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Using SCPT and DMT data for settlement prediction in sand

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ABSTRACT: This paper deals with the determination of parameters for use in predicting the settlement of footings on sand, recognising the non-linear stiffness characteristics of sand. Because of the difficulty of obtaining non-disturbed samples of sand for laboratory determination of the stiffness parameters, reliance must be placed on *in situ* tests. The approach adopted here is to combine the 'small strain' stiffness obtained from measurement of shear wave velocity (such as in a seismic cone penetrometer test, SCPT) with the 'larger-strain' value obtained from flat dilatometer (DMT) tests. These results are combined to deduce a strain dependent 'operational stiffness' for prediction of settlements of spread foundations on sand. It is shown that, when appropriate corrections are made to the dilatometer data, the approach outlined is capable of accurately predicting the load-displacement response measured in five footings tests.

1 INTRODUCTION

Although ground freezing may be employed to facilitate the retrieval of undisturbed samples of sand and gravel, the geotechnical profession needs to rely, almost exclusively, on *in situ* tests to assess design parameters for cohesionless soil. However, many of the *in situ* test devices in common use do not provide a direct measure of the non-proportional relationships between stress and strain changes in a soil (i.e. its non-linear stiffness) and, to estimate ground movements, practitioners generally resort to the derivation of a single 'operational' stiffness from empirical relationships with an *in situ* test parameter. The actual 'operational' stiffness relevant to foundation settlement predictions is extremely difficult to quantify as it depends on the combined influence in the vicinity of foundations of many factors such as strain, stress level, density, stress history, anisotropy and ageing. Given this range of dependencies and the tenuous link between stiffness and many *in situ* test parameters (such as penetration tests, which involve a shear failure of the soil), it is not at all surprising that empirical relationships between operational stiffness and *in situ* test parameters are not generally applicable and have a relatively poor reliability (Briaud & Gibbens 1994).

Shear wave velocities measured in seismic cone penetration tests (SCPTs) provide a direct and unambiguous measure of the *in situ* shear modulus at very small strains (G_{vho}). The strain levels immediately beneath most spread foundations are, however, much larger than those for which the use of G_{vho} is appropriate.

Fahey (1998) suggested that the self-boring pressuremeter (SBP) could be used in conjunction with the SCPT to deduce non-linear stiffness parameters for sand, thereby providing stiffness information relevant to any footing settlement calculation. However, it was also recognised that SBP testing is complicated, and unlikely to be used in routine practice, and suggested that the flat dilatometer test (DMT) may provide an appropriate (though less ideal) alternative. The DMT test provides a measure of soil stiffness at a relatively high strain level and it is therefore reasonable to presume that combining SCPT and DMT data may lead to the derivation of strain dependent 'operational moduli' and hence a much improved settlement prediction approach than one employing a single 'operational modulus'.

However, it is not clear how the modulus measured in a DMT may be converted to one suitable for settlement prediction, other than by empirical correlations. This is primarily because (i) the insertion of the dilatometer blade causes significant changes to the *in situ* stress conditions prior to the modulus measurement and (ii) the measured modulus is more strongly related to the horizontal stiffness than to the vertical stiffness. An ideal dilatometer would be 'wished-in-place' and provide a measure of vertical stiffness.

To explore the suitability of a settlement prediction approach combining SCPT and DMT data, this paper firstly presents trends indicated by SCPT and DMT data in the sands of Perth, Western Australia. These trends are then used together with findings from a set of field tests that investigated effects of DMT disturbance, to deduce an expression for a vertical operational stiffness that may be inferred from the DMT. Finally, the suitability of combining DMT with SCPT data in a settlement prediction method is examined using footing test results presented by Briaud & Gibbens (1994).

