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:

- soil profiling and characterization;
- evaluation of in situ horizontal stress;
- assessment of hydraulic conductivity;
- evaluation of the stiffness of cohesionless soils;
- 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 $E_s$ is determined from in situ seismic tests, e.g. cross-hole, down-hole, seismic cone, spectral analysis of surface waves, etc.

The above outlined approach is substantiated by the results of recent laboratory tests indicating that in a given granular soil the $G_o$ or $E_o$ 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 $G_o$ and $E_o$ 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 $E_o$ 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 $E_s$ from PLT’s have been computed referring to the following formula of the theory of elasticity:

$$E_s = \frac{G_0}{s} \cdot I_s \cdot f \left( \frac{Z}{D} \right) \cdot (1-v^2)$$
being:

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

The modulus number \( K[E_J] \) has been computed as follows:

\[
K[E_J] = \frac{E_o}{\left(\frac{\sigma_{vc} + \Delta \sigma_{vc}}{P_o}\right)^{0.5}}
\]

being:

\[ \sigma_{vc} \] = 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
\[ P_o \] = reference stress = 98.1 kPa

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

\[
K[E_o] = \frac{E_o}{\left(\frac{P_o}{\sigma_{vm}}\right)^{0.44}}
\]

where:

\[ \sigma_{vm} \] = mean consolidation stress acting on the CC specimen prior to the start of the PLT.

In case of the DMT the AF is represented by the ratio of Marchetti’s (1980) horizontal stress index \( K_p \) to the reference value of the in situ coefficient of the earth pressure at rest \( K_s \).

\[
AF = \frac{\sigma_h}{\sigma_{ho}}
\]

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

a. The direct 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 \( \sigma_{ho} \) [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 \( \sigma_{ho} \) 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 \( \sigma_{vc} \). 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 \( \sigma_h \) 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 \( \sigma_{ho} \) is then referred to the amplification factor:

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 = 0.10% 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 \( K[E_J]/K[E_o] \leq 1 \):

NC. CC specimens: \( K[E_J] = 0.045 K[E_o] \quad (s/D \leq 100) \)

OC. CC specimens: \( K[E_J] = 0.061 K[E_o] \quad (s/D \leq 100) \)

Fig. 1. Plate loading tests in Calibration Chamber.

While the writers are fully aware that for the coherence reasons also \( E_o \) should be normalized with respect to the value of the current \( \sigma_{vc} \), the current value of \( \sigma_{vc} \) has been chose for sake of practicality. (simplicity ?).

Figures 2 to 4 show the ratio of the secant modulus number \( K[E_J] \) to the initial modulus number \( K[E_o] \) 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 \( E_i \) with increasing s/D. These figures suggest that once \( \sigma_{vc} \) (or \( E_i \)) in situ is assessed via seismic tests, a value of \( E_o \) 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.
Table 1. Summary of deep plate loading tests performed in Calibration Chamber tests in dry Ticino sand.

<table>
<thead>
<tr>
<th>TEST N.</th>
<th>BC</th>
<th>D (X)</th>
<th>OCR</th>
<th>$\sigma_{sc}$ (kPa)</th>
<th>$\sigma_{sc}$ (kPa)</th>
<th>$G_o$ (MPa)</th>
<th>$M_{f}$ (MPa)</th>
<th>(s/D)$_{max}$ (%)</th>
<th>$q_{ult}$ (MPa)</th>
<th>$q_{cbr}$ (MPa)</th>
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<tr>
<td>307</td>
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<td>93</td>
<td>1</td>
<td>314</td>
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<td>10.8</td>
<td>1.00</td>
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<tr>
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<td>65</td>
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<td>82</td>
<td>54</td>
<td>33.00</td>
<td>29.4</td>
<td>2.04</td>
</tr>
</tbody>
</table>

B-1: $\sigma'_v$ =CONST., $\sigma'_h$ =CONST.; B-3: $\sigma'_v$ =CONST., $\sigma'_h$ = 0
B-2: $\varepsilon_v$ = 0; $\varepsilon_h$ = 0; $q_{cbr}$ = 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 $\sigma'_w$ 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, lightly 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 $\sigma'_w$ inferred from three LSCPT's at the same location normalized with respect to the effective overburden stress $\sigma'_w$. The measured values of $\sigma'_w$ lead to the AF $= 1.48 \pm 0.25$ if the reference $\sigma'_w$ 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 $\sigma'_w$ 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 $\sigma'_w$ 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 ($V_h^+$) and the vertically ($V_h^-$) polarized shear waves, the
assessment of $\sigma'_v$ 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:

$$V_{vh}^s = C_{vh}(\sigma '_v)^{nh} \cdot (\sigma _h)^{nh} ; \quad V_{vh}^s = C_{vh}(\sigma '_v)^{nv} \cdot (\sigma _h)^{nh}$$

being:

$C_{vh}$ and $C_{vh}$ = material constants
$nh$ and $nv$ = material exponents
$\sigma '_v$ = effective overburden stress

assuming typically for sands [Stokoe et al. (1985), Lo Presti and O'Neill (1991)]:

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

it is possible to derive the following relationship:

$$K_o = 0.47 \left[ \frac{V_{vh}^s}{V_{vh}} \right]$$

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 $V_{vh}^s$ 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 $V_{vh}^s$ 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 $\sigma'_v$ 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 $\sigma'_v$ have been measured, run on high quality undisturbed samples are available.

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

Table 2 reports the measured values of $V_{vh}^s$ 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. $K_o$ values of cohesive layers inferred from shear wave velocities and from oedometer tests.

<table>
<thead>
<tr>
<th>DEPTH meters below G.L</th>
<th>OEDOMETER TESTS</th>
<th>SEISMIC TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma'_v$ (kPa)</td>
<td>OCR</td>
</tr>
<tr>
<td>15.85</td>
<td>215</td>
<td>2.09</td>
</tr>
<tr>
<td>34.85</td>
<td>330</td>
<td>2.33</td>
</tr>
<tr>
<td>48.10</td>
<td>440</td>
<td>1.93</td>
</tr>
<tr>
<td>54.82</td>
<td>520</td>
<td>1.91</td>
</tr>
</tbody>
</table>

$(*) K_o = K_o^OOCR^(OCR)^n$ with $0.42<n<0.52$ relationship from oedometer tests allowing to measure $\sigma'_v$.

REFERENCES


