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

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 (G_0) and degradation (G/G_o) 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 (G_o) 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, K_0, over consolidation ratio (OCR) and undrained shear strength, s_u. The dilatometer modulus, M_{DMT}, 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 (K_0), 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, κ, E_{50}', E'_{ur}, M_0), post-yield parameter (C_c, λ, E_{oed}, M_L), and creep parameter (r_s or μ^*). 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.
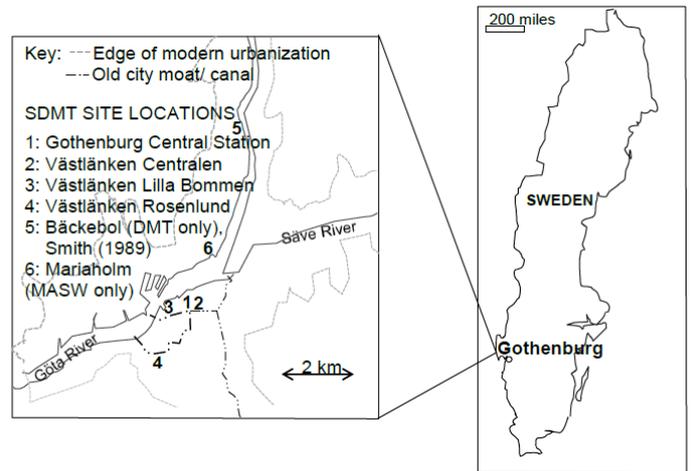
Fig.1. Location of test sites around Gothenburg

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 Ip≈40. They are predominantly illitic but plasticity is also influenced by the silt fraction (≈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 $K_D=(P_0-u_0/\sigma'_{vo})$. 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 23m due to erosion, this boundary appears be identified in the V_s profile. Further erosion and increased sedimentation events are apparent below this level in the V_s profile and E_D profile but not in the K_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

Fig. 3. Results of SDMT tests at Site 4

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

Strata	Age (years)	Base of strata (±1m)
Made Ground	≈ 150	≈ 2m (Site 3 ≈ 7.5m)
Post Glacial 1A clay	8000	5.5 m
Post Glacial 1B clay	9000	8 m
Post Glacial 1C clay	10000	12 m
Post Glacial 2D clay	10600	21 m
Glacial 3aD clay	12000	42 m
Glacial 3bD clay	13000	57 to 100 m
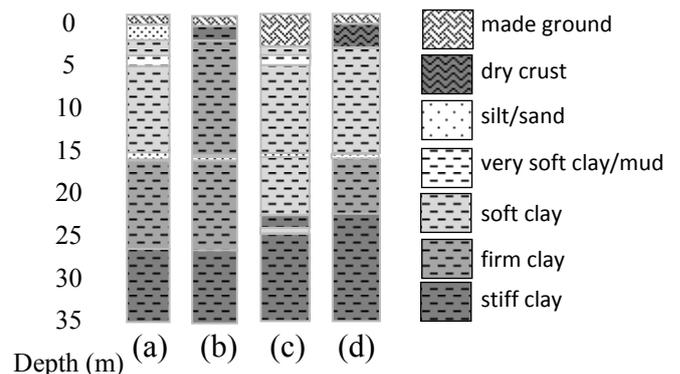
Fig.4. Ground profile with different assessment methods
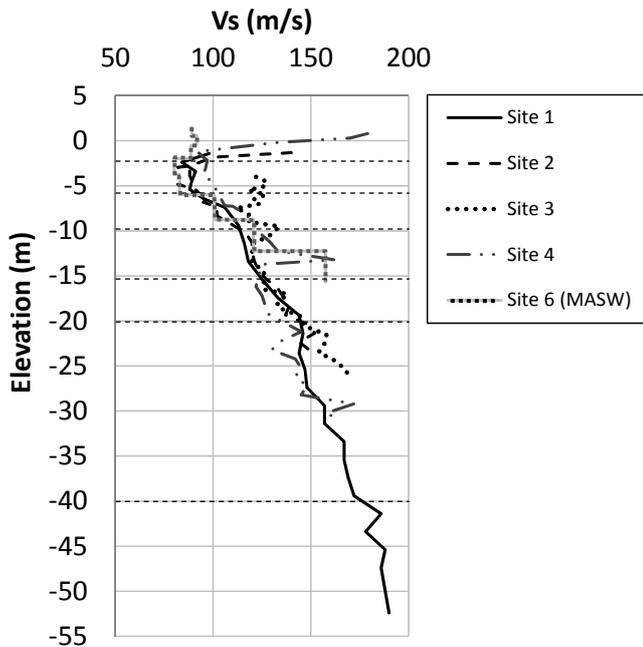
Fig.5. Comparison of V_s measurements in Gothenburg

