Relationship between Shear-Wave Velocity and Geotechnical Parameters for Norwegian Clays

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Abstract: A database of shear-wave velocity (V_s) measurements using a variety of techniques and soil properties measured on high-quality samples for 28 Norwegian sites has been established. The purpose was to evaluate the different methods of measuring V_s, to present guidelines and correlations to assist in estimating V_s profiles in these clays in the absence of site-specific data, and to outline relationships that can be used to give first-order estimates of soil properties. It was found that consistent measurements of V_s can be obtained from a variety of techniques and that for practical engineering purposes the V_s values obtained from the different methods are similar. Surface wave techniques can be particularly useful but careful survey design is necessary and in particular the inversion process needs to be carefully controlled. Differences of about 15–20% can be obtained in the V_s values depending on the algorithm used. V_s values for Norwegian clays are consistent with well-established frameworks for other materials, based on relationships between effective stress and index parameters. Piezocone penetration testing (CPTU) can be used to give acceptable estimates of V_s and this includes techniques which utilize the CPTU data only and are independent of any index property. V_s correlates well with triaxial compression and direct simple shear derived undrained shear strength (s_u) values. There appears to be a particularly good link between V_s and preconsolidation stress (p'_c). Satisfactory relationships also exist between V_s and the tangent moduli of the clays at in situ stress (M_0) and at p'_c (M_L). **DOI: 10.1061/(ASCE)GT.1943-5606.0001645.**

Introduction

Characterization of the stress–strain behavior of soils is an integral part of many geotechnical design applications including site characterization, settlement analyses, seismic hazard analyses, site response analysis, and soil-structure interaction. The shear modulus (G) of geomaterials is highly dependent upon strain level. The small-strain shear modulus ($G_{\max}$ or G_0) is typically associated with strains on the order of $10^{-3}\%$ or less. With information of $G_{\max}$, the shear response at various level of stain can be estimated using published modulus reduction curves (i.e., $G/G_{\max}$). According to elastic theory, $G_{\max}$ may be calculated from the shear-wave velocity using the following equation:

$$G_{\max} = \rho V_s^2 \tag{1}$$

where $G_{\max}$ = shear modulus (in Pa); V_s = shear-wave velocity (in m/s); and ρ = density (in kg/m^3).

$G_{\max}$ and V_s are primarily functions of soil density, void ratio, and effective stress, with secondary influences including soil type, age, depositional environment, cementation, and stress history (e.g., Hardin and Drnevich 1972). $G_{\max}$ can be measured in the laboratory using a resonant column device or bender elements.

As suggested by Kramer (1996), although the void ratio and stress conditions can be recreated in a reconstituted specimen, other factors such as soil fabric and cementation cannot. Laboratory testing requires very high-quality, undisturbed samples, which is often a challenging and expensive task given the soft and sensitive clays of eastern Canada and Scandinavia. Additionally, laboratory tests only measure $G_{\max}$ at discrete sample locations, which may not be representative of the entire soil profile.

Unlike laboratory testing, in situ geophysical tests do not require undisturbed sampling, maintain in situ stresses during testing, and measure the response of a large volume of soil. In situ measurement of V_s has become the preferred method for estimating the small strain shear properties and has been incorporated into site classifications systems and ground motion prediction equations worldwide.

As an example, Eurocode 8 (Norsk standard, NS-EN 1998-1:2004 + NA:2008), for seismic design, requires an earthquake risk assessment to be carried out for all important structures. Sites are classified based on the V_s of the top 30 m of the soil profile (V_{s30}). In addition to site classification, V_s may be required for site-specific seismic evaluation or dynamic analysis when required by the seismic design criteria.

In this paper, a short overview of different geophysical methods for assessing V_s is initially presented. Some emphasis is placed on the use of the multichannel analysis of surface waves (MASW) technique, which has proven to be a cost effective, accurate, and efficient technique in Norwegian conditions (e.g., Long and Donohue 2010). A database of results from 28 Norwegian clay sites is then presented where V_s and soil geotechnical properties were gathered for correlation purposes. At 12 of the sites direct V_s measurements using more than one technique were available. Relationships between V_s and index properties, piezocone penetration parameters, undrained shear strength and one-dimensional (1D) compression parameters are subsequently presented and compared to existing correlations in the literature. Laboratory undrained shear strength and compression properties from high-quality, undisturbed samples (usually Sherbrooke block samples) only are used. Relationships presented herein can be then used to evaluate either V_s from a given soil property, or the way around to evaluate soil properties from V_s. The principle objective of this paper is to present guidelines for reliable estimation of V_s in Norwegian clays and to outline relationships that can be used by practicing engineers

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Note. This manuscript was submitted on March 21, 2016; approved on September 11, 2016; published online on February 13, 2017. Discussion period open until July 13, 2017; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Geotechnical and Geoenvironmental Engineering***, © ASCE, ISSN 1090-0241.

