XIII ICSMFE, 1994, New Delhi, India / XIII CIMSTF, 1994, New Delhi, Inde

VALIDITY OF IN SITU TESTS RELATED TO REAL BEHAVIOUR

VALIDITE DES ESSAIS IN SITU PAR RAPPORT AU COMPORTEMENT REEL

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SYNOPSIS : This presentation, after a brief review of the areas within which in situ testing plays a relevant role in the geotechnical characterization of foundation soils, focuses on the evaluation of Young's stiffness of sand from plate loading and on the attempts at determining the initial in situ horizontal stress, the solution of which, in a satisfactory manner, still represents a problem, in spite of the progresses achieved, in the last two decades, by the experimental soil engineering.

It happens quite frequently that discussions arise about the relative merits of in situ versus laboratory tests. The writers believe that this debate is generally of little practical value, because laboratory and in-situ soil testing are complementary rather than competing methodologies.
Another important point is that progress that is achieved within the two areas of soil testing renders the comparison in-situ and laboratory testing mutable with time.
At present, considering the remarkable progress made in the eighties in the area of laboratory testing and undisturbed sampling, especially in cohesive soils, the most relevant applications of in situ tests are the following:

a. soil profiling and characterization;
b. evaluation of in situ horizontal stress;
c. assessment of hydraulic conductivity;
d. evaluation of the stiffness of cohesionless soils;
e. calculation of settlements and bearing capacity of foundations directly from in situ test results, especially for cohesionless soils.

This list of priority applications appears clearly to be finalized at:

- testing of soil volumes which are appreciably larger than those usually involved in conventional laboratory tests, e.g., a) and c);
- assessment of stress-strain properties of soils in which undisturbed sampling is still very difficult, e.g., d) and e);
- assessment of the spatial variability of soil deposits, e.g., a);
- employment of non destructive techniques, such as geophysical methods, in order to evaluate the soil state variables, e.g., b), c) and d), with special reference to the assessment of soil stiffness at very small strain or the attempts to evaluate the void ratio of cohesionless deposits in situ;
- empirical design of foundations, correlating the in-situ test results directly to the behavior of the relevant prototype, e.g., e).

Due to the constraint in space the writers will only briefly deal with the evaluation of in situ horizontal stress and stiffness of sands via in situ tests.
The assessment of the design stiffness represented by an "operational" value of the secant Young's modulus E_s appropriate for settlement calculations using the formulae of the theory of elasticity can be attempted following the procedure outlined below:

a. The initial shear modulus G_0 is determined from in situ seismic tests, e.g. cross-hole, down-hole, seismic cone, spectral analysis of surface waves, etc.
b. The value of initial Young's modulus E_0 is assessed within the frame of the theory of isotropic elasticity assuming elastic Poisson ratio $0.1 \leq \nu_0 \leq 0.2$, see Jamiolkowski et al. (1994), Fioravante et al. (1994).
c. Thereafter the E_0 is reduced to the desired level of strain or of relative displacement according to an appropriate degradation resulting from laboratory test [Tatsuoka and Shibuya (1992), Vucetic and Dobry (1991)] model tests [Ghionna et al. (1993)] or observed behaviour of prototypes [Burland and Burbridge (1984), Berardi and Lancellotta (1991)].

The above outlined approach is substantiated by the results of recent laboratory tests indicating that in a given granular soil the G_0 or E_0 are uniquely related to the current state represented by a combination of the fabric, of the void ratio e and of the magnitudes of the principal effective stresses but are virtually independent from mechanical overconsolidation [Shibuya et al. (1992), Tatsuoka and Shibuya, (1992), Lo Presti et al. (1993)]. This is well documented for granular silica soils. For crushable carbonate sands the values of G_0 and E_0 seem to be moderately influenced by the overconsolidation ratio OCR [Fioravante et al. (1994)].
In this light, the results of deep plate loading tests PLT's [Ghionna et al. (1993, 1994)] performed in the calibration chamber (CC) [Bellotti et al. (1982, 1988)] in dry Ticino silica sand, see fig.1, are interpreted to obtain an engineering correlation between E_s and E_0 via the relative displacement of the plate s/D, s and D being the plate settlement and the diameter of the plate, respectively. The values of E_0 have been assessed based on G_0 determined from resonant column tests (RCT's) [see Lo Presti (1987)] adopting ν_0=0.15. The main features of the PLT's discussed herein are summarized in Table 1. The values of E_s from PLT's have been computed referring to the following formula of the theory of elasticity:

$$E_s = \frac{qD}{s} \cdot I_s \cdot f\left(\frac{z}{D}\right) \cdot (1-\nu^2)$$

being:

q	=	stress acting on the plate
D	=	plate diameter = 104 mm
I_s	=	dimensionless influence factor, for rigid circular disc laying on the upper limit of the isotropic elastic half-space = 0.79
f (z/D)	=	dimesionless coefficient, function of the plate embeddement = 0.65
ν	=	Poisson ratio = 0.25 adopted here.

