Wave Velocities Due to Unloading," M. Sc Thesis, University of Texas at Austin.

Nigbor, R.L. and Imai, T., (1994), " The Suspension P-S Velocity Logging Method," Proc. XIII ICSMFE New Delhi, India TC # 10, pp. 57-61.

Noda, S., Kurata, E. and Tsuchida, H., (1988), "Observation of Earthquake Motions by Dense Instrument Arrays at Soft Ground," Proc. of 9th World Conference on Earthquake Engineering, Vol. II, pp. 151-158.

Okubo, T., Arakawa, T. and Kawashima, K., (1984), "Dense Instrument Array Programm of the Public Works Research Institute and Preliminary Analysis of the Records," Proc. of 8th World Conference on Earthquake Engineering, Vol. II, pp. 79-86.

Ohta, Y. and Goto, N., (1978), "Empirical Shear Wave Velocity Equations in Terms of Characteristic Soil Indexes," Earthquake Engineering and Structural Dynamics, Vol. 6.

Pallara O., (1995), Comportamento sforzi-deformazioni di due sabbie soggette a sollecitazioni monotone e cicliche, Ph. D. Thesis, Department of Structural Engineering, Politecnico di Torino.

Palmer, A.C., (1972), "Undrained Plane Strain Expansion of a Cylindrical Cavity in Clay; a Simple Interpretation of the Pressuremeter Test," Géotechnique, No. 22. pp. 451-457.

Pang, D.D-J., (1972), "Resonant Footing Test," Soil Mechanics Series No. 11 University of Kentucky, UKY TRG1-72-CE22, pp. 156.

Papa, V., Silvestri, F. and Vinale, F., (1988), "Analisi delle Proprietà di un Tipico Terreno Piroclastico Mediante Prove di Taglio Semplice," Atti del Convegno del Gruppo Nazionale di Coordinamento per gli Studi di Ingegneria Geotecnica, Monselice, Italy.

Patel, N.S., (1981), "Generation and Attenuation of Seismic Waves In Downhole Testing," Geotechnical Engineering Thesis GT81-1, Department of Civil Engineering, University of Texas at Austin.

Piccoli, S. and Smits, F.P., (1991), "Real Time Evaluation of Shear Wave Velocity During the Seismic Cone Penetration Test," Proc. of 3rd IS on Field Measurements in Geomechanics, Oslo, Balkema, Rotterdam, pp. 159-166.

Poulos, S.J., Castro, G. and France, J.W., (1985), "Liquefaction Evaluation Procedure," Journal of Geotechnical Engineering, Vol. 111, No. 6, pp. 772-792.

Prakash, S. and Puri, V.K., (1981), "Dynamic Properties of Soils from In-Situ Tests," Journal of Geotechnical Engineering, ASCE, Vol. 107, No. GT7, pp. 943-963.

Redpath, B.B., Edwards, R.B., Hale, R.J. and Kintzer, F.C., (1982), "Development of Field Techniques to Measure Damping Values for Near Surface Rocks and Soils," Prepared for NSF Grant No. PFR-7900192.

Redpath, B.B. and Lee, R.C., (1986), "In-Situ Measurements of Shear-Wave Attenuation at a Strong-Motion Recording Site," Prepared for USGS Contract No. 14-08-001-21823.

Richardson, D., (1988), "Investigations of Threshold Effects In Soil Deformation," Ph. D. Thesis, The City University, London.

Rix, G.J., (1984), "Correlation of Elastic Moduli and Cone Penetration Resistance," M. Sc. Thesis, The University of Texas at Austin.

Rix, G.J. and Stokoe, K.H. II, (1991), "Correlation of Initial Tangent Modulus and Cone Penetration Resistance," Proc. of 1st ISOCCT, Postdam, New York.

Robertson, P.K. and Campanella, R.G., (1983), "Interpretation of Cone Penetration Tests - Part I: Sands," Canadian Geotechnical Journal, Vol. 20, No. 4, pp. 718-733.