2 PERTH DATABASE OF SCPT AND DMT DATA IN SAND

The use of the seismic cone in Perth has grown significantly over the past five years and, more recently, dilatometer testing has also gained considerable popularity. As a consequence, a relatively large database of *in situ* test results (which also includes self-boring pressuremeter data) has been accumulated. The SCPT and DMT data presented here were obtained at 15 sand sites in the Perth region. The sand at these sites may be classified into four groups (i) Quaternary alluvial siliceous sand, (ii) mid-late Pleistocene aeolian/dune (Spearwood) siliceous sand, (iii) Quaternary calcareous sand and (iv) hydraulic fill.

The G_{vho} values measured at the 15 Perth sites are normalised by the CPT end resistance (q_c) and plotted on Fig. 1 against q_{c1}, where q_{c1} is a measure of the sand's relative density (D_r) and is defined as:

$$q_{c1} = q_c / (\sigma'_{vo}\, p_a)^{0.5} \quad (1)$$

where σ'_{vo} is the *in situ* vertical effective stress and p_a is the atmospheric pressure (= 100 kPa). The data are seen to follow the same patterns as those deduced by Robertson (1997) – i.e. G_{vho}/q_c reduces strongly with q_{c1} (implying that the relationship between G_{vho} and q_c is weak) and G_{vho}/q_c at a given q_{c1} (or D_r) increases with ageing or cementation of the sand. The very small strain vertical Young's modulus of the sand (E_{vo}) is approximately 2.2 times G_{vh0} (as Poisson's ratio at these strain levels is typically about 0.1).

The DMT is described, in detail, in numerous references by its inventor Prof. Silvano Marchetti (e.g. Marchetti, 1980; www.marchetti-dmt.it). The test essentially involves pushing a 15mm thick, 95mm wide and 250mm high stainless steel blade into the ground before (pneumatic) measurement of (i) the lateral stress required to cause lift-off of a steel membrane located at the centre of one side the blade (p_o) and (ii) the stress required to cause movement at the membrane centre (s_c) of 1.1mm (p_1). The increase in pressure (p_1-p_o) required to induce this membrane movement is used to derive the dilatometer modulus (E_D) from the following solution for a circular hole in a semi-infinite medium of modulus, E, bounded by a rigid wall (i.e. the DMT blade):

$$E_D = E/(1-\nu^2) = [(2D/\pi)\,(p_1 - p_o)]/s_c \quad (2)$$

For the membrane diameter D=60mm and s_c=1.1mm, Equation 2 reduces to:

$$E_D = 34.7\,(p_1 - p_o) \quad (3)$$

The dilatometer moduli for Perth sands calculated using Equation 3 are presented on Fig. 2 in the same format as that of the SCPT data on Fig. 1. The re-

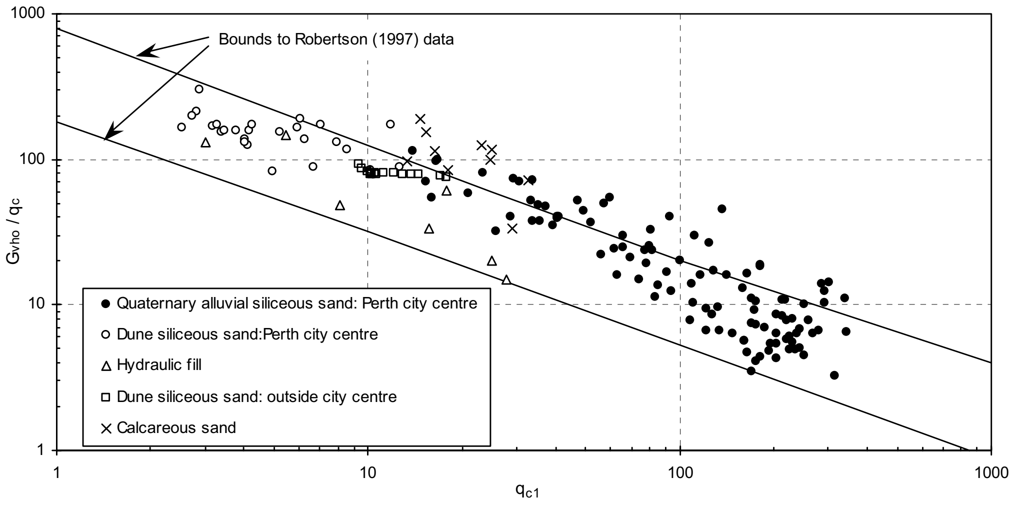

Fig. 1. Normalised G_{vho} values from SCPT tests from 15 Perth sand sites plotted against normalised q_c values.