The modulus number $K[E_s]$ has been computed as follows:

$$K[E_s] = \frac{E_s}{\left(\dfrac{\sigma'_{vc} + \Delta\sigma'_{vc}}{p_a}\right)^{0.5}}$$

being:

σ'_{vc}	=	vertical consolidation stress applied to the CC specimen
$\Delta\sigma'_{vc}$	=	increase of vertical stress at the depth D/2 under the centerline of the plate
p_a	=	reference stress = 98.1 kPa

while that related to the initial stiffness was obtained as follows:

$$K[E_o] = \frac{E_o}{\left(\dfrac{\sigma'_m}{p_a}\right)^{0.44}}$$

where:

σ'_m	=	mean consolidation stress acting on the CC specimen prior to the start of the PLT.

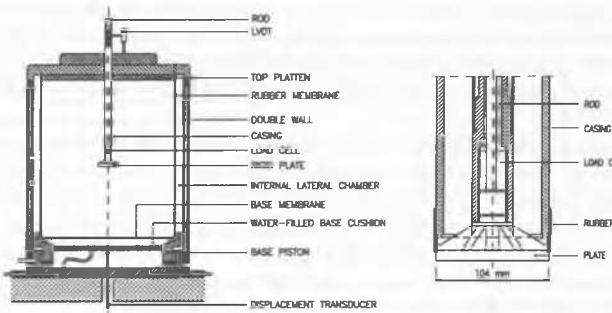

Fig. 1. Plate loading tests in Calibration Chamber.

While the writers are fully awere that for the coherence reasons also E_s should be normalized with respect to the value of the current σ'_m, the current value of σ'_v has been chose for sake of practicality. (semplicity ?).
Figures 2 to 4 show the ratio of the secant modulus number $K[E_s]$ to the initial modulus number $K[E_0]$ as function of s/D. It appears that at least up to s/D ≅ 10% a double logarithmic plot leads to a straight line which describes adequately the decay of the average operational stiffness E_s with increasing s/D. These figures suggest that once G_0 (or E_0) in situ is assessed via seismic tests, a value of E_s at the desired stress level for the relevant value of s/D can be computed.
This very pronounced degradation of stiffness as evidentiated in Figs. 2 through 4 holds for pluvially deposited Ticino sand. As suggested by Ishihara (1993), the degradation of soil stiffness in natural granular deposits might result even more pronounced than that observed in freshly deposited cohesionless soils.
Finally, the numerical analysis of PLT's performed by Bocchio (1993), reported by Ghionna et al. (1993), indicate that within the range of s/D ≤ 10% the experimental results are not influenced by the limited dimensions of the CC specimens (height = 1500 mm, diameter = 1200 mm).
Within this range of the s/D the experimental results can be fitted by the following formulae, valid for $K[E_s]/K[E_o] \leq 1$:
NC. CC specimens : $K[E_s] = 0.045 \quad K[E_o] \quad (s/D\ 100)^{-0.59}$
OC. CC specimens : $K[E_s] = 0.061\ K[E_o] \quad (s/D\ 100)^{-0.66}$

As to the determination of the in situ horizontal stress in cohesionless soils two basic approaches are presently available, Robertson (1986):

a. The <u>direct</u> methods, exemplified by the Self-Boring Pressuremeter tests, (SBPT) for which it is assumed that the insertion of the probe does not cause any appreciable disturbance in the surrounding soil so that the lift-off pressure p_o of the pressuremeter membrane corresponds to the initial in situ total horizontal stress σ_{ho} [Fahey and Randolph (1894), Bruzzi et al (1986)].
However, the experience gained in late eighties [Fahey and Randolph (1984), Jamiolkowski et al. (1985), Bruzzi et al. (1986), Bellotti et al. (1989)] have clearly shown that even small amounts of disturbance and mechanical compliance of the sensors measuring cavity strain can render the assessed values of σ_{ho} not fully reliable. This is especially true in case of the cohesionless soils.