Robertson, P.K., and Campanella, R.G. (1985), "Liquefaction of Sands Using the CPT," JGED, ASCE, GT3, pp. 384-403

Robertson, P.K., (1990), "Evaluation of Residual Shear Strength of Sands During Liquefaction from Penetration Tests," Proc. of 43rd Canadian Geotechnical Engineering Conference, Quebec City.

Robertson, P.K., Woeller, D.Y., Kokan, M., Hunter, Y., and Luternauer, Y., (1992), Seismic Cone to Evaluate Liquefaction Potential, Proc. 45th Canadian Geotechnical Conference.

Roblee, C.J., (1990), "Development and Evaluation of Thomographic Seismic Imaging Techniques for Characterization of Geotechnical Sites," Ph.D. Thesis, The University of Texas at Austin.

Roesset, J.M., (1989), "Mechanisms for Energy Dissipation," Proc. of NSF/EPRI Workshop on Dynamic Soil Properties and Site Characterization.

Roesler, S.K., (1978), "Correlation Methods in Soil Dynamic," Proc. of Dynamic Method in Soil and Rock Dynamics, Karlrushe, Balkema, Rotterdam, Vol. 1, pp. 309-334.

Roesler, S.K., (1979), "Anisotropic Shear Modulus Due to Stress-Anisotropy," Journal of the Geotechnical Engineering Division, Vol. 105, No. GT 7, pp. 871-880.

Santamarina, J.C., (1994), "An Introduction to Geotomography," Proc. of XIII ICSMFE New Delhi, India TC # 10, pp. 35-43.

Schmertmann, J.H. and Palacios, A., (1979), "Energy Dynamics of SPT," Journal of Geotechnical Engineering, ASCE, No. GT8.

Schofield, A. and Wroth, P., (1968), "Critical State Soil Mechanics," McGraw-Hill, New York.

Seed, H.B., Idriss, I.M. and Arango, I., (1983), "Evaluation of Liquefaction Potential Using Field Performance Data," Journal of Gotechnical Engineering, ASCE, Vol. 109, No. 3.

Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R.M., (1984), "The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," Journal of Gotechnical Engineering, ASCE, No. 12.

Seed, H.B. and De Alba, P., (1986), "Use of SPT and CPT Tests for Evaluating the Liquefaction Resistance of Sands," Proc. of In Situ '86, ASCE, pp. 281-302.

Seed, R.B., (1986), "Design Problems in Soil Liquefaction," JGED, ASCE, No. 8, pp. 827-845.

Seed, R.B. and Harder, F.Yr., (1990), "SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength," Proc. of H. Bolton Seed Memorial, Edit Y.M. Duncan.

Shibuya, S., Mitachi, T., Fukuda, F. and Degoshi, T., (1994), "Strain Rate Effects on Shear Modulus and Damping of Normally Consolidated Clay," Dynamic Geotechnical Testing: Second Volume, ASTM STP 1213 (in Press).

Smith, D.M.A., (1987), "Geotechnical Applications of the Screw Plate Tests, Pert Western Australia," Proc. of VIII CPMSIF - PCSMFE, Cartagena, Columbia, pp. 153-164.

Smith, P.R., (1992), "The Behaviour of Natural High Compressibility Clay with Special Reference to Construction on Soft Ground," Ph.D. Thesis, Imperial College, London.

Smits, F.P., (1982), "Penetration Pore Pressure Measured with Piezometer Cones," Proc. of ESOPT II, Amsterdam.

Stallebrass, S.E., (1990), "Modelling the Effect of Recent Stress History on the Deformation of the Overconsolidated Soils," Ph.D. Thesis, The City University of London.

Stewart, W.P. and Campanella, R.G., (1991), "In Situ Measurement of Damping of Soils," Proc. of Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, Vol. I, pp. 83-92.

Stokoe, K.H. II and Hoar, R.J., (1978), "Variable Affecting In Situ Seismic Measurements," Proc. of the Conference on Earthquake Engineering and Soil Dynamics, ASCE Geotechnical Engineering Division Specialty Conference, Pasadena, CA, Vol. II., pp. 919-939.

