Re-appraisal of the Dilatometer for In-situ Assessment of Geotechnical Properties of Swedish Glacio-Marine Clays

Tara Wood
Department of civil and environmental engineering, Gothenburg, Sweden. E-mail: tara.wood@chalmers.se
NCC Construction Sweden AB, Gothenburg, Sweden E-mail: tara.wood@ncc.se

Keywords: ground profile, small strain stiffness, undrained shear strength, in-situ stresses, degradation

ABSTRACT: This paper compares the results of in-situ field and high quality laboratory tests on Swedish soft highly structured glacio-marine clays. The applicability of SDMT measurements for both soil profiling and determination of soil properties are considered. Seismic dilatometer (SDMT) tests were found to define the ground profile as well as piezocone penetration tests (CPTU). Furthermore, it was found that in-situ stress state can be determined using existing correlations. However new correlations were required to define soft clay anisotropy in undrained shearing and clay stiffness, pre and post yield, consistent with high quality laboratory test results. Determination of small strain stiffness (G₀) and degradation (G/G₀) determined with the SDMT probe are compared with high quality triaxial tests and show reasonable agreement.

1 INTRODUCTION

Field testing with the dilatometer (DMT) was first introduced to Sweden in the late 1980’s. Initially DMT testing was used extensively within research and industry for clays. In total 10 DMT blades are registered within Sweden. However, soon after the introduction of the dilatometer the popularity of cone penetration testing (CPTU) increased due to its speed and ease of use in the field. This led to a significant reduction in the use of the dilatometer in Sweden. Currently only 2 of the 10 blades are still used. Among Swedish geotechnical practitioners there is a general consensus that DMT testing is unsuitable for clays and that other methods, such as shear vane and CPTU are more reliable for soil profiling and determination of soil parameters.

The initial purpose of the SDMT testing was to find the in-situ small strain stiffness (G₀) and allow comparisons with laboratory determined values. It was found however that SDMT measurements could also be used to give initial estimations of some parameters used in finite element analyses (FEA).

2 BACKGROUND

2.1 DMT revisited

Swedish practice for interpretation of DMT tests in clay was presented by Larsson & Eskilson (1989). Swedish correlations were developed as those specified by Marchetti (1980) were found to be unreliable for Swedish normally to lightly over consolidated soft clays. The information obtained from the dilatometer using the Swedish correlations for clays were soil type, density, earth pressure at rest coefficient, K₀, over consolidation ratio (OCR) and undrained shear strength, sᵤ. The dilatometer modulus, MDMT, was found to lie in between the reloading modulus and post yield (plastic) modulus thus was not deemed applicable. Other correlations for soil parameter determination from dilatometer tests in soft clays have been specified among others by Chang (1991) and Lunne et al. (1989). Various methods of interpretation of DMT tests for soft clays are considered in this paper and compared.

2.2 Information required for FEA analysis

Numerical FEA models for soft clays are discussed by Karstunen (2013) and Olsson (2013). The Gothenburg clays are structured, anisotropic, non-linear, rate dependent, viscous materials. Both an adequate characterisation of the ground profile is required in addition to input parameters for FEA constitutive model. Preliminary assessments of both based on SDMT tests are investigated in this paper.

Soil properties of particular interest for advanced FEA analysis are: earth pressure at rest (K₀), over consolidation ratio (OCR) and unit weight (γ) for determination of initial stress state in the ground. The failure criterion is normally related to drained shear strength (c’, φ’, M), however comparison to
undrained strengths is also useful when validating drained model parameters. The deformation parameters required depends on constitutive model but potentially includes small strain stiffness \((G_0)\), shear modulus degradation \((G/G_0)\), pre-yield parameter \((C_s, \kappa, E_{50}, E_{ur}, M_0)\), post-yield parameter \((C_c, \lambda, E_{oed}, M_0)\), and creep parameter \((r_s, \mu^*)\). No attempt of creep from SDMT tests is made in this paper. The other model parameters are discussed.