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duction of E_D/q_c with q_{c1} is evidently comparable to that of the G_{vh0}/q_c ratios, suggesting that, despite disturbance effects, E_D provides a measure of *in situ* soil stiffness. A comparison of Figs 1 and 2 indicates that E_D/q_c, at a given q_{c1} value, is typically about ten times smaller than E_{vo}/q_c ($\approx 2.2G_{vh0}/q_c$); this difference may be largely attributed to the much higher strain levels induced by membrane expansion in the DMT.

Based on the trends evident on Figs 1 and 2, statistical analysis were conducted to search for relationships of the following form for Perth sand:

$$E_{vo} = f(q_c^{\,v}, \sigma'^{\,w}_{vo}) \qquad (4)$$

$$E_D = f(q_c^{\,x}, \sigma'^{\,y}_{v}, p'^{\,z}_{o}) \qquad (5)$$

where v, w x, y and z are best-fit coefficients and p'_o is the effective dilatometer lift-off pressure (= $p_o - u_o$).

The analyses conducted for Equation 4 indicated relatively similar coefficients of correlation for v = 0.25 and w = 0.5 and for v = w = 0.33. For the former combination, the relationships obtained are:

$$E_{vo} = [1100 \pm 400]\,\xi \quad \text{(for aged sand)} \qquad (6)$$

$$E_{vo} = [330 \pm 130]\,\xi \quad \text{(for sand fill)} \qquad (7)$$

where

$$\xi = q_c^{\,0.25}\, \sigma'^{\,0.5}_{vo}\, p_a^{\,0.25}. \qquad (8)$$

The ± variations in Equations 6 and 7 represent one standard deviation from the mean and their relatively large magnitudes confirm the need for *in situ* shear wave velocity measurements to assess G_{vho} (rather than inferring G_{vho} from correlations with other *in situ* test devices).

Initial statistical analyses conducted for Equation 5 assumed z=0 (i.e. that E_D may be estimated using q_c data alone). The best-fit relationship obtained for this case was:

$$E_D = 95\,[\,q_c\ \sigma'_{vo}\ p_a\,]^{1/3} \qquad (9)$$

with a coefficient of variation (COV) of 0.34.

A significantly improved relationship emerged when E_D was also allowed to vary with the effective lift-off pressure, p'_o:

$$E_D = 70\,[q_c^{\,0.25}\ p'^{\,0.5}_{o}\ p_a^{\,0.25}] \qquad (10)$$

with a COV of 0.22.

An illustration of the suitability of Equation 10 to predict the (600 or so) E_D values in the Perth sand database is provided on Fig. 3. Relatively good agreement for most data points is apparent and there is no systematic variation of the plotted ratio with σ'_{vo}. In addition, it may be noted that, contrary to trends indicated by G_{vho} data, the best-fit constant of 70 in Equation 10 does not depend on the sand type or stress history. This is presumably because the large strains imposed by the dilatometer destroy any *in situ* structure that the sand deposits may possess.

Equation 10 may also be expressed in terms of the DMT K_D parameter (=p'_o/σ'_{vo}) as follows, which suggests that, if the q_c and σ'_{vo} values are relatively constant, E_D varies with the square root of K_D:

$$E_D = 70\,[q_c^{\,0.25}\ K_D^{\,0.5}\ \sigma'^{\,0.5}_{vo}\ p_a^{\,0.25}]$$

$$= 70\,\xi\,K_D^{\,0.5} \qquad (11)$$

with a COV of 0.22.