Stokoe, K.H. II, (1980), "Field Measurement od Dynamic Soil Properties," Proc. of 2nd ASCE Conference on Civil Engineering and Nuclear Power, Knoxville, Tennessee, Vol. II.

Stokoe, K.H. II, Lee, S.H.H. and Knox, D.P. (1985), "Shear Moduli Measurement Under True Triaxial Stresses", Proc. of Advances in the Art of Testing Soils Under Cyclic Loading Conditions, ASCE Convention, Detroit

Stokoe, K.H. II, Lee, J.N.K. and Lee, S.H.H. (1991), "Characterization of Soil in Calibration Chambers with Seismic Waves", Proc. of ISOCCT1, Postdam, NY

Stokoe K.H. II, Nasir S.H. and Andrus R.D., (1992) "In Situ and Laboratory Measurements of the Dynamic Properties of Cemented Granular Soils: a Case History," Proc. of US/Brazil Geotechnical Workshop, Application of Classical Soil Mechanics to Structured Soils, belo Horizonte Brasil, editor Alberto S. Nieto, University of Illinois at Urbana-Champain

Stokoe, K.H. II, Hwang S.K., Lee, J.N.K. and Andrus R.D., (1994a), "Effects of Various Parameters on the Stiffness and Damping of Soils at Small to Medium Strains," Keynote Lecture 2, IS Hokkaido.

Stokoe, K.H. II, Wright, S.G., Bay, J.A. and Roësset, J.M., (1994b), "Characterization of Geotechnical Sites by Sasw Method," Proc. XIII ICSMFE New Delhi, India TC # 10, pp. 15-25.

Stokoe, K.H. II, Hwang, S.K., Roësset, J.M. and Sun, C.W., (1994c), "Laboratory Measurement of Small Strain Material Damping of Soil Using a Free-Free Resonant Column," Proc. of ERCAD Berlin, Balkema, Vol 1, pp. 195-202.

Sully, J.P., (1991), "Measurements of In Situ Lateral Stress During Full-Displacement Penetration Tests," Ph.D. Thesis, Department of Civil Engineering, The University of British Columbia, Vancouver, Canada.

Suzuki, Y., Hatanaka, M., Konno, T., Ishihara, K. and Akino, K., (1992), "Engineering Properties of Undisturbed Gravel Sample," Proc. 10th World Conference on Earthquake Engineering, Madrid.

Suzuki, Y., Goto, S., Hatanaka, M. and Tokimatsu, K., (1993), "Correlation Between Strengths and Penetration Resistances for Gravelly Soils," Soils and Foundations, Vol. 33, No. 1, pp.92-101.

Sycora, D.W. and Stokoe, K.H. II, (1983), "Correlations of in Situ Measurements in Sands with Shear Wave Velocity," Geotechnical Engineering Report GR83-33, University of Texas at Austin, Austin, Texas.

Tallin, A.G. and Santamarina, J.C., (1990), "Geotomography in Site Investigation: Simulation Study," Geotechnical Testing Journal, Vol. 13, No, 2, pp. 129-133.

Tang, H.T., (1987), "Large-Scale Soil-Structure Interaction," Electric Power Research Institute, Report EPRI NP-5513-SR.

Tatsuoka, F. and Shibuya, S.,1992,"Deformation Characteristics of Soil and Rocks from Field and Laboratory Tests," Keynote Lecture, IX Asian Conference on SMFE, Bangkok, 1991, vol. 2, pp. 101-190.

Tatsuoka, F. and Kohata, Y., (1994), "Stiffness of Hard Soils and Soft Rocks in Engineering Applications," Keynote Lecture 8, IS Hokkaido, Pre-print volume, pp. 227-336.

Tatsuoka, F., Lo Presti, D.C.F. and Kohata, Y., (1995), "Deformation Characteristics of Soils and Soft Rocks Under Monotonic and Cyclic Loads and Their Relations," Proc. of 3rd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, State of Art Paper No. 1.