3 FIELD TESTING

Four new SDMT test sites were studied in Gothenburg indicated in Fig. 1 located where deep excavations (>10 m) are planned for construction. At site 4 two tests were carried out to verify the repeatability of the SDMT. Other field tests carried out at these sites included; shear vanes, CPTU, piezometric measurements and sampling with the Swedish fixed piston sampler (STII). Field testing and sampling was done with a Geotech 504 boring rig. For dilatometer tests care was taken to keep the expansion of the membrane at a constant rate as work by Smith (1989) showed that rate of expansion can affect the \(P_0\) and \(P_1\) pressure measurements, where \(P_0\) and \(P_1\) relate to the pressures to inflate the membrane 0.05 mm and 1.1 mm respectively. For seismic testing the shear wave was produced using a 10 kg hammer hitting a reinforced timber beam, similar to the arrangement described by Marchetti et al. (2008). The boring rig was used to provide reaction force on the shear beam and efficient energy transfer from the hammer to the ground. The procedure was repeated at least 3 times and shear wave velocity, \(V_s\), assessed using the method outlined by Marchetti et al. (2008). If the variability coefficient of \(V_s\) exceeded 1% further tests were performed, although this was rarely necessary. The seismic probe used to determine \(V_s\) consists of a cylindrical probe placed above the DMT blade and contained within the pushing rods. The probe is equipped with two mono-axial geophone receivers compliant with the ASTM standards. The receivers were spaced 0.5 m apart and the signal was amplified and digitized in the probe.

The location of earlier measurements of in-situ \(V_s\) in Gothenburg by Andreasson (1979) using downhole and crosshole methods is presented in Fig. 1 as Site 5. This site was also used to determine DMT correlations specified by Larsson (1989). Also shown in Fig. 1 is the location of Site 6 where \(V_s\) was determined with multichannel analysis of surface waves (MASW) using the method described in Donohue et al. (2004) and is included for comparison with SDMT \(V_s\) values in Fig.5.

4 LABORATORY TESTING

Calibration of DMT correlations were done using results from high quality fixed piston samples taken at site 1. Following extraction samples were immediately taken to the laboratory for testing. Index tests and CRS oedometer tests were carried out within 1 hour of extraction. Four of the eleven triaxial tests were started within 2 hours of extraction and all but two triaxial samples were tested within 7 days. These later samples were tested within 1 month (45m CkoUE and 55m CkoUC). Stepwise (IL) oedometer tests for 10, 18 and 27 m were carried out after 4 days, whereas IL tests at 35 m, 45 m and 55 m were carried out after approximately 1 month.

The Swedish STII piston sampler provides 3 samples of height 170 mm and diameter 50 mm. The quality of the samples taken from the middle and lower tubes from 10 m, 18 m, 27 m, 45 m and 55 m was assessed to be very good to excellent for samples tested within 7 days based on Lunne et al. (1997) and Landon et al. (2007). Samples taken from 35 m were disturbed during extraction. Assessment of these samples ranged from good to poor, as did the 1 month old triaxial samples.

5 GROUND PROFILE

The area of Gothenburg is characterised by the crystalline bedrock sculpted by the effects of glaciation. The deep gorges in the rock have been filled with sediments after recession of the glaciers and in the area of central Gothenburg these sediments are principally clays. The varying sedimentation conditions in the glacio-marine environment during clay deposition are significant
as they gave rise to different clay structures principally due to the different ionic strength of the pore water, but also influenced by the speed of sediment transport, water depth, landslides, ice rafting activity and bacteria.