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 ϕ' were taken from laboratory tests. The value of K_{0nc} was estimated from the ratio of horizontal and vertical yield stress (σ'_{ch} and σ'_{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.
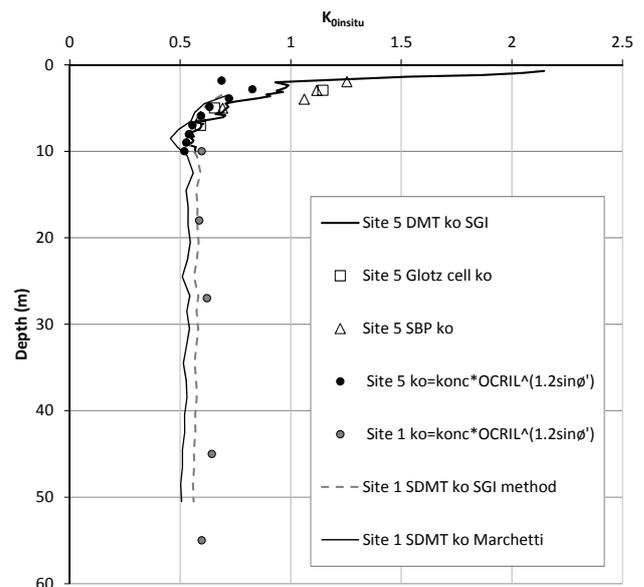
Fig.6. Comparison of in-situ K_0 estimations

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
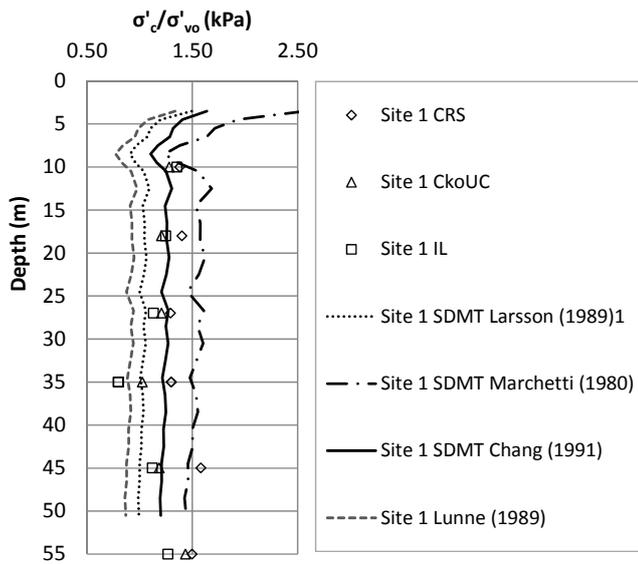
Fig. 7. Comparison of estimated overconsolidation ratio σ'_c/σ'_{vo} using DMT with laboratory tests

based on soft marine clays thus is recommended for soft glacio-marine clays and can be used in the assessment of soil stiffness discussed in 6.3.1.

6.2 Determination of strength properties

6.2.1 Determination of undrained strength

The Gothenburg clays are highly structured, and exhibit significant anisotropy. This is illustrated in Fig. 8 by the differences in the undrained shear strengths of triaxial tests in compression and extension. It should also be noted that the results of undrained direct simple shear tests lie 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, τ_{mob}, calculated from landslides, pile tests, foundations, and embankments in addition to average laboratory derived shear strengths, s_{uav}, (s_{uCKoUC}, s_{uDSS}, s_{uCKoUE}). It is also said to incorporate effects of loading rate and varying plasticity. In Larsson et al. (1985) s_{uDSS} of post glacial clays are shown to be similar to s_{uav}, and τ_{mob} assessed from failures in the ground and are in agreement with correlated fall cone and shear vane tests, τ_u. Based on the results presented in Fig. 8 the average characteristic undrained

strength, s_{uav} is significantly higher (75-85%). The difference between s_{uav} from site 1 and τ_{mob} 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 s_{uav} reported by Larsson et al. (1985) and site 1 s_{uDSS} 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/σ'_{vo}) is defined in Eq.1.