Teachavorasinskun, S., Shibuya, S., Tatsuoka, F., Kato, H. and Horii; N., (1991), "Stiffness and Damping of Sands in Torsion Shear," Proc. of 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri.

Thomann, G.T. and Hryciw, R.D., (1991), "Seismic Downhole, CPT, and DMT Correlations in Sand," Proc. of 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, Vol. I, pp. 97-101.

Toki, S., Shibuya, S. and Yamashita, S., (1994), "Standardization of Laboratory Test Methods to Determine the Cylcic Deformation Property of Geomaterials in Japan," Keynote Lecture No. 1, IS Hokkaido, Pre-print volume, pp. 47-89.

Tokimatsu, K. and Yoshimi, Y., (1983), "Empirical Correlation of Soil Liquefaction Based on SPT N-Value and Fines Content," Soils and Foundations, Vol. 23, No. 4.

Tumay, M.T., Boggers, R.L. and Acar, Y., (1981), "Subsurface Investigations with Piezocone Penetrometer," Proc. of Symposium on Cone Penetration Testing and Experience, ASCE National Convention, St Louis, Missouri.

Vlasblom, A., (1973), "Density Measrement In Situ and Critical Density," Delft Soil Mechanics Laboratory, Delft.

Vučetic, M., (1994), "Cyclic Threshold Shear Strains in Soils," Journal of Geotechnical Engineering, ASCE, Vol. 120, No. 12, pp. 2208-2228.

Whiters, N.J., Schaap, L.H.J., Kolk, K.J. and Dalton, J.C.P., (1986), "The Development of the Full Displacement Pressuremeter," Proc. of 2nd IS on The Pressuremeter and Its Marine Applications, University of Texas, ASTM STP 950.

Wilson, S.D., Brown, F.R.,Jr and Schwarz, S.D., (1978), "In Situ Determination of Dynamic Soil Properties," Dynamic Geotechnical Testing ASTM, STP 654, pp. 295-317.

Woods R.D. 1978 Measurement of dynamic soil properties. Earthquake Engineering and Soil Dynamics, Pasadena CA, Vol. 1,pp 91-179.

Woods, R.D. and Stokoe, K.H. II, (1985), "Shallow Seismic Exploration in Soil Dynamics," Richart Commemorative Lectures, ASCE, pp 120-156.

Woods, R.D., (1991), "Field and Laboratory Determination of Soil Properties at Low and High Strains," Proc. of 2nd International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, SOA1.

Woods, R.D., (1994), "Borehole Methods in Shallow Seismic Exploration," Proc. of XIII ICSMFE New Delhi, India TC # 10, pp. 91-100.

Wroth, C.P., (1984), "The Interpretation of In-Situ Soil Tests," XXIV Rankine Lecture, Géotechnique, No. 4.

Wroth, C.P., (1988), "Penetration Testing - A More Rigorous Approach to Interpretation," Proc. of ISOPT-I, Orlando, Florida.

Yang, J.C.S.,Qi, G.Z., Pavlin, V. and Durelli, A.J., (1989), "In-Situ Determination of Soil Damping in the Lake Deposit Area of Mexico City," Soil Dynamics and Earthquake Engineering, Vol. 8, No. 1, pp. 43-52.

Yoshida, Y., Esahi, Y., Kokusho, K. and Nishii. Y.. (1984), "Dynamic Moduli and Damping Ratios of Soil Evaluated from Pressuremeter Test," Report 383058 August, CRIEPI Japan, (In Japanese)

Yoshida, Y., (1988), "A Proposal on Application of Penetration Tests on Gravelly Soils," Abiko Research Laboratory, Rep. No. U 87080 (in Japanese).

Yoshida, Y., Kokusho, T. and Motonori, I., (1988), "Empirical Formulas of STP Blow Counts for Gravelly Soils," Proc. of ISOPT-I, Orlando, Florida.

Yu, H.S., Schnaid, F. and Collins, I.F., (1994), "Analysis of Cone PressuremeterTests in Sands," Department of Civil Engineering, The University of Newcastle, New South Wales, Australia, RR No. 105.09.