The Gothenburg clays have a plasticity index $I_p \approx 40$. They are predominantly illitic but plasticity is also influenced by the silt fraction ($\approx 30\%$), pore water and other clay minerals. Different clay structures and sedimentary boundaries due to varying conditions should be identifiable within the $V_s$ profile. To help identify if this is possible the sedimentary geology classifications of Alte et al. (1989) and Bergsten (1991) have been amalgamated in Table 1. The results from two SDMT tests taken 3 m apart at site 4 are shown in Fig. 3 together with these boundaries. The dilatometer parameters $E_D$ and $K_D$ are defined as $E_D = 34.7(P_1 - P_0)$ and $K_D = (P_0 - u_0/\sigma'_w)$. The repeatability of SDMT tests is excellent, particularly for the seismic. Clearly small local variations exist but the clay appears homogeneous. A sand layer was identified at 16 m in both SDMT profiles and confirmed by CPTU tests. This layer could be significant for the planned cut and cover tunnel at this site as the layer lies just below the planned excavation depth. Bergsten (1991) noted fissures in samples below 23 m due to erosion, this boundary appears to be identified in the $V_s$ profile. Further erosion and increased sedimentation events are apparent below this level in the $V_s$ profile and $K_D$ profile but not in the $E_D$ profile.

The classification of the ground profile at site 4 using different methods is presented in Figure 4. Assessment (a) from DMT uses Marchetti and Crapps (1981), while (b) uses the chart given by Larsson (1989), (c) uses CPTU tests from Larsson (2007) and (d) is based on all measurements. Assessment (a) erroneously identified the dry crust as silt otherwise it is very similar to (c) and (d). Method (b) correctly identified the stiffer dry crust but failed to identify the very soft clay within the zone 1B clays. All methods identified a frictional layer at around 16 m depth. Based on Fig. 4 CPTU and DMT tests provide similar evaluations of the ground profile.

The usefulness of $V_s$ as both a profiling tool and for understanding the effects of stress history can also be seen in Figure 5 where $V_s$ profiles from 5 sites are compared. The 5 sites were all subjected to loading in the 1800’s due to land reclamation. At this time excavations for a dock at site 3 and canal at site 4 were carried out. This dock was later refilled in 1934. The $V_s$ values in the zone 1 clays are greater at site 3 and 4 due to recent stress history but are most prevalent at site 3 where additional loads were applied. SDMT measurements at site 2 and 3 were done during a cold period (< -10°C), which clearly caused very high measurements of $V_s$ in the upper 5 m of the ground profile. Below the Zone 1 clays (12 m) profiles for all the sites are very similar confirming the homogeneity of these clays and the ability of the $V_s$ profile to identify changes at the expected geological boundaries.

<table>
<thead>
<tr>
<th>Strata</th>
<th>Age (years)</th>
<th>Base of strata (±1m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Made Ground</td>
<td>≈ 150</td>
<td>≈ 2m (Site 3 ≈ 7.5m)</td>
</tr>
<tr>
<td>Post Glacial 1A clay</td>
<td>8000</td>
<td>5.5 m</td>
</tr>
<tr>
<td>Post Glacial 1B clay</td>
<td>9000</td>
<td>8 m</td>
</tr>
<tr>
<td>Post Glacial 1C clay</td>
<td>10000</td>
<td>12 m</td>
</tr>
<tr>
<td>Post Glacial 2D clay</td>
<td>10600</td>
<td>21 m</td>
</tr>
<tr>
<td>Glacial 3aD clay</td>
<td>12000</td>
<td>42 m</td>
</tr>
<tr>
<td>Glacial 3bD clay</td>
<td>13000</td>
<td>57 to 100 m</td>
</tr>
</tbody>
</table>

**Table 1. Geological profile of Gothenburg with Zone 1 (1A, 1B, 1C), Zone 2 (2D) and Zone 3 (3aD, 3bD) clays**

![Fig. 3. Results of SDMT tests at Site 4](image)

![Fig. 4. Ground profile with different assessment methods](image)
6 ESTIMATION OF DESIGN PARAMETERS

Parameters of particular interest for FEA analysis of soft soils were discussed in Section 2.2. Methods of determining preliminary estimations of all of these parameters with the exception of creep parameters are put forward based on SDMT tests in this section. The laboratory high quality samples described in section 4 were used to assess the validity of existing empirical correlations and determine some new correlations.

6.1 Determination of in-situ stresses

6.1.1 Determination of vertical stress

The unit weights of clays tested have been evaluated from DMT tests using Marchetti and Crapps (1981) and Larsson (1989) and were compared with measured values from samples extracted at site 1. Measurements of \( V_s \) were also used to assess soil density using the mass density correlation presented by Mayne et al. (1999).