b. The indirect methods are based on the empirical correlations between the large strain parameters measured using different penetration devices and σ_{ho}. Among them, the Marchetti's Flat Dilatometer test (DMT), Marchetti (1980) and the Lateral Stress Cone Penetration test (LSCPT), [Huntsman (1985), Jefferies et al. (1987), Sisson (1990), Campanella et al. (1990)] allow to measure directly the total horizontal stress σ_h acting on the device after penetration.
If the penetration pore pressure u is also measured during the penetration the horizontal effective stress can be assessed.
The evaluation of the initial in situ effective horizontal stress σ'_{ho} is then referred to the amplification factor:

$$AF = \frac{\sigma'_h}{\sigma'_{ho}}$$

In case of the DMT the AF is represented by the ratio of Marchetti's (1980) horizontal stress index K_D to the reference value of the in situ coefficient of the earth pressure at rest K_o.
The AF is obtained via calibration of said devices in the calibration chambers, e.g. Sisson (1990) or on sites where the reference values of σ'_{ho} or K_o have been assessed based on geological information or on SBPT's, e.g. Jamiolkowski et al. (1985), Bruzzi et al. (1986) and Bellotti et al. (1989).
Accumulated experimental evidences show that the AF increases with increasing the density of the cohesionless deposit or more precisely it increases with decreasing its state parameter ψ [Robertson (1986), Been and Jefferies (1985), Been et al., (1986)].
On overall, the reliability of the methods to assess σ'_{ho} or/and K_o from penetration tests results still appears to be very uncertain. This is mainly due to the subjectivety of the reference values against which the results of LSCPT and DMT have been calibrated. In addition, the AF values are substantially higher than one especially in dense sand and largely depend on the volume change characteristics during shearing of a given sand at the given state.

Table 1. Summary of deep plate loading tests performed in Calibration Chamber tests in dry Ticino sand.

TEST N.	BC	D_R (%)	OCR	σ'_{vc} (kPa)	σ'_{hc} (kPa)	G_o (MPa)	M_t (MPa)	$(s/D)_{max}$ (%)	q_{ult} (MPa)	q_b^{crit} (MPa)
307	B-1	93	1	314	122	165	102	9.57	47.8	3.95
308	B-1	93	1	216	86	143	98	10.61	42.8	3.45
317	B-1	54	1	62	24	63	36	11.98	10.8	1.00
318	B-1	58	1	216	94	115	59	18.36	21.5	1.78
319	B-1	57	6.3	65	55	79	95	29.14	15.0	1.42
320	B-1	58	6.5	63	51	78	93	31.65	15.8	1.80
321	B-1	58	1	412	178	154	101	34.74	29.0	2.71
323	B-1	92	6.3	67	65	104	106	57.45	36.9	3.00
324	B-1	91	1	66	28	84	51.2	50.14	29.4	1.90
326	B-3	91	1	65	26	81	50	46.74	29.4	1.89
327	B-2	91	1	65	26	82	54	33.00	29.4	2.04

B−1: σ'_v =CONST., σ'_h =CONST.; B−3: σ'_v =CONST., ε_h =0

B−2: ε_v =0 ; ε_h =0 ; q_b^{crit} = STRESS ON PLATE AT s/D=5%

q_{ult} =ULTIMATE BEARING CAPACITY ACCORDING TO SALGADO (1993)

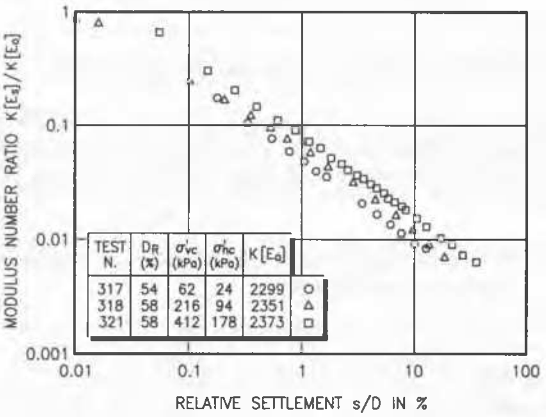

Fig. 2. Modulus number of medium dense NC dry Ticino Sand from deep plate loading tests performed in Calibration Chamber.