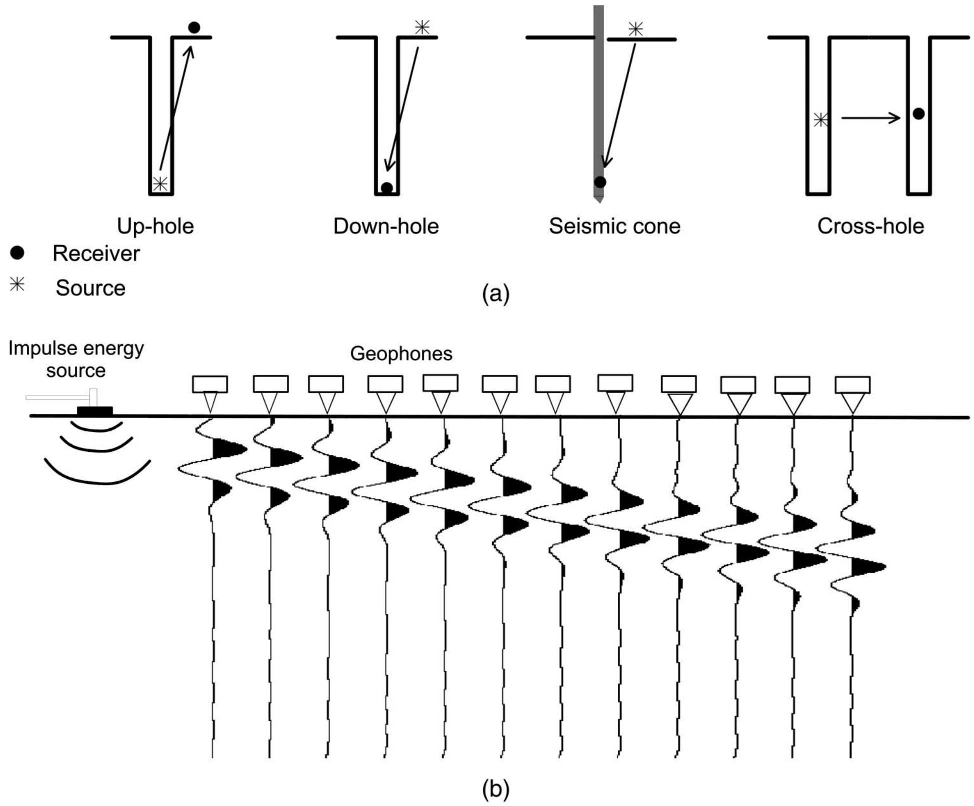

Fig. 1. Techniques for measurement of V_s (modified from Menzies and Matthews 1996, with permission): (a) invasive techniques; (b) MASW

to give first-order estimates of soil properties and for controlling the results of laboratory tests.

Techniques Used for the Measurement of Shear-Wave Velocity

Invasive Methods

Geophysical methods can be divided into two categories: invasive and noninvasive. Invasive methods require drilling into the ground. Common invasive methods include downhole logging (ASTM 2014), crosshole logging (ASTM 2014), suspension logging, seismic dilatometer (SDMT), and the seismic cone penetration test (SCPTU) [Fig. 1(a)]. In Norway, most invasive testing is done with the SCPTU but use has also been made of SDMT and crosshole tests. The SCPTU was first introduced in 1984 at the University of British Columbia (Rice 1984; Campanella et al. 1986; Robertson et al. 1986). Recent upgrades include development of continuous V_s measurement during cone penetration using a specially developed automatic seismic source (e.g., Ku et al. 2013).

In the work presented here all the SCPTU equipment had a single geophone only. The seismic signals are only recorded during pauses in penetration, commonly every 0.5 or 1.0 m. A horizontal beam coupled to the ground surface by the weight of the testing vehicle is the source of the seismic energy. The beam is struck on end with a hammer to generate horizontally polarized vertically propagating shear waves that can be detected by the horizontal receiver within the cone penetrometer embedded below. The velocity is determined from the travel-time differences between recorded waves and the difference in the assumed travel path length for receiver depth. In principle, it is advantageous and recommended to use multiple geophones, and use identical shots to determine shear wave velocities for the intervals in between the receivers. This would make the shear wave velocities less dependent on the source signature. Having multiple geophones also alleviates potential issues with inaccuracies of the target depths. The SCPTU method was used for collecting shear-wave velocity information at seven of the sites presented in the database.