Assessment using Marchetti and Crapps (1981) slightly over estimated densities (<5%), while Larsson (1989) gave overestimations of 10-15%. The Mayne et al. (1999) correlation gave very accurate soil densities, within 1.3% of measured values thus appears to provide the best basis for determination of vertical total stress. The impact of small inaccuracies of unit weight for effective stress determination will be small considering uncertainties in pore water pressures.

6.1.2 Determination of horizontal stresses

Horizontal stresses can be determined if the in-situ earth pressure at rest coefficient, \( K_0 \), is known. At site 1 in-situ \( K_0 \) was assessed using the relationship derived by Schmidt (1966). Values of OCR and \( \phi' \) were taken from laboratory tests. The value of \( K_{0\text{nc}} \) was estimated from the ratio of horizontal and vertical yield stress (\( \sigma'_{ch} \) and \( \sigma'_{cv} \)) determined from undrained triaxial stress paths in compression and extension. These values were confirmed by \( K_0 \) consolidation tests reported by Olsson (2013). The assessment of \( K_0 \) from the dilatometer using Marchetti (1980) and Larsson (1989) are presented in Figure 6. The \( K_0 \) correlation by Lunne et al. (1989) is almost identical to Larsson (1989) thus is not plotted. Further verification is provided by field measurement of \( K_0 \) at site 5 presented by Smith (1989) which includes measurement with Glotz cells and self-boring pressuremeter (SBP). Larsson (1989) appears to be slightly more consistent with field and laboratory assessed values at these two sites.

Fig. 6. Comparison of in-situ \( K_0 \) estimations

6.1.3 Determination of over consolidation ratio

Four existing correlations of vertical OCR based on the dilatometer horizontal stress index \( K_D \) are presented in Figure 7 together with laboratory determined values. The rate dependency of the clays is evident particularly with depth seen by the enhanced yield stress of 1 day CRS oedometer tests when compared to increment load (IL) tests which took 10 days. The best correlation appears to be obtained using Chang (1991) whose correlation was...
and varying plasticity. In Larsson et al. (1985) and of post glacial clays are shown to be similar to it is also said to incorporate effects of loading rate derived shear strengths, and embankments in addition to average laboratory calculated from landslides, pile tests, foundations, This correlation considers mobilized strengths, fall cone and shear vane strengths by 15 to 20%. presented by Larsson et al. (1985) and reduces the correlation applied is correlated in-situ shear vane and fall cone tests are very close to values from triaxial extension tests. Based on work covering soft clay anisotropy by Hight (1998), Lunne et al. (1997) and Karstunen (2013) one would expect direct simple shear strengths to be greater, more similar to the average value from triaxial compression and extensions tests. The results of correlated in-situ shear vane and fall cone tests are also plotted in Fig. 8. The correlation applied is presented by Larsson et al. (1985) and reduces the fall cone and shear vane strengths by 15 to 20%. This correlation considers mobilized strengths, \( \tau_{moh} \), calculated from landslides, pile tests, foundations, and embankments in addition to average laboratory derived shear strengths, \( \sigma_{uav} \).\( \tilde{\sigma}_{uav} \) and \( \tilde{\tau}_{uav} \) are shown to be similar to \( \sigma_{uav} \) and \( \tilde{\tau}_{uav} \) assessed from failures in the ground and are in agreement with correlated fall cone and shear vane tests, \( \tau_{u} \). Based on the results presented in Fig. 8 the average characteristic undrained strength, \( \sigma_{uav} \), is significantly higher (75-85%). The difference between \( s_{uav} \) from site 1 and \( \tau_{moh} \) determined by Larsson et al. (1985) will be due to the impact of softening, and uncertainties like drainage, rate effects and geometry. Post peak softening in undrained triaxial test results presented here was up to 80% thus is of a similar order to the differences between site 1 test results and Larsson (1985). The discrepancy between site 1 \( s_{uav} \) and \( \sigma_{uav} \) reported by Larsson et al. (1985) and site 1 \( s_{uav} \) is most likely related to issues of storage effects and sample disturbance.