$$(s_u/\sigma'_{vo})_{OC}=a*OCR^{\wedge m} \tag{1}$$

The DMT correlation uses the horizontal stress index, K_D and estimated σ'_{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/σ'_{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 σ_{h0}; Larsson (1989), Roque et al. (1988), refer to Eq.2.

$$s_u=(P_1-\sigma_{h0})/N_b \tag{2}$$

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 τ_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 τ_u and give values of s_u that lie close to both τ_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_{uCkoUE} can be

made from Lunne et al. (1989). Below 35m Marchetti (1980) gives a better indication of s_{uCkoUE}.

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 ϕ'_{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.

$$s_u/\sigma'_{vo}=\tfrac{1}{2}\sin\phi'_{cs}*OCR^{\Lambda} \qquad (3)$$

For the samples tested at site 1 the ratio s_u/σ'_{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 Λ is very close to 1.
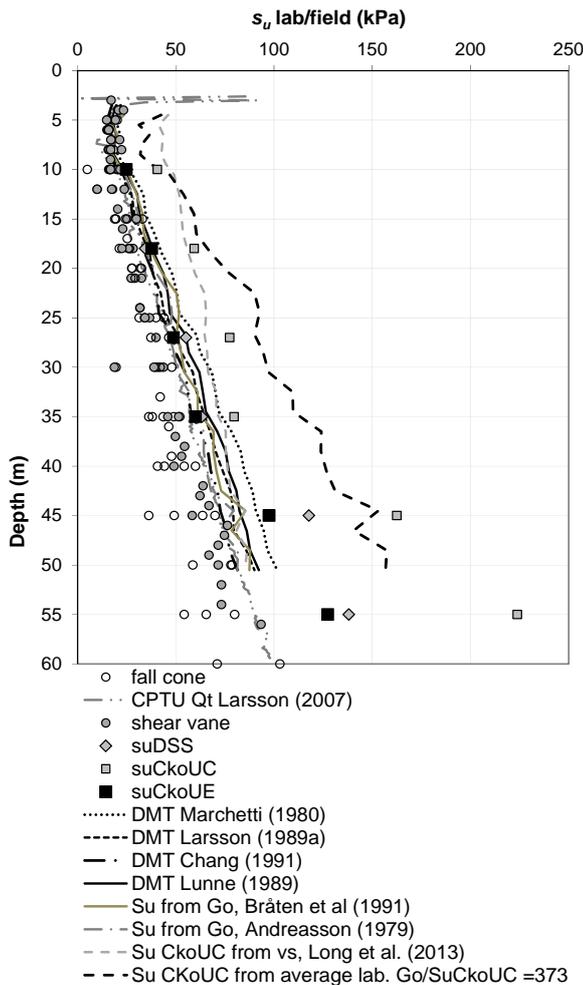
Fig.8. Comparison of estimated undrained shear strength using field and laboratory tests

Using Eq. 3 and the OCR using Chang (1991), the critical state friction angle can be assessed if a ratio of s_u/σ'_{vo} is assumed. Clearly the ratio s_u/σ'_{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 φ'_{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 ϕ'_{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 σ'_{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 $M_L=1/M_v$). It is not either the initial "disturbed" modulus found from initial loading in oedometer tests.
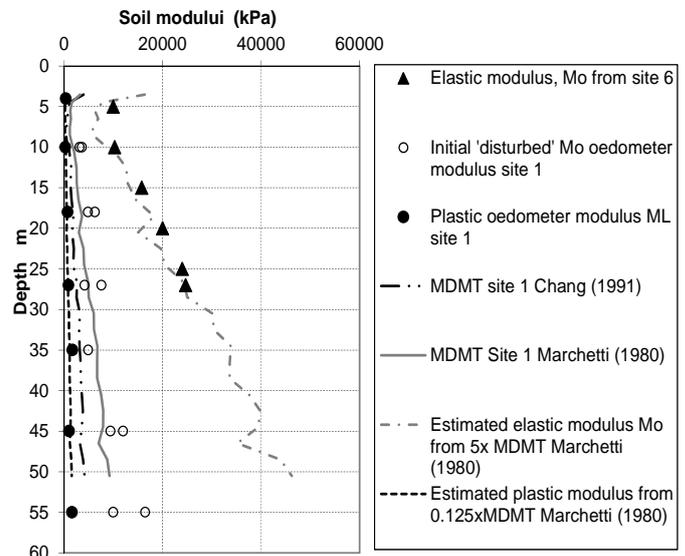
Fig. 9. Comparison of estimated oedometer moduli.