The seismic dilatometer is the combination of the standard flat dilatometer (DMT) with a similar seismic module for measuring V_S as employed in the SCPTU (Marchetti et al. 2008). The crosshole test (CHT) is often considered the reference standard by which other in situ shear-wave velocity tests are compared. The tests are performed in a series of two or more cased boreholes. A borehole seismic source generates waves that propagate past receivers at the same depth in adjacent boreholes. In these tests the velocity is determined from the travel time of the waves over the distances between adjacent boreholes. A review of crosshole test procedures can be found in Hoar and Stokoe (1978) and Woods (1978). One major advantage of crosshole testing is the direct measurement through only the desired material of a particular select layer. The greatest disadvantage of CHT is the need for multiple boreholes. As a consequence, the CHT is slow, time consuming, and very expensive. In the database presented here CHT was used at five of the sites.

Noninvasive Methods

Noninvasive geophysical methods include spectral analysis of surface waves (SASW), multichannel analysis MASW, continuous surface waves (CSW), frequency wavenumber methods (f-k methods), seismic refraction, and seismic reflection. The SASW technique was developed in the early 1980s by Heisey et al. (1982) and Nazarian and Stokoe (1984). This method uses a single pair of receivers that are placed collinear with an impulsive source (e.g., a sledgehammer) and utilizes the dispersion property of surface waves for the purpose of V_s profiling. Lo Presti et al. (2003) and Soccodato (2003) compared V_s derived from SASW with that obtained from other techniques for Pisa clay and Fucino clayey soil, respectively. Reasonable agreement was found in both cases. Crice (2005) acknowledged the usefulness of SASW but suggested that solutions are neither unique nor trivial and that an expert user is required for interpretation. The SASW method was used for recording and processing of surface wave data for four sites discussed in this report.

The MASW technique was introduced in the late 1990s by the Kansas Geological Survey (KGS) (Park et al. 1999), to address the problems associated with SASW. The entire procedure for MASW usually consists of four steps [Fig. 1(b)]:

1. Acquire field records by using a multichannel recording system and a receiver array deployed over a few to a few hundred meters of distance, similar to those used in conventional seismic reflection surveys. In this study the test configuration comprised either 24 10-Hz geophones or 12 4.5-Hz geophones spaced at 3 m center over the survey length. Although KGS recommends the use of 4.5-Hz geophones on soft clay sites, it was found that they provided little advantage over the higher frequency instruments (Sauvin et al. 2016). For the 10-Hz geophones, the lower frequency level was not limited by their natural frequency, and they could detect signals as low as 5-Hz. With the 4.5-Hz geophones, the lowest recordable frequency was 2–3 Hz. A similar finding is reported by Park et al. (2002), who discuss optimum acquisition parameters for MASW surveying. An impulsive source (10-kg sledgehammer in this case) was used to generate the surface waves at the Norwegian clay sites. Seismic data were recorded using an RAS-24 seismograph (Seistronix, Rancho Cordova, California) and the corresponding *Seistronix* software.
2. Use is then made of the dispersive properties of the soil, i.e., longer wavelength signals reflect the deeper soils and shorter wavelengths represent the shallower soils to produce a phase velocity versus wavelength relationship from the measured data.
3. This phase velocity versus wavelength trace is converted into a dispersion curve (phase velocity versus frequency). Usually fundamental mode dispersion only is used.
4. The dispersion curve is inverted to obtain 1D (depth) V_s profiles (one profile from one curve). The inversion process involves the user specifying a synthetic ground profile (number of layers as well as the density, V_s, and Poisson ratio of each layer) and the software then iterates until the synthetic and field dispersion curves match. The software tools used in this study for the purpose of inversion were *Surfseis* (Park and Brohammer 2003), *winMASW* (Eliosoft), and a Norwegian Geotechnical Institute (NGI) in-house inversion code.

Advantages of the MASW method include the need for only one-shot gather and its capability of identifying and isolating noise. Also, its ability to take into full account the complicated nature of seismic waves that always contain noise waves such as unwanted higher modes of surface waves, body waves, scattered waves, traffic waves, as well as fundamental-mode surface waves. These waves may often adversely influence each other during the analysis of their dispersion properties if they are not properly accounted for. The MASW method was used for recording and processing of surface wave data for nearly all sites presented in the database (i.e., 28 out of 29).

Scaling Issues

When comparing V_s data from different methods, care needs to be taken with respect to the scale of the measurements. For example, MASW allows relatively large volumes of soil to be investigated but suffers loss in resolution with depth. However, crosshole testing allows for the detailed investigation of a particular horizon in the soil profile. Larger volumes will encompass factors such as layering and anisotropy, which are not evident in smaller-scale testing. Cercato (2009) suggests that the various methods can be considered to be complementary to one another.