The calculation of undrained strength from DMT tests has been determined in three different ways. The most common method uses a critical state soil mechanics type model such as that proposed by Ladd (1977) where \((s_{u}/\sigma'_{vo})\) is defined in Eq.1.

\[
(s_u/\sigma'_{vo})a=a*OCR^m
\]  

The DMT correlation uses the horizontal stress index, \( K_D \) and estimated \( \sigma'_{vo} \) to define undrained shear strength; Marchetti (1980), Lunne et al. (1989), Chang (1991). The DMT undrained strengths obtained are essentially corrected field shear vane strengths and agree well with correlated shear vanes from site 1 in Fig. 8. There is a significant variation in the ratio \((s_u/\sigma'_{vo})\) for different soils as shown by Lutengger (1991) which explains why so many “local” correlations exist to determine \( a \) and \( m \).

Other authors argue it is more appropriate to estimate undrained strength based on a simple bearing capacity approach using the inflation pressure \( P_1 \), and estimated \( \sigma_{th0} \); Larsson (1989), Roque et al. (1988), refer to Eq.2.

\[
s_u=(P_1-\sigma_{th0})/N_b
\]  

For Swedish clays Larsson (1989) suggests a value for \( N_b \) of 10.3. The correlation is based on correlated shear vane tests. At site 1 estimations of \( s_u \) determined with Eq.2 are similar to \( \tau_u \) from shear vane tests and DMT correlations using \( K_D \).

Alternative methods of assessment of \( s_u \) from \( G_0SDMT \), and empirical \( G_0 \) estimations by Andreasson (1979) and Bråten et al. (2010) have been investigated. The values \( s_u \) in the Scandinavian \( G_0 \) correlations again relate to \( \tau_u \) and give values of \( s_u \) that lie close to both \( \tau_u \) assessed at site 1 and DMT estimates using \(-K_D \) and \( P_1 \). None of the correlations discussed so far provide good estimates of undrained characteristic shear strengths to help validate FEA analysis. To depths of around 35 m reasonable estimates of strength in extension \( s_{uav} \) can be

---

**Fig. 7.** Comparison of estimated overconsolidation ratio \( \sigma'_{vo}/\sigma'_{vo} \) using DMT with laboratory tests

**Fig. 8** The average characteristic undrained strength, \( \sigma_{uav} \) is significantly higher (75-85%). The difference between \( s_{uav} \) from site 1 and \( \tau_{moh} \) determined by Larsson et al. (1985) will be due to the impact of softening, and uncertainties like drainage, rate effects and geometry. Post peak softening in undrained triaxial test results presented here was up to 80% thus is of a similar order to the differences between site 1 test results and Larsson (1985). The discrepancy between site 1 \( s_{uav} \) and \( \sigma_{uav} \) reported by Larsson et al. (1985) and site 1 \( s_{uav} \) is most likely related to issues of storage effects and sample disturbance.

The calculation of undrained strength from DMT tests has been determined in three different ways. The most common method uses a critical state soil mechanics type model such as that proposed by Ladd (1977) where \((s_u/\sigma'_{vo})\) is defined in Eq.1.

\[
(s_u/\sigma'_{vo})a=a*OCR^m
\]  

The DMT correlation uses the horizontal stress index, \( K_D \) and estimated \( \sigma'_{vo} \) to define undrained shear strength; Marchetti (1980), Lunne et al. (1989), Chang (1991). The DMT undrained strengths obtained are essentially corrected field shear vane strengths and agree well with correlated shear vanes from site 1 in Fig. 8. There is a significant variation in the ratio \((s_u/\sigma'_{vo})\) for different soils as shown by Lutengger (1991) which explains why so many “local” correlations exist to determine \( a \) and \( m \).