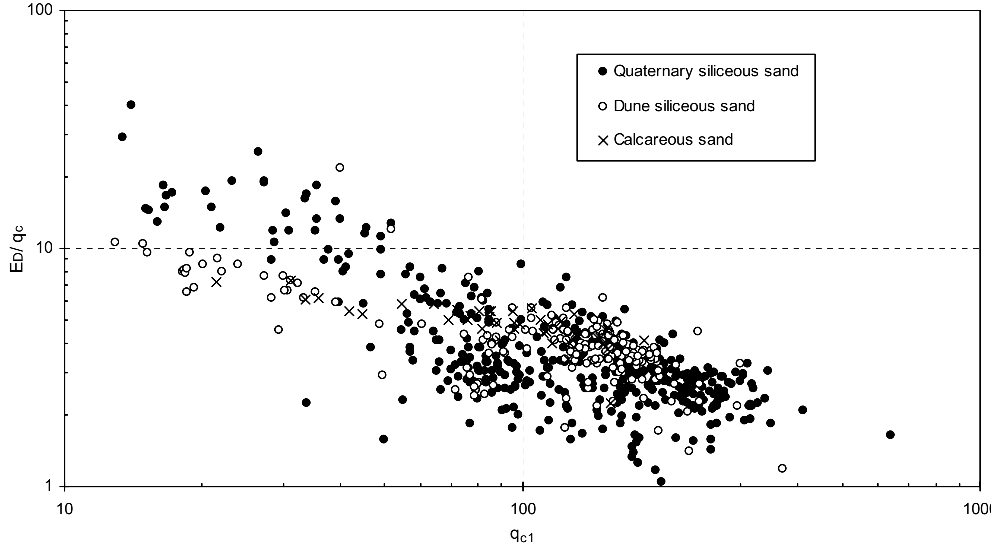

Fig. 2. Normalised DMT E_D values plotted against normalised q_c value for some Perth sands.

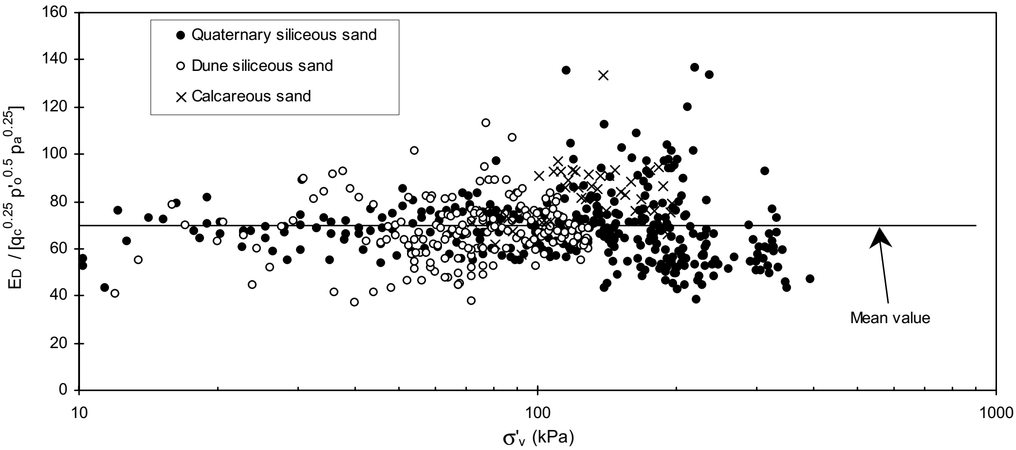

Fig. 3. DMT E_D values normalised by q_c and p'_o for the same data as in Fig. 2.