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 λ, κ, E'_{oed}, 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 υ', is known) using eq. 4:

$$E'_{DMT}=F.M_{DMT}((1+\upsilon')(1-2\upsilon'))/(1-\upsilon') \qquad (4)$$

The value of υ' is not a constant and varies during shearing. Values of υ' 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 ($\upsilon'=0.42$) 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 $\upsilon'= 0.42$ 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:

$$G_0=\rho V s^2 \qquad (5)$$

Where ρ 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.
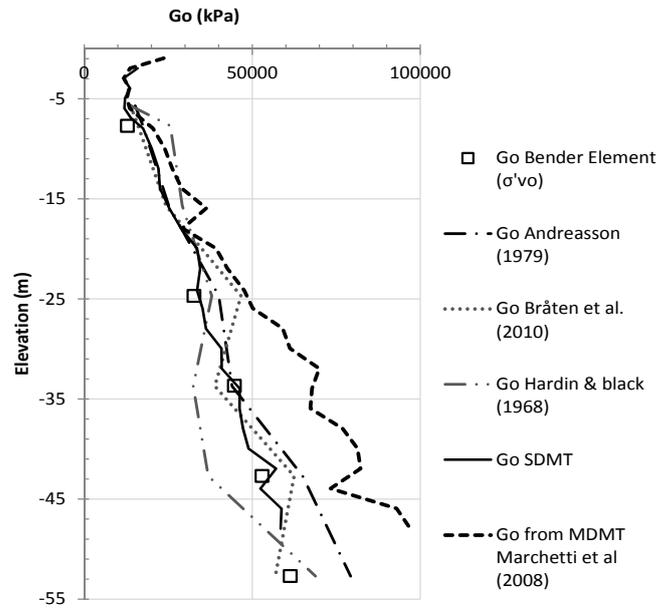
Fig. 10. Comparison of estimated small strain stiffness G_0

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.

$$G/G_0=1/(1+\gamma/\gamma r) \qquad (6)$$

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}.
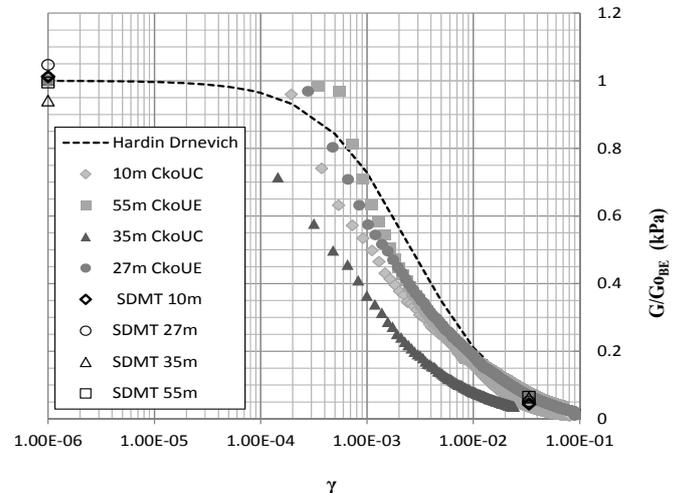
Fig.11. Shear modulus degradation curves from laboratory and in-situ measurements

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) 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

Alte, B., Olsson, T., Sällfors, G. Bergsten, H. (1989) "Djupdykning I Göteborgsleran", Chalmers University of Technology.

Andréasson, B. (1979). "Defromation characteristics of soft high –plastic clays under dynamic loading". PhD Thesis, Chalmers University of Technology.

Bergsten, H. (1991). "Late Weichselian-Holocene stratigraphy and environment conditions in the Göteborg area, south-western Sweden", PhD Thesis, Chalmers University of Technology.

Bråten, C. Døssland, Ä. L. Gjestvang, M., Kaynia, A.M. Loe, M. M., Løset, Ø. (2010). "Dimensionering for jordskjelv". Rådgivande Ingenjørers Forening.

Chang, M. F. (1991). "Interpretation of overconsolidation ratio from in situ tests in recent clay deposits in Singapore and Malaysia." *Canadian Geotechnical Journal* 28.2, 210-225.