Uncertainties in the MASW Method

As the MASW method was used extensively in this work it is important to consider the potential pitfalls and limitations from survey design to final interpretation of the results. Sauvin et al. (2016) have studied these issues in detail with special reference to work in Norwegian soft clays including those considered in this paper. They found that care is needed when planning field surveys and that source offset distance, geophone spacing, array length, source frequency content, and the sampling time can all influence the results. Following some careful trials of the previously mentioned parameters stable raw data with high signal to noise ratio which requires minimal preprocessing can be obtained.

The inversion technique applied is the largest source of error in the MASW method due to the inversion process and the subsequent lack in uniqueness of the V_s profile (e.g., Xia et al. 2003; Socco and Strobbia 2004; O'Neill and Matsuoka 2005; Cercato 2011; Luo et al. 2007; Foti et al. 2015). Some issues that arise include *mode jumping* in the dispersion curve especially when a steep nonlinear gradient in V_s exists near the surface. With careful surveying, mode jumping can be overcome by prior identification of situations where difficulties may arise (Boaga et al. 2014) or by varying offset distance (Cercato 2009). Cercato (2011) proposes a global inversion algorithm to help overcome these problems.

Sauvin et al. (2016) studied this issue specifically for Norwegian soft clays by employing different inversion routines to good quality data from the Esp site near Trondheim. The surface wave V_s profiles were compared to those obtained from SPTU and CH testing. Although good agreement was obtained, differences of up to 20 m/s (i.e., about 10%) were obtained from the different inversion procedures. Similarly, Sutton (1999) concluded that errors of the order of $\pm 8\%$ (≈ 10 m/s) could be obtained when comparing surface wave and other data for Bothkennar soft clay in the United Kingdom. The Bothkennar site is also included in this study. Xia et al. (2000) found an overall difference of approximately 15% when comparing MASW results with borehole measurements on unconsolidated sediments of the Fraser Delta. Similarly, Luo et al. (2007) found relative errors up to 15.9% when comparing joint inversion results to borehole results. Mulargia and Castellaro (2009) suggested an intrinsic 20% error in the field estimation of V_s is generally found. One should note that according to Eq. (1), a 20% error in estimation of V_s leads to an approximate 30% error in the estimation of the small strain shear modulus (G_{max}).

The authors are not in agreement with Crice (2005) who suggests that MASW survey data can be reliably interpreted by computer software without human intervention. The authors have found that this is only accurate for simple soil profiles. Significant user experience and intervention are required for more complex profiles. In the view of the authors an informed user is certainly important for MASW data analysis.

Test Sites and Soil Properties Included in the Database

In situ shear-wave velocity measurement has been carried out at a few Norwegian clay sites during the last decades for research