3 INVESTIGATION OF DMT DISTURBANCE

Installation of the DMT blade clearly causes disturbance to the *in situ* sand and therefore the value of E_D needs to be corrected for such disturbance if it is to be of use to predict *in situ* soil behaviour. In addition, as E_D is a measure of horizontal stiffness, a correction to obtain an appropriate vertical modulus is required for settlement prediction purposes. Because of these complications, settlement predictions using DMT data have, to date, only been performed using a range of empirical correlations developed by Marchetti (1980). These correlations apply a factor (R_m) to the E_D values to obtain a single operational soil constrained modulus (M), which for most sands approximates to:

$$M = R_m E_D = (0.5 + 2 \log_{10} K_D) E_D \tag{12}$$

A pilot study was therefore initiated to examine the effects of disturbance induced by dilatometer installation at a sand site at Shenton Park, which is a suburb to the west of Perth city centre. Previous research at this site had indicated that suction led to a strong seasonal dependency of *in situ* test parameters in areas with an abundance of native trees. All DMT investigations were therefore performed in an open area of the site, where suctions pressures within the sand were shown to be insignificant (see Lehane and Fahey, 2003).

The soil conditions at the test locations comprised between 8m and 12m of the slightly moist Spearwood dune sand with effective particles sizes, D_{50}, D_{60} and D_{10} of 0.42 mm, 0.47 mm and 0.21 mm respectively. Sand replacement tests indicated an *in situ* relative density (D_r) of between 45% and 55%. The programme of experiments comprised standard DMTs and a series of non-standard DMTs, which involved expansion of the DMT membrane in three trial pits following:

- pit excavation to a depth of 1.35m, 2.1m or 3.1m and placement of excavated sand around the dilatometer blade after locating it at the pit base i.e. a ‘wished-in-place’ dilatometer
- jacking the DMT into the backfilled sand to the base of each pit as in normal practice.

A variety of sand placement techniques were employed around the DMT blade, and sand replacement density tests were conducted to assess the corresponding relative densities. The values of p_o and p_1 measured in each of the experiments are plotted on Fig. 4, which also indicates the backfill D_r values and shows p_o and p_1 values obtained at the corresponding depths during standard DMT installations. It is apparent that values of p_o and p_1 (and hence also E_D) measured after installing the DMT into loose and medium dense backfill are typically about three times higher than those values measured for the ‘wished-in-place’ dilatometer. It is clear, therefore, that disturbance effects during installation are significant and need to be accounted for when assessing *in situ* stiffness from E_D.

DMT parameters measured after installation into backfill are only about 60% of the corresponding values measured following installation into *in situ* sand at the same relative density; e.g. see test results on Fig. 3 at a depth (z) = 3m. This result may reflect effects of ageing on the stiffness of the natural material.

The E_D and K_D values measured in the backfilled sand for the pushed-in and ‘wished-in-place’ dilatometer are summarised in Table 1. The ratio of E_D recorded for push-in DMT installation (E_{D1}) is seen to be ≈2.7 times E_D recorded for the ‘wished-in-place’ case (E_{D2}), irrespective of the sand’s relative