Donohue, S., Long, M., Gavin, K., & O'Connor, P. (2004). The use of multichannel analysis of surface waves in determining Gmax for soft clay. In Proceedings 2nd. Int. Conf on Geotechnical Site Characterisation, ISC (Vol. 2, pp. 459-466).

Hardin, B. O., and Black, W.L. (1968). "Vibration Modulus of Normally Consolidated Clay," J. *J. Soil Mech. Fdns Div. Am. Sot. Civ. Engrs*, (11), 1531-1537.

Hardin, B. O., & Drnevich, V. P. (1972). Shear modulus and damping in soils. *Journal of the Soil Mechanics and Foundations Division*, 98(7), 667-692.H

Hight, D. W. (1998). "Soil characterisation: the importance of structure, anisotropy and natural variability". 38th Rankine Lecture. *Géotechnique.*

Karstunen, M. (2013) "Modelling rate-dependent behaviour of structured clays", *Proc. Installation Effects in Geotechnical Engineering*, 43-50, Taylor & Francis Group, London.

Ladd, C. C. (1977). Stress-deformation and strength characteristics, State of the Art Report. *Proc. of 9th ISFMFE, 1977, 4*, 421-494.

Landon, M. DeGroot, D., and Sheahan, T. (2007). "Non-destructive sample quality assessment of shear wave velocity", Journal of geotechnical and geo-environmental engineering, 133 (4), 424-432.

Larsson, R. (1989)."Dilatometerförsök- En in-situ metod för bestämning av lagerföljd och egenskaper i jord", SGI Information 10, Linköping.

Larsson, R. Bergdahl, U. Uriksson, L. (1985). "Evaluation of shear strength in cohesive soils with special reference to Swedish practice and experience", SGI Information 3, Linköping.

Larsson, R. (2007). "CPT test: Equipment, testing, evaluation. An in-situ method for determination of stratigraphy and properties in soil profiles", SGI, Information 15, Linköping.

Long, M, Quigley, P., O'Connor, P. (2013). "Undrained shear strength and stiffness of Irish glacial till from shear wave velocity", Ground Engineering, 46.11

Lunne, T., Berre, T. & Strandvik, S. (1997). "Sample disturbance effects in soft low plastic Norwegian clay", *Proc. Int. Symp. on Recent Developments in Soil and Pavement Mechanics*, pp. 81-102, Rotterdam : Almeida, Missouri, 259-264.

Lunne, T., S. Lacasse, and N. S. Rad. (1989). "State of the art report on in situ testing of soils." *Proc. XII ICSMFE, Rio de Janeiro*. Vol. 4.

Lutenegger, A. J. (2006). Cavity expansion model to estimate undrained shear strength in soft clay from dilatometer. In *Proceedings from the Second International Flat Dilatometer Conference* , 319-326.

Mayne, Paul W., James A. Schneider, and G. K. Martin. (1999). "Small-and large-strain soil properties from seismic flat dilatometer tests." *Proc. 2nd Int. Symp. on Pre-Failure Deformation Characteristics of Geomaterials, Torino*. Vol. 1.

Marchetti, S. (1980). "In situ tests by flat dilatometer." *Journal of the Geotechnical Engineering Division* 106.3, 299-321.

Marchetti, S., and Crapps, D. K. (1981). "Flat dilatometer manual". *Int Report of GPE Inc.*

Marchetti, S., Monaco, P., Totani, G., & Marchetti, D. (2008). "In situ tests by seismic dilatometer (SDMT)". *From research to practice in geotechnical engineering*, 180, 292-311.

Olsson, M. (2013). "On rate dependency of Gothenburg Clay",PhD, Chalmers University of Technology.

Roque, R., Janbu, N., & Senneset, K. (1988). Basic interpretation procedures of flat dilatometer tests. In *Proceedings of 1st International Symposium on Penetration Testing, Orlando, Florida*, Vol. 1.

Schmidt, B. (1966) "Earth pressures at rest related to stress history." *Canadian Geotechnical Journal* 3.4, 239-242.

Smith, M. (1989). "Dilatometer tests in soft Swedish clays", MSc Thesis. Chalmers University of Technology.

Wroth, C. P. (1984). "The interpretation of in situ soil tests." *Géotechnique* 34(4), 449-489.