Table 1. Summary of Sites Surveyed

Number	Location	Site	Soil type	Technique	References for sites
			Southeast Norway		
1	Østfold	Onsøy	Soft clay	SCPT/MASW	Eidsmoen et al. (1985), Lunne et al. (2003)
2		Seut Bridge	Soft organic clay (quick)	MASW	APEX files, Multiconsult files
3	Akershus	Eidsvoll	Firm to stiff clay (silty)	MASW	Karlsrud et al. (1996, 2005), Karlsrud and Hernandez-Martinez (2013), and Lunne et al. (1997, 1997, 2006)
4		Hvalsdalen	Firm to stiff clay	MASW	As Eidsvoll
5		Skøyen-Asker	Very soft clay (quick)	MASW	NGI files, e.g., NGI 990032-1
6		RVII	Soft clay	MASW	Long et al. (2009), Hagberg et al. (2007)
7	Oslo	NGI car park	Soft clay	MASW/SASW	NGI files, Kaynia and Cleave (2006)
8	Buskerud-Drammen	Danviksgata/ Museum-park	Soft clay	SCPT/MASW/Raleigh/CHT	Lunne and Lacasse (1999), Eidsmoen et al. (1985), Butcher and Powell (1996), and BRE (1990)
9		Lierstranda	Soft clay	MASW/Raleigh	Lunne and Lacasse (1999), and Lunne et al (1997)
10		Hvittingfoss	Soft to firm quick clay	SW inversion (MASW)/ SCPTU/seismic reflection	Sauvin et al. (2013, 2014)
11		Smørgrav	Soft (quick) clay	MASW	Donohue et al. (2009, 2012), and Pfaffhuber et al. (2010)
12		Vålen	Soft clay	MASW	Sauvin et al. (2011)
13	Vestfold	Farriseidet	Organic quick clay	MASW	NGI files
14		Månejordet	Silty quick clay	MASW	Statens Vegvesen/UCD files
15	Telemark	Skienselven	Soft to firm quick clay	MASW	NGI files, e.g., 20011544-1, February 2003
			Mid-Norway		
16	Trondheim	Tiller	Soft to firm (quick) clay	MASW/SASW/SCPTU/CHT	Gylland et al. (2013), Sandven et al. (2004), Sandven (1990), and Takle-Eide (2015)
17		Berg	Firm clay	MASW/CHT	Rømoen (2006), Westerlund (1978)
18		Esp	Soft to firm (quick?) clay	MASW/CHT/SCPTU	Torpe (2014), King (2013), Montafia (2013), Knutsen (2014), and Hundal (2014), NGI files
19		Klett (south)	Soft silty (quick) clay	MASW/SCPTU	APEX, Multiconsult and NGI files
20		Dragvoll	Very soft quick clay	MASW, SW inversion	Montafia (2013), Pasquet et al. (2014), and Eide–Helle et al. (2015)
21		Rosten	Soft clay	MASW	NGI files
22		Saupstad	Firm to quick clay	MASW	NGI files
23		Eberg	Soft organic clay	SASW/Seismic ref.	Røsand (1986), Sandven (1990), and Langø (1991)
24		Hoseith	Quick clay (silty)	MASW	APEX, Multiconsult and Trondheim Kommune files
25		Okstad	Stiff, silty clay	MASW	As Hoseith
26	Rissa	Rein Kirke	Soft and quick clay	MASW	Sauvin et al. (2013), Aasland (2010), and Kornbrekke (2012)
27	Stjørdal	Glava	Firm clay	MASW/SASW	Sandven (1990), and Sandven and Sjursen (1998)
28	Namsos	Kattmarka	Layered soft clay	MASW	NGI and NTNU files
29	Scotland	Bothkennar	Soft clay/silt	SCPT/SDMT/MASW/CSW CHT	See *Géotechnique*, No. 2, 1992. For summary of V_s values see Long et al. (2008)

purposes and/or as a part of construction projects. Source of existing data includes Langø (1991), Long and Donohue (2007, 2010), and L'Heureux et al. (2013). In this paper, existing data are assembled and collated with field data from about 12 new sites. The additional sites were chosen based on the availability of high-quality samples and associated laboratory testing. In all the data originate from a total of 28 Norwegian sites as summarized in Table 1. Out of these sites, 15 are located in southeastern Norway and 13 are in mid-Norway (Fig. 2). A 29th site included in the database is the Bothkennar clay site in Scotland where much work has been carried out over the last 30 years (including testing of block samples by NGI) (e.g., Long et al. 2008).

The database includes index properties such as total unit weight, water content, clay content, remolded shear strength, sensitivity, and Atterberg limits. Also, engineering properties such as undrained shear strength derived from anisotropically consolidated undrained triaxial compression and extension tests (CAUC and CAUE), direct simple shear tests (DSS) and in situ vane tests, net CPTU cone resistance, in situ effective vertical stress and 1D compression parameters based on the classical Janbu theory (Janbu 1963, 1969). Full details of the database are given in NGI (2015).

The Norwegian clays in the database are of marine or glacio-marine origin. Natural water content (w) data range between 20 and 80% [Fig. 3(a)]. Most of the plasticity index data vary between

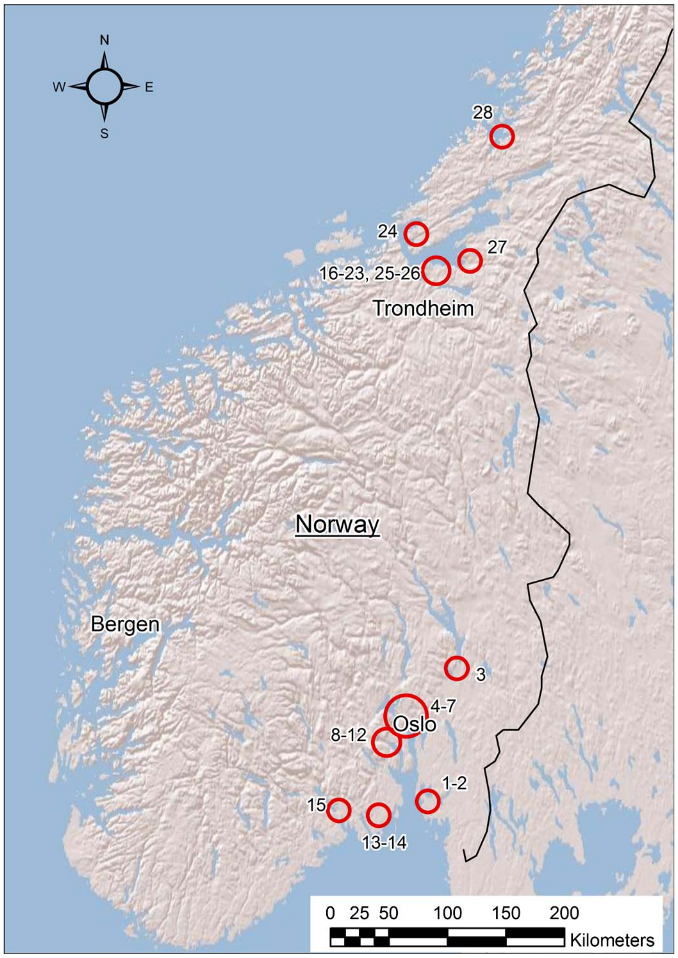

Fig. 2. Location of sites in database (reprinted from NGI 2015, with permission)

5 and 20% [Fig. 3(b)]. The clay content of the soil tested ranges from 10 to 70% with the data mainly being in the range of 30–50% [Fig. 3(c)]. Due to the isostatic uplift and resulting emergence of the marine and glaciomarine deposits during the last 10,000 years or so, fluxes of fresh groundwater through the clay deposits have led to leaching of the salts within the grain structure of the material. According to Rosenqvist (1953), such process is the main factor affecting the sensitivity of the clays. Sensitivity is defined as the ratio of the undrained peak shear strength over the remolded shear strength. In the database, the sensitivity of the clays (as measured by the Swedish fall cone) ranges between 0 and 240 with most of the data in the interval 0–20 [Fig. 3(d)].

The histogram of sample depth for the various clay samples in the database is presented in Fig. 4(a) and the corresponding vertical in situ effective vertical stress for these depths is shown in Fig. 4(b). The effective vertical stress in the database varies between 10 and 240 kPa with the highest number of observations at around 100 kPa corresponding to a depth of approximately 6–7 m below ground surface. Most of the clays have developed some apparent overconsolidation due to aging. The overconsolidation ratio (OCR) data range between 1.0 and 8 with most of the OCR data falling between 1.5 and 2.0, indicating that most of the soil samples in the database are normally consolidated to lightly over consolidated [Fig. 5(a)]. Hence, correlations developed later may not be valid for heavily overconsolidated clays. The undrained shear strength data from CAUC triaxial tests concentrates in the range 25–60 kPa, whereas results from CAUE and DSS tests are mostly below 50 kPa [Fig. 5(b)].

Validation of Data

Sites in Mid-Norway Area

To validate the data and to gain confidence in the techniques used, comparative studies were undertaken at a number of sites. Data for the three sites Tiller, Esp, and Klett, in mid-Norway are shown in Fig. 6. The sites are reasonably similar in nature with water contents (w) of 30–40%, bulk unit weight (γ) of 18–19.5 kN/m^3, clay content between 35 and 40%, and average plasticity index (I_p) of about 5%. Each site has clear zones of low sensitivity (S_t) to medium sensitivity clay and of quick clay. In Norway, according to NGF (1982), quick clay has remolded shear strength, s_{ur}, <0.5 kPa. MASW, SCPTU, and CH testing were carried out by APEX Geoservices, NGI, and NTNU, respectively, without any party being aware of the others' results. In addition SASW results, obtained by GDS Ltd., are available for Tiller. At several of the sites the MASW and/or the SCPTU measurements were repeated and there was good repeatability of the data. It can be seen from the results that all methods gave comparable results and can be considered to give equivalent V_s values for engineering design and site characterization purposes.

Sites in Southeast Norway

Although the marine clays in southeast Norway have similar depositional history and mineralogy to those in the mid-Norway area, they often have slightly higher clay content and consequently higher water content and lower bulk unit weight. Most notably they are usually of higher plasticity (e.g., Gylland et al. 2013).

Long and Donohue (2007) previously presented data similar to that shown in Fig. 5 for the Onsøy and Drammen Museumpark sites in southern Norway and concluded that for practical purposes all the methods used will give similar values of V_s. Additional data for the Hvittingfoss site in southern Norway are shown in Fig. 7. Data are taken from the work of Sauvin et al. (2013, 2014). Here MASW and SCPTU data can be compared with results from seismic reflection. Once more it can be seen the MASW data produces repeatable results and it can be seen that all the techniques give very similar values of V_s. The difference in the V_s value at any one depth is of the order of 20 m/s, which is less than the possible 20% intrinsic error suggested by Mulargia and Castellaro (20089).

Summary of All V_s Values

A summary of all the available MASW data are given in Fig. 8, with sites from southeast Norway shown in Fig. 8(a) and those from mid-Norway in Fig. 8(b). All sites show a very similar trend between V_s and depth and differ only in the value of V_s close to the surface. Teachavorasinskun and Lukkunaprasit (2004) found a similar pattern for soft Bangkok clays and they expressed the relationship in the form

$$V_{sz} = V_{sg} + mz \tag{2}$$

where $V_{sz} = V_s$ m/s at any depth z (m); $V_{sg} = V_s$ close to the ground surface (m/s); and m = slope of the line of V_s versus depth (units m/s · m).

Some exceptions are the very soft, high water content and organic clays at Onsøy and especially Farriseidet, which show much lower values of V_s.

Data for the Trondheim and mid-Norway sites can be broadly divided into two groups. The main group shows similar values to

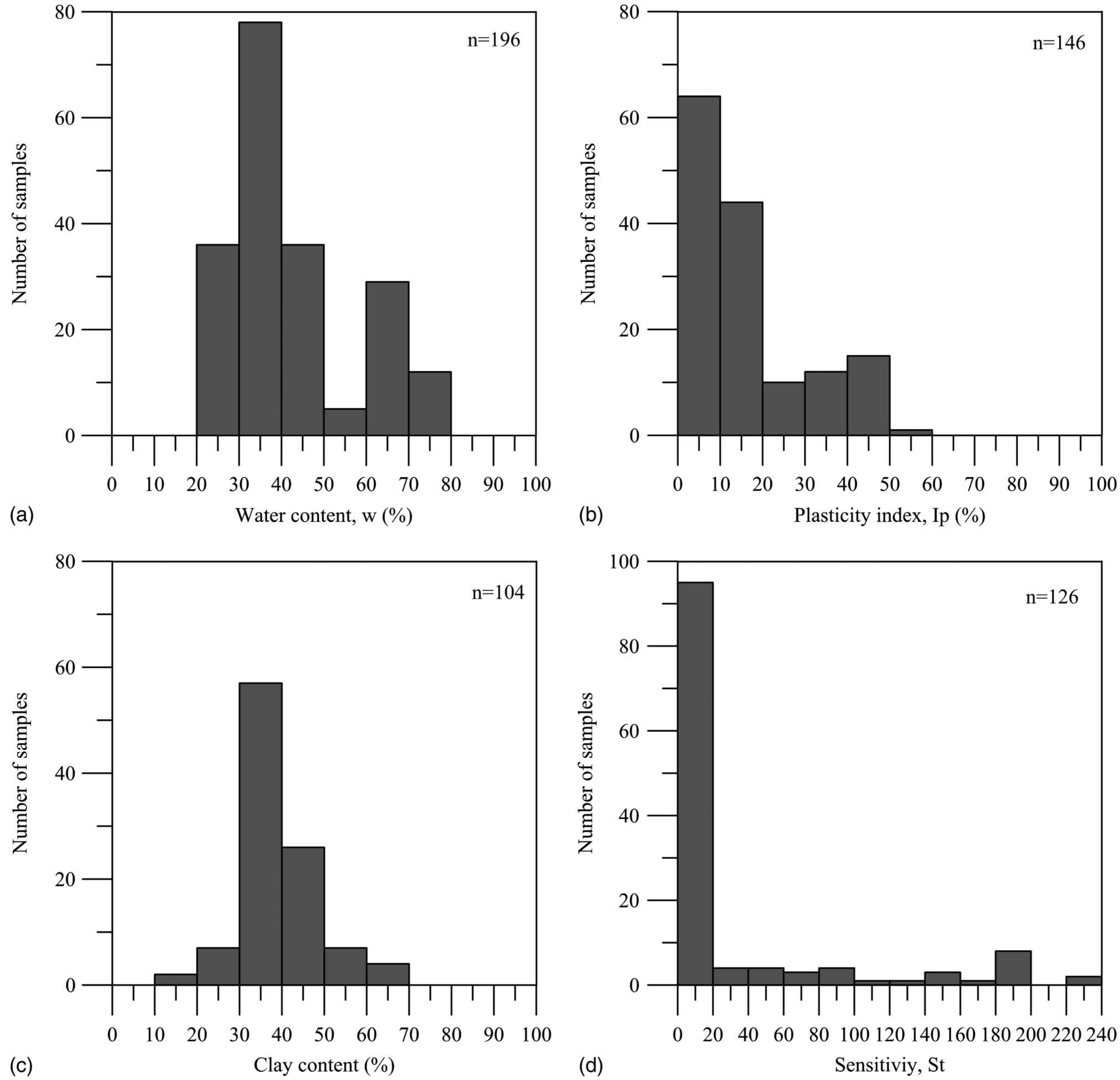

Fig. 3. Summary of soil properties from database of Norwegian clays: (a) water content; (b) plasticity index; (c) clay content; (d) sensitivity

those from southern Norway. However, there is a second group of sites all located in south and southwest Trondheim (comprising the Rosten, Saupstad, Okstad, and Hoseith sites) with higher values. All of these sites are located at the bottom of high slopes and are more overconsolidated than the other sites. The very soft clay at Dragvoll shows the lowest V_s values.

As has been shown, the V_s values deduced from the different geophysical methods (i.e., MASW, SASW, SCPTU, and CHT) at a given site generally give very similar results. For the data presented here, the results do not seem to be affected by the technique used or the directions of propagation and polarization of the waves. This is likely to be due to the largely isotropic nature of these materials. Isotropy of V_s measurement in soft clay has also been documented by Soccodato (2003). However, as pointed out by Butcher and Powell (1996) and others, V_s values measured with different techniques can be significantly different in heavily overconsolidated clays or layered soils.

Correlations with Index Parameters

Correlations between index parameters and V_s or $G_{\max}$ can provide rapid estimates useful for preliminary design and for verifying in situ and laboratory results. According to Leroueil and Hight (2003) and Hardin (1978) the empirical equation describing the influence of the controlling factors on $G_{\max}$ can then be written as follows

$$G_{\max} = SF(e)(\sigma'_v \sigma'_h)^n p_a^{(1-2n)} \tag{3}$$

where S = dimensionless parameter characterizing the considered soil; $F(e)$ = void ratio function; σ'_v and σ'_h (kPa) = vertical and horizontal effective stresses, respectively; n = parameter indicating the influence of stress; and p_a (kPa) = atmospheric pressure.

Fig. 9 presents the relationship between in situ shear-wave velocity and σ'_{v0} for samples at all sites in the database. Results show a clear tendency for V_s to increase with σ'_{v0}. The best fit

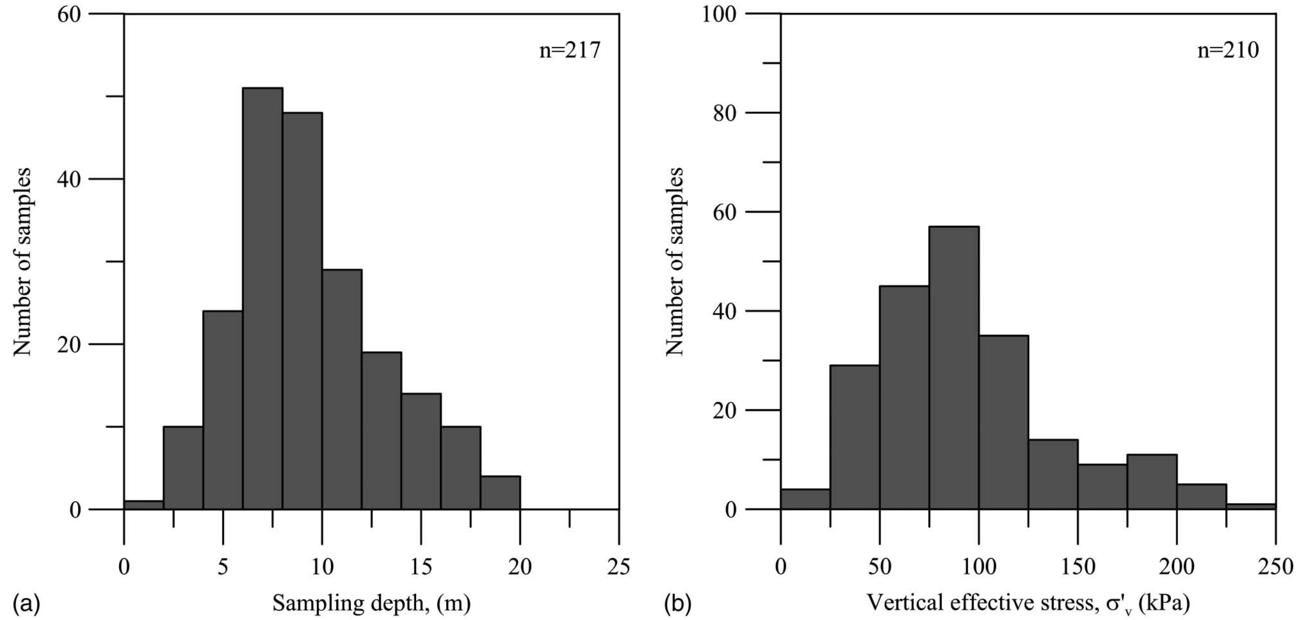

Fig. 4. (a) Sampling depth; (b) in situ vertical effective stress for samples in the database