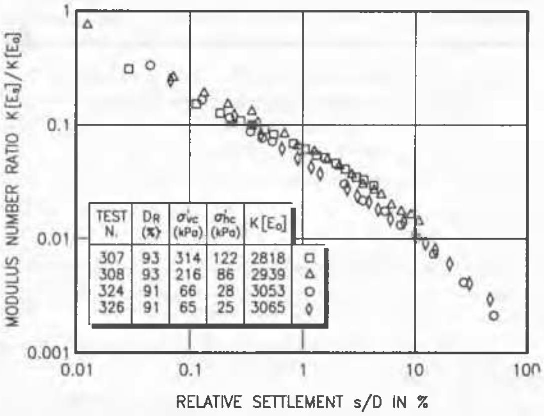

Fig. 3. Modulus number of very dense NC dry Ticino Sand from deep plate loading tests performed in Calibration Chamber.

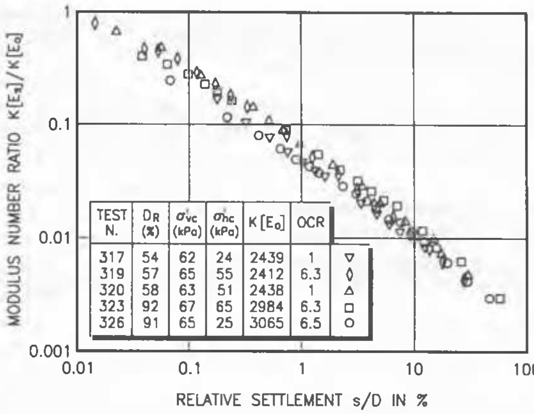

Fig. 4. Modulus number of medium dense NC and OC Ticino Sand from deep plate loading tests performed in Calibration Chamber.

Based on the above it clearly appears that at present the state of the art of evaluation of σ'_{ho} or K_o in cohesionless soils from in situ tests, is far from being satisfactory. Fig.5 shows an example of the evaluation of K_o in the geotechnically well investigated, ligthly overconsolidated, Po river sand based on the results of the DMT's and SBPT's, for further details see; Jamiolkowski et al. (1985), Jamiolkowski and Robertson (1988) and Bellezza (1992). The same figure reports also the σ'_h inferred from three LSCPT's at the same location normalized with respect to the effective overburden stress σ'_{vo}. The measured values of lead to the AF = 1.48 ± 0.25 if the reference σ'_{ho} is inferred from the SBPT's whose results are taken as the best estimate of the existing in situ horizontal stress.

The above results clearly indicate the many existing uncertainties when attempting to evaluate the σ'_{ho} and K_o in sands.

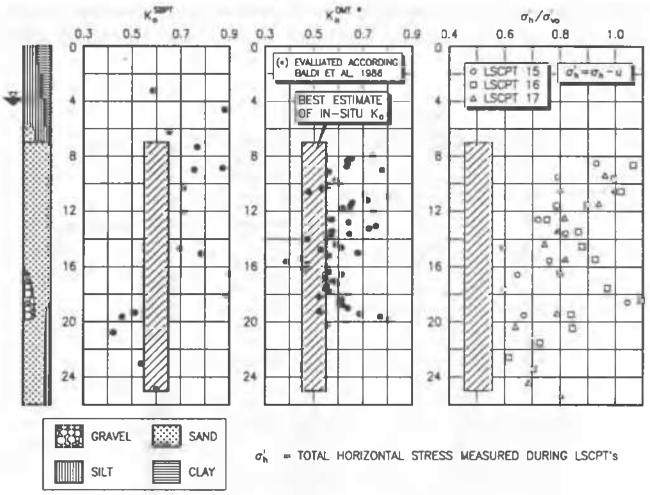

Fig. 5. Coefficient of earth pressure at resp of Po River sand from SBP and DM tests.

The possibility to improve our ability in assessing σ'_{ho} and K_o in cohesionless deposits is, at least in principle, offered by geophysical tests as firstly suggested by Stokoe (1985).

In fact, if one is able to generate and to measure the velocity of the horizontally (V_{hh}^{s}) and the vertically (V_{sv}^{h}) polarized shear waves, the assessment of σ'_{ho} and K_o can be attempted taking into account the following relationships [Roesler (1979), Stokoe et al. (1985, 1991)] referred to the effective in situ stresses:

$$V^s_{hh} = C_{hh}(\sigma'_{ho})^{nh} \cdot (\sigma'_{ho})^{nh}; \quad V^s_{vh} = C_{vh}(\sigma'_{vo})^{nv} \cdot (\sigma'_{ho})^{nh}$$

being:
C_{hh} and C_{vh} = material constants
nh and nv = material exponents
σ'_{vo} = effective overburden stress
assuming typically for sands [Stokoe et al. (1985), Lo Presti and O'Neill (1991)]:

$$C_{hh}/C_{vh} = 1.1; \quad nh = nv = 0.125$$

it is possible to derive the following relationship:

$$K_o = 0.47 \left[\frac{V^s_{hh}}{V^s_{sh}}\right]^8$$

The main problem arising when using in practice this approach is linked to the high exponent to which the ratio of shear wave velocities is raised that requires the utmost accuracy when measuring V_{hh}^s and V_{vh}^s in order to obtain reliable values of K_o. A similar approach can be inferred from the recent work by Hryciw and Thomann (1993).
An example of application of this approach at a site in the central part of Italy where both V_{hh}^s and V_{vh}^s have been measured using cross-hole technique is reported in Fig. 6 and 7. Unfortunately, the complex soil profile and lack of reference values of σ'_{ho} and K_o makes the assessment of the reliability of the approach used quite difficult. A severe scatter of the K_o observed in Fig.7 is probably associated to the inaccuracy of the measured shear wave velocities.
A partial validation of the method can be attempted for cohesive layers of the examined soil profile at the points where the results of oedometer tests, during which the σ'_h have been measured, run on high quality undisturbed samples are available.

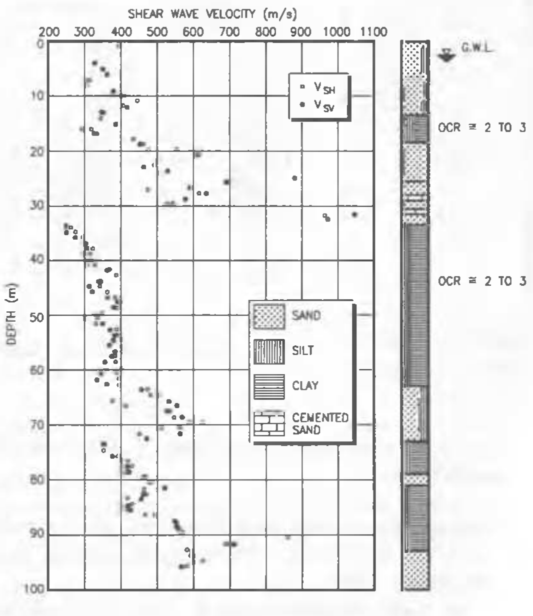

Fig. 6. Horizontally and vertically polarized shear waves

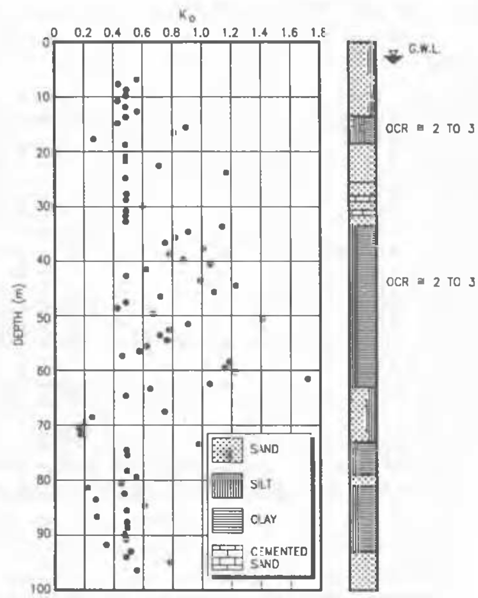

Fig. 7. Coefficient of earth pressure at rest from seismic tests.

Table 2 reports the measured values of V_{hh}^s and V_{vh}^s together with the values of OCR results from oedometer tests. The comparison reported in Table 2 seems to be quite promising and suggests further validation of said approach.

Table 2. Ko values of cohesive layers inferred from shear wave velocities and from oedometer tests.

DEPTH meters below G.L.	OEDOMETER TESTS			SEISMIC TESTS			
	σ'_{vo} (kPa)	OCR	K_o (•)	V^s_{hh} (m/s)	V^s_{vh} (m/s)	V^s_{hh}/V^s_{vh}	K_o
15.85	215	2.09	0.85	316	293	1.08	0.88
34.85	330	2.33	0.63	280	259	1.07	0.82
48.10	440	1.93	0.76	375	357	1.05	0.71
54.82	520	1.91	0.76	410	382	1.06	0.75

(•) $K_o = K_o^{NC} \cdot (OCR)^n = 0.58 \cdot (OCR)^n$, WITH $0.42<n<0.52$
RELATIONSHIP FROM OEDOMETER TESTS ALLOWING TO MEASURE σ_h.

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