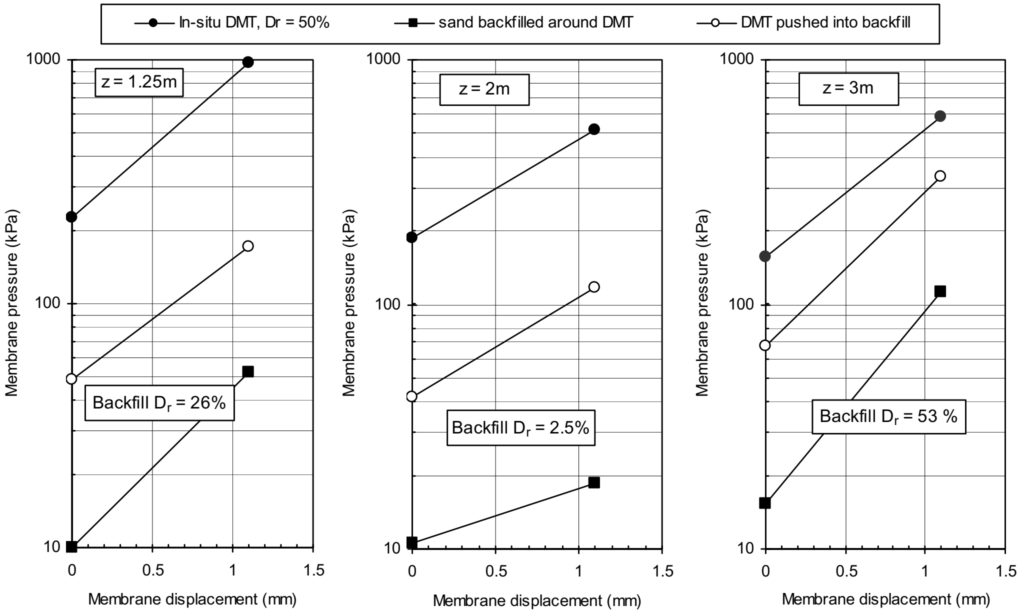

Fig. 4. Results of DMT tests at Shenton Park, with DMT pushed into *in situ* soil (I_D = 50%); pushed into backfilled soil; or placed in pit and backfilled placed around it.

density. E_D evidently increases as K_D increases and, as indicated in Table 1, the E_{D1}/E_{D2} ratio is approximately proportional to the square root of the respective K_D values. This trend is consistent with that indicated by Equation 11.

The foregoing suggests that Equation 11 may be modified as follows to predict the DMT (constrained) modulus that would be measured by a 'wished-in-place' dilatometer orientated horizontally i.e. measuring vertical stiffness.

$$M_{DV} = E_D/K_D^{0.5} \approx 70\xi \qquad (13)$$

The value of M_{DV} has clearly more relevance to settlement prediction than E_D and may be thought of as an operational modulus at the settlement ratio (s_c/D) of 1.8% applied by the dilatometer. This 'transformation' ignores inherent anisotropy in the sand and may require adjustment. The tendency of natural sand deposits to possess anisotropy may well explain why the adjustment to E_D in Equation 13 contrasts with the traditional empirical correction factor given in Equation 12.

Table 1. DMT parameters obtained in backfill

Depth (m)	D_r (%)	Pushed in E_{D1} (MPa)	Pushed in K_{D1} (MPa)	"Wished" in E_{D2} (MPa)	"Wished" in K_{D2} (MPa)	$\frac{E_{D1}}{E_{D2}}$	$\frac{K_{D1}}{K_{D2}}$	$\sqrt{\frac{K_{D1}}{K_{D2}}}$
1.25	26	4.3	2.45	1.5	0.50	2.87	4.9	2.2
2.0	2.5	2.9	1.03	1.1	0.33	2.64	3.1	1.8
3.0	53	9.2	1.40	3.4	0.32	2.71	4.4	2.1

4 NUMERICAL ANALYSIS OF DMT MEMBRANE EXPANSION

A series of Finite Element (FE) analyses of membrane expansion were performed using soil constitutive models incorporating the strong stiffness nonlinearity of sand. Both the AFENA FE program (Carter and Balaam, 1990) coupled with the *f-g* shear stiffness formulation (Fahey and Carter, 1993) as well as the SAFE FE program (OASYS, 2003) with the BRICK constitutive model (Simpson, 1992) were employed. These numerical analyses indicated that the restraint provided by the dilatometer blade has a strong effect on the displacements in a linear elastic soil but has little effect in a soil possessing stiffness characteristics of 'real' soil. Various membrane expansion profiles predicted using SAFE by application of a uniform pressure on a membrane of radius (R) equal to 30mm in axi-symmetric analyses are provided on Fig. 5. It is evident that, in the linear elastic soil, the absence of the restraint offered by the blade leads to 30% higher displacements at the membrane centre. It is also apparent that, in a 'real soil', the membrane profile is closer to that induced by a rigid foundation than that typical of a flexible foundation.

