SYNOPSIS: Based on the results of a large number of in situ tests performed in a calibration chamber and in the field, the reliability of empirical correlations between the results CPT's and DMT's and the stiffness of predominantly silica sands are discussed. A tentative correlation between in situ test results and maximum shear modulus $G_0$ of cohesionless soils is proposed.

1 INTRODUCTION

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

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

- Working-out empirical correlations between a reference deformation modulus (RDM), measured in laboratory usually on reconstituted soil specimens and the result of a penetration test performed either in the field or in laboratory (De Beer, 1948; D'Appolonia et al., 1966; Schmertmann et al., 1978; Ohta and Goto, 1978).

- Attempting to correlate directly the observed settlements of constructions or large-scale prototypes against the results of a preselected penetration test. This method allows a direct assessment of the settlement of a given foundation (e.g.: Burland & Burbridge, 1985), or the evaluation of an average value of the RDM (e.g.: Schmertmann, 1970; Schmertmann et al., 1978; D'Appolonia et al., 1968).

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

- Numerous electrical CPT's (quasi static cone penetration tests) and DMT's (Marchetti dilatometer tests) performed in pluvially deposited Ticino (TS) and Hokksund (HS) predominantly silica sands (Figure 1), using large calibration chambers (CC) (see Belotti et al. 1982; 1988; Baldi et al., 1986, 1986a).

- A large number of conventional laboratory tests performed on pluvially deposited specimens of the same test sands.

- A large number of in situ and laboratory tests performed in the quaternary deposits of the predominantly silica Po river sand and Giola Tauro sand with gravel (see Figure 1).

![Figure 1. Granulometric characteristics of tested materials.](image)

2 FACTORS ON WHICH ENGINEERING CORRELATIONS DEPEND

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

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

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

<table>
<thead>
<tr>
<th>Factors</th>
<th>Soil Response</th>
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<tbody>
<tr>
<td>Material</td>
<td>Compressibility</td>
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<tr>
<td></td>
<td>Crushability</td>
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<tr>
<td>Grading</td>
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<tr>
<td>Mineralogy</td>
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<tr>
<td>Angularity</td>
<td></td>
</tr>
<tr>
<td>Fabric</td>
<td>Compressibility</td>
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<tr>
<td></td>
<td>Anisotropy</td>
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<tr>
<td>Grain Arrangement</td>
<td></td>
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<tr>
<td>Orientation</td>
<td></td>
</tr>
<tr>
<td>Stress</td>
<td>Compressibility</td>
</tr>
<tr>
<td>Stress-strain</td>
<td>Anisotropy</td>
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<tr>
<td>history</td>
<td></td>
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<tr>
<td>Mean effective</td>
<td></td>
</tr>
<tr>
<td>stress</td>
<td></td>
</tr>
<tr>
<td>Shear strain</td>
<td>Plastic Hardening</td>
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<tr>
<td>Drainage</td>
<td></td>
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<tr>
<td>Undrained</td>
<td>Excess Pore Pressure</td>
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<tr>
<td></td>
<td>Volumetric strain</td>
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<tr>
<td>Time</td>
<td></td>
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<td></td>
<td>Creep, Rate Effects</td>
</tr>
</tbody>
</table>

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

These special tests have been devised to investigate the influence of mechanical overconsolidation (OC) and cyclic pre stressing on the drained Young modulus $E'$ and the results of CPT and DMTs in TS.

During mechanical overconsolidation the sand specimen experiences both plastic hardening, due to accumulated irrecoverable strain, and an increase of the coefficient of earth pressure at rest $K_0$. While, at the end of the cyclic pre stressing $K_0$ has the same initial value and only plastic strain hardening is induced to the sand.

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

a) one dimensional consolidation along NC $K_0$-line;

b) penetration of one half of CC specimen;

c) cyclic pre stressing along different stress paths;

d) completion of penetration of CC specimen.

Figure 2 shows the results of one CPT and one DMT performed before and after the cyclic pre stressing along a stress path following the NC $K_0$-line (ABA):

- As a result of cyclic pre stressing $E'_3$ increases 4.5 to 7 times while PR and DMT indices $K_0$ and $E_3$ (see Marchetti, 1980) change very little.

- As a result of mechanical OC, the ratio of $E'_3/E'_2$ increases 3 to 5 times, while all the above mentioned penetration parameters normalized with respect to $E'_2$ or $E'_3$ increase much less, less than 10 to 30%.

- "IN-SITU" TESTS IN CC. PERFORMED ALWAYS AT $\phi$

- CYCLIC LOADING BETWEEN $\phi$ AND $\beta$

- $\beta$-TANGENT YOUNG'S MODULUS AT A

- $N_0$-NUMBER OF UNLOADING-RELOADING CYCLES

- $q_C$-CONE RESISTANCE; $q_D$-DILATOMETER WEDGE RESISTANCE

- $E_D$-DILATOMETER MODULUS; $K_0$-HORIZONTAL STRESS INDEX

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

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

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

**3 CORRELATION BETWEEN $E'$ AND CPT AND DMT RESULTS**

Figures 3 and 4 show the correlation between the results of CPT's and DMT's performed in the CC in predominantly silica TS and $E'_0$ or $M_0$ adopted as $E_M$. In these figures the following parameters are involved:

- Cone resistance $q_C$ measured at mid-height of the CC specimen.
- Constrained modulus $M_0$ inferred from the dilatometer modulus $E_D$ according to Marchetti (1980). The $E_D$ has been measured at mid-height of CC specimen.
- Tangent constrained modulus $M_0$ measured on the CC specimen at the stress level at which $q_C$ and $E_D$ have been measured.
- $E_D$ corresponding to the secant drained Young modulus inferred from CKD triaxial compression tests performed on pluvially deposited TS sample in a Bishop-Williams triaxial cell. The adopted values relate to the effective mean consolidation stress of the CC specimen and to a level of axial strain $T_e$ equal to 0.1%.

[Diagram and table have been omitted due to the nature of the text conversion.]
This value corresponds to the upper limit of the average vertical strain of practical interest when designing shallow and deep foundations in cohesionless soils (Jardine et al., 1986; Battaglio & Jamiolkowski, 1987).

Substantially analogous results have been obtained for HS.

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

- Even for the same sand the ratio of RDM to the penetration test results (\(q_c, M_o\)) is substantially higher for mechanically OC specimens than for NC specimens. This trend is observed in all sands and for all kinds of penetration tests validated in CC's around the world. It reflects the high sensitivity of RDM and the low sensitivity of penetration test results to the strain and stress history of the sand as shown by the special CC tests mentioned before.

- The ratio decreases, increasing \(D_R\) which, in first approximation, can be expressed as \(q_c / \sqrt{q_v} = \rho_c / \sqrt{q_v}\). This reflects the different influence of \(D_R\) on RDM and penetration test results.

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

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

4 CORRELATION BETWEEN \(G_o\) AND CPT AND DMT RESULTS

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

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

\[
G_o = f(D_R, \sigma'_d, \sigma'_b) \quad \ldots (1)
\]

where:

- \(\sigma'_d\) = effective stress acting in direction of seismic wave propagation, and
- \(\sigma'_b\) = effective stress acting in direction of soil particle displacement.

On the other hand, the same basic variables \((D_R, \sigma'_d)\) influence PR and DMT indices. This might explain the good performance of the Ohta & Goto (1978) formula (or other similar) which relate \(V_{pmax}\) and shear wave velocity \(V_s\). As demonstrated by Seed et al. (1986), Jamiolkowski et al. (1988) and Baldi et al. (1988), this formula provides a reliable estimate of \(V_s\) limited to cohesionless Holocene deposits. Figures 5 and 6 show the correlations between \(G_o\) measured during
resonant column tests (RCT) and the results of DMTs and CPTs performed in the CC, both related to the pluviately deposited silica TS. The same figures include also data from CPT (qc), cross-hole (Vg) and seismic cone penetration (qc, Vg) tests performed at the sites of Po river sand and of the Gioia Tauro sand with gravel. Good agreement between the measured Vg and those inferred from qc for the above mentioned sites is indicated by the data shown in Figure 7 (Lo Presti & Lai, 1988). Although the data for TS were obtained on recently deposited, unaged sand the results in Figures 5 and 6 show good agreement with the data of natural cohesionless deposits having an age of 3,000 to 20,000 years at the maximum considered depth of 30 m. However, Figure 6 shows some scatter between laboratory and field data than Figure 5. In view of the above statements, Figure 8 gives a tentative correlation between Gm and qc for uncedented predominantly quartz sands.