Other authors argue it is more appropriate to estimate undrained strength based on a simple bearing capacity approach using the inflation pressure \( P_1 \), and estimated \( \sigma_{th0} \); Larsson (1989), Roque et al. (1988), refer to Eq.2.

\[
s_u=(P_1-\sigma_{th0})/N_b
\]  

For Swedish clays Larsson (1989) suggests a value for \( N_b \) of 10.3. The correlation is based on correlated shear vane tests. At site 1 estimations of \( s_u \) determined with Eq.2 are similar to \( \tau_u \) from shear vane tests and DMT correlations using \( K_D \).

Alternative methods of assessment of \( s_u \) from \( G_0SDMT \), and empirical \( G_0 \) estimations by Andreasson (1979) and Bråten et al. (2010) have been investigated. The values \( s_u \) in the Scandinavian \( G_0 \) correlations again relate to \( \tau_u \) and give values of \( s_u \) that lie close to both \( \tau_u \) assessed at site 1 and DMT estimates using \(-K_D \) and \( P_1 \). None of the correlations discussed so far provide good estimates of undrained characteristic shear strengths to help validate FEA analysis. To depths of around 35 m reasonable estimates of strength in extension \( s_{uav} \) can be
made from Lunne et al. (1989). Below 35m Marchetti (1980) gives a better indication of $s_{uc}$. 

6.2.2 Determination of drained strength

The determination of drained strength from DMT tests is generally limited to frictional soils. An attempt has been made to assess the critical state friction angle $\phi'_{cs}$ using the critical state soil mechanics concept presented by Wroth (1984) given in Eq. 3 was investigated where $\Lambda = 1 - (C_s/C_c)$. An assumption is made that the cohesive intercept $c'$ is zero during critical state shearing.

$$\frac{s_u}{\sigma'_{vo}} = \frac{1}{2} \sin \phi'_{cs} \cdot OCR^\Lambda$$

(3)

For the samples tested at site 1 the ratio $s_u/\sigma'_{vo}$ for laboratory tests in extension, simple shear and compression ranged between 0.25 to 0.35, 0.3 to 0.4 and 0.45-0.6 respectively with an average value around 0.37. For the highly structured Gothenburg clays the ratio $(C_s/C_c)$ is around 0.015-0.025 thus $\Lambda$ is very close to 1.

Using Eq. 3 and the OCR using Chang (1991), the critical state friction angle can be assessed if a ratio of $s_u/\sigma'_{vo}$ is assumed. Clearly the ratio $s_u/\sigma'_{vo}$ depends on a number of factors such as, direction of shearing, structure and not just OCR thus even use of specific ratio’s for compression and extension tests were not found to yield reliable estimations of critical state friction angles based on Wroth (1984) equation. Use of the average value of $s_u/\sigma'_{vo} = 0.37$ gave a friction angle of 36° which is similar to laboratory assessed critical state friction angle in extension. However laboratory values of $\phi'_{cs}$ in compression varied between 32° in the post glacial clays to 30.5° in the glacial clays. This method is therefore unreliable and not recommended for the determination of $\phi'_{cs}$. Reliance should instead be put upon good quality laboratory tests for determination of this parameter.

6.3 Determination of stiffness properties

6.3.1 Medium to large strain stiffness properties

The constrained modulus, $M_{DMT}$, is the confined drained vertical modulus at $\sigma'_{vo}$. This is determined from the dilatometer modulus $E_D$, which is effectively a disturbed modulus at strains slightly greater than many engineering situations calculated using elasticity theory which is then multiplied by an empirical factor $R_M$ (for clays is based on $K_D$). Correlations for $R_M$ have been suggested by Marchetti (1980) and Chang (1991) however as seen in Fig. 9 the assessments of $M_{DMT}$ do not represent either of the moduli typically used to define pre-yield “elastic” stiffness ($M_0$) or post yield “plastic” stiffness $M_L$ (where $M_L = 1/M_I$). It is not either the initial “disturbed” modulus found from initial loading in oedometer tests.
Instead a reasonable assessment of both “elastic” and “plastic” modulus could be found for all the levels studied by applying a factor to the original $M_{DMT}$ modulus. This factor was found to be 5 for the pre yield elastic modulus in the range where OCR varies between 1 and 2. The factor for the plastic modulus $M_L$ was found to be 0.125. Therefore, similar direct correlations should exist for stiffness parameters that are more appropriate for FEA such as $\lambda$, $\kappa$, $E'_{uted}$, $E'_{ur}$. However, correlation of correlated values is generally inappropriate. The deviatoric stiffness $E_{50}$ from compression triaxial tests at site 1 were found at axial strains of 0.35 to 0.6% which is less than the strains applied during inflation of the membrane and determination of $M_{DMT}$. Using elastic theory $E'_{DMT}$ can be found from $M_{DMT}$ (again if $v'$, is known) using eq. 4:

$$E'_{DMT}=F.M_{DMT} \left((1+v')(1-2v')(1-v')\right)$$

The value of $v'$ is not a constant and varies during shearing. Values of $v'$ at engineering strains are in the range of 0.1 to 0.3 but this gives gross under estimation of compression $E_{50}$ ($<35\%$). If the poisons ratio for the clays at failure ($v'=0.42$) is used and the factor $F=5$ (as used for the $M_0$) Eq. 4 gives estimations of $E'_{50}$ similar to those from CKoUE tests for the clay studied. One could therefore make an approximate estimation of compression $E'_{50}$ by first determining the value for extension $E'_{50}$ using Eq. 4 and $v'=0.42$ and then adjusting for anisotropy.

6.3.2 Small strain stiffness properties and degradation

The determination of in-situ small strain stiffness parameters with downhole seismic measurements was first reported in Sweden by Andreasson (1979). SDMT field measurements were later reported by Marchetti et al. (2008). The determination of small strain stiffness $G_0$ is found using elastic wave theory using the relation in Eq. 5:

$$G_0=\rho Vs^2$$

Where $\rho$ is the mass density and can be determined from the correlation suggested by Mayne (1999). The results of SDMT measurements of $G_0$ are presented in Fig. 10 together with laboratory values using bender elements and empirical correlations. The similarity of laboratory and field values of $G_0$ is a clear indication of the quality of the samples tested. Empirical correlations based on undrained strength and plasticity (PI or LL) gave reasonable agreement; Bråten et al. (2010), Andreasson (1979), whereas correlations presented by Hardin & Black (1968) gave poor agreement. Marchetti et al (2008) reported correlations of $G_0/M_{DMT}$ and $K_D$ however this correlation is only in agreement up to 5 m depth.

The use of SDMT tests to define stiffness degradation is discussed by Mayne et al (1999). A relationship is presented for the normalized shear modulus $G/G_0$ based on the degree of mobilized shear strength. If results of e.g. field shear vane tests are available. An alternate method can be used based on Hardin and Drnevich (1972) where the reference strain, $\gamma_r=\tau_{max}/G_0$, can be assessed from Fig. 8. The modulus degradation is defined using Eq. 6:

$$G/G_0=1/(1+\gamma_r)$$

This hyperbolic function is plotted Fig. 11 together with degradation curves determined in the laboratory and SDMT points. A reasonable fit is achieved, there is some under and overestimation of stiffness at small and medium strains respectively but these will tend to counterbalance each other at typical engineering strains of $10^{-4}$ to $10^{-3}$.
7 CONCLUSIONS

The work presented in this paper shows that SDMT tests are useful for both soil profiling and determination of initial conditions required for advanced FEA analyses. The use of $V_s$ as a profiling tool and for determination and cross checking of soil models and parameters in FEA analysis is very useful.

When assessing soil parameters using empirical correlations, it is important to consider what correlations are based upon and if this is relevant. A reliable method to find drained strength ($M_d$) from SDMT tests was not found however a correlation of peak undrained strength from standard $K_0$ consolidated triaxial tests appears promising.

The greatest contribution of the SDMT test in the characterization of soft clays for FEA analysis is its ability to assess the stiffness degradation using small and intermediate strain properties in conjunction with the Hardin-Drnevich backbone curve. Such curves are difficult and expensive to achieve in the laboratory due to problems associated with sampling disturbance and storage. This is a very promising direction for advanced field testing in soft soils.

8 REFERENCES