Based on these analyses, it is concluded that the actual operational modulus measured in a DMT is about 1.3 times the E_D value calculated using Equations 2 and 3. Equation 13 should therefore be modified to:

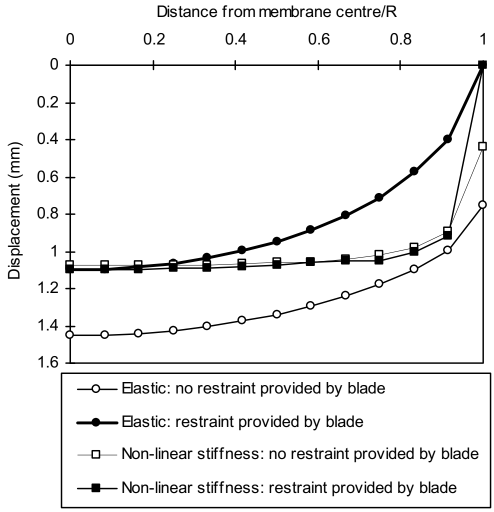

Fig. 5. FEM modelling of DMT blade expansion, in linear elastic and non-linear elastic soil.

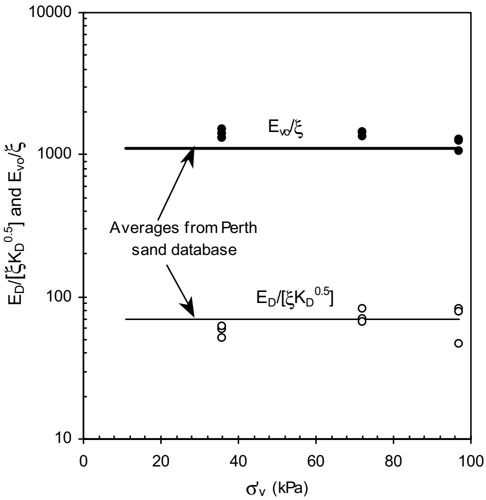

Fig. 6. DMT data from Texas site (data from Briaud and Gibbens, 1994).

$$M_{DV} = 1.3\ E_D/K_D^{0.5} \approx 90\ \xi \text{ at } s/B = 1.8\% \quad (14)$$

5 APPLICATION OF DMT TO PAD FOOTING SETTLEMENT PREDICTION

Briaud & Gibbens (1994) present the load-displacement data obtained in five footing tests performed at a medium dense silty fine sand site in Texas. The DMT data obtained at the site are presented on Fig. 6 in the same form as those of the Perth sand data on Fig. 2, and evidently are in good agreement with Equations 10 and 11.

The stresses applied to the footings (q_{app}) to induce a settlement ratio of 1.8% (i.e. the same as the DMT) are listed in Table 2 in addition to the constrained moduli (M) backfigured from the following equation for a rigid punch:

$$M = (\pi/4)\ q_{app}/(s/B) \quad (15)$$

Table 2 also lists the average initial vertical effective stress (including that due to the footing self-weight) and average CPT q_c within the depth of influence of the footings. This depth of influence was taken equivalent to that assumed by Burland and Burbidge (1986). These average q_c and σ'_{vo} values were used to derive the tabulated values of M_{DV} using Equation 14.

Table 2 Backfigured and predicted constrained moduli at s/B=1.8%.

Footing width (m)	q_{app} (kPa)	$M_{backfigured}$ (MPa)	$q_{c\ average}$ (MPa)	$\sigma'_{vo\ average}$ (kPa)	M_{DV} (predicted) (MPa)
3	830	36	9.6	64	23
3	681	30	4.1	69	19
2.5	684	30	7.0	61	21
1.5	712	31	4.5	55	17
1	784	34	8.7	50	20