5 USE OF Gm IN SETTLEMENT PREDICTION

The application in practice of the correlations, like the one shown in Figure 8, depends strongly on the development of a link between the small strain shear modulus and soil moduli involved in the geotechnical design problems as strain level and mode of deformation are different (Swiger, 1977; Robertson, 1982; Bellotti et al., 1998b; Yoshida et al., 1984). To evaluate settlements of shallow foundations in cohesionless soils, the following simplified tentative procedure is suggested:

- Supposing, that in first approximation the theory of isotropic elasticity is valid, small
strain Young’s modulus can be computed as
\[ E'_s = \frac{1}{\gamma_{oc}} E'_n = \frac{1}{\gamma_{oc}} E'_s \] assuming a Poisson ratio \( \nu = 0.1 \), according to Lade (1988).

With the so obtained \( E'_n \), compute Young’s modulus degradation \( C_a \) using one of the available empirical rules such as the hyperbolic (Hardin & Drnevich, 1972) or Ramberg-Osgood (Ishihara, 1986) equations.

Once the \( E'_s \) vs \( C_a \) relationship has been established, the appropriate \( \gamma_{oc} \) Young’s modulus for settlement calculation can be selected.

It is extremely important to realize that the obtained \( E'_n \) vs \( C_a \) curve refers to the second stiffness of soil after it has been subjected to a large number of unloading-reloading cycles. This means that:

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

It is postulated that the above outlined approach to infer \( E'_n \) from \( C_a \) can also be applied to the NC cohesions deposits having age exceeding a few thousand years. In fact, the writers believe that due to the processes occurring in the natural cohesionless deposits such as: ageing, small strain cyclic stressing due to low amplitude earthquakes, weak cementation, etc., their behaviour matches with that of slightly mechanically OC deposits (see Figure 2).

To investigate the validity of the above mentioned approach for estimating \( E'_n \) from \( C_a \), the results of a series of CTS and TX-CKD compression tests, performed on the pluvially deposited TS and HS specimens, have been compared.

The comparison has been made at the same level of octahedral shear strain \( \gamma_{oc} \) and octahedral mean stress \( \sigma_{oc} \) adopting \( \theta = 0.20 \). Degradation of shear modulus with strain have been simulated using Hardin & Drnevich (1972) hyperbolic stress-strain relationship. The results of this comparison are summarized in Table 2 and Figure 9. The Table 2 shows the ratio of \( E'_n \) to \( E'_n \) both evaluated at \( \gamma_{oc} = 0.14 \) which represents the upper limit of axial strain of practical interest. Figure 9 compares \( E'_n \) against \( E'_n \) at different strain levels for three OC and one NC specimens of TS.

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

- In OC sands \( E'_n \) at \( \gamma_{oc} = 0.14 \) is about 20% lower than \( E'_n \). This fact might be attributed to the following factors the simplified hypothesis adopted, the fact that \( E'_n \) reflects implicitly soil stiffness after a large number of loading cycles, and the contribution of irreversible strains to the assessment of \( E'_n \) particularly at low values of OCR.

In NC sand, as expected, the suggested approach to obtain \( E'_n \) from \( C_a \) does not work due to the reasons explained before.

The comparison of \( E'_n \) with \( E'_n \) along the broad range of axial strains (0.05 to 5%) indicates a good agreement at lower strains which becomes worse with increasing \( \gamma_{oc} \). This again confirms the usefulness of the suggested procedure only when the plastic strains are small (e.g. when the \( E'_n \) is measured within the current yield surface).

REFERENCES


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