It is evident from Table 2 that the backfigured values of M are significantly larger than the respective predicted M_{DV} values. The reason for this can, at present, only be surmised to be associated with inherent anisotropy in the sand deposit at Texas. Encouragingly, while the ratio of $M_{backfigured}/q_c$ varies significantly (from 3.7 to 7.2), the $M_{bacfigured}/M_{DV}$ ratio is consistent and is approximately 1.6 for all five footing cases. Such consistency supports the general form of Equation 14 and suggests the following amendment for inherent anisotropy:

$$M_{DV} = 1.3\ f_{aniso}\ E_D/K_D^{0.5} \quad (16)$$

where $f_{aniso} \approx 1.6$ at Texas (assuming creep differences between the DMT and footing tests can be ignored). The effects of inherent anisotropy can be appreciated on inspection of test results reported by Kohata et al. (1997), which indicated a ratio of small strain vertical to horizontal Young's modulus of between 1.1 and 2.5 for sand and gravel that was first consolidated to isotropic stress conditions; the higher ratios were obtained for compacted gravels.

6 COMBINING SCPT AND DMT DATA TO PREDICT FOOTING SETTLEMENT

Crosshole shear wave velocity measurements were also obtained at the Texas test site and the E_{vo} data derived using these are summarized in the same format as the E_D measurements at this site on Fig. 6.

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A best fit relationship to the data within the shallower depths affected by the footings is:

$$E_{vo} = [1300 \pm 100]\ \xi \qquad (17)$$

Equation 17 may be considered appropriate for the evaluation of settlements under very low levels of loading when footing settlement ratios (s/B) are less than about 0.006%. Atkinson (2000) suggests that this settlement ratio corresponds approximately to an elemental stiffness at a vertical strain of not more than 0.06/3 =0.002%. The constrained modulus at these very low strain levels (when Poisson's ratio is about 0.1) may be assumed to be equivalent to E_{vo}.

The shear wave velocity and DMT data are combined to produce a settlement ratio, s/B, (or strain) dependent modulus on Fig. 7 by assuming a linear degradation with log (s/B) of M/ξ, where the ratio of 1300 plotted at s/B≤0.006% is from Equation 17 and the ratio of 90 at s/B=1.8% is compatible with the mean value indicated on Fig. 6 with the correction indicated in Equation 14. The tentative correction for anisotropy suggested in Equation 16 is not employed here.

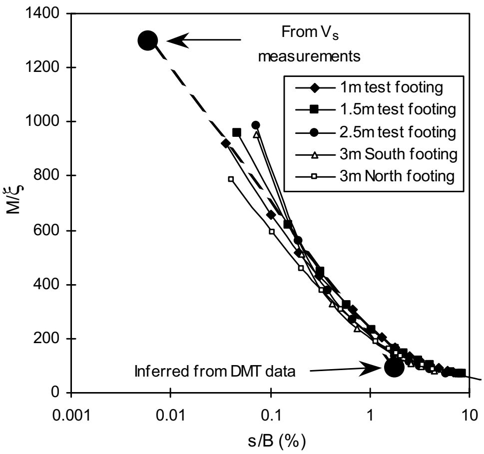

Fig. 7. Back-analysed stiffness values from footing tests of Briaud and Gibbens (1994) compared with values inferred from seismic (V_s) and DMT data.

The full load displacement data recorded for each Texas test footing were employed to backfigure the respective variations of constrained modulus (M) with s/B (in a similar way to that employed in Equation 15 for s/B=1.8%). Values of ξ were again derived at half the depth of influence of the foundations. The resulting variation of backfigured M/ξ values with s/B are plotted on Fig. 7.

It is apparent on Fig. 7 that, withstanding the under-prediction of settlement at s/B=1.8%, discussed previously, the linear variation of M/ξ with log (s/B) established using the SCPT and DMT data provides an excellent fit to the complete set of Texas footing load test data. Further studies of this nature for a variety of case histories should be conducted to test what is apparently a most promising approach for settlement prediction in sand.

7 CONCLUSIONS

This paper has illustrated the potential of employing an *in situ* test based non-linear settlement prediction method for spread footings on sand. This has been achieved by expressing the stiffness measurements in SCPT and DMT data in a similar normalised format and deriving appropriate corrections to DMT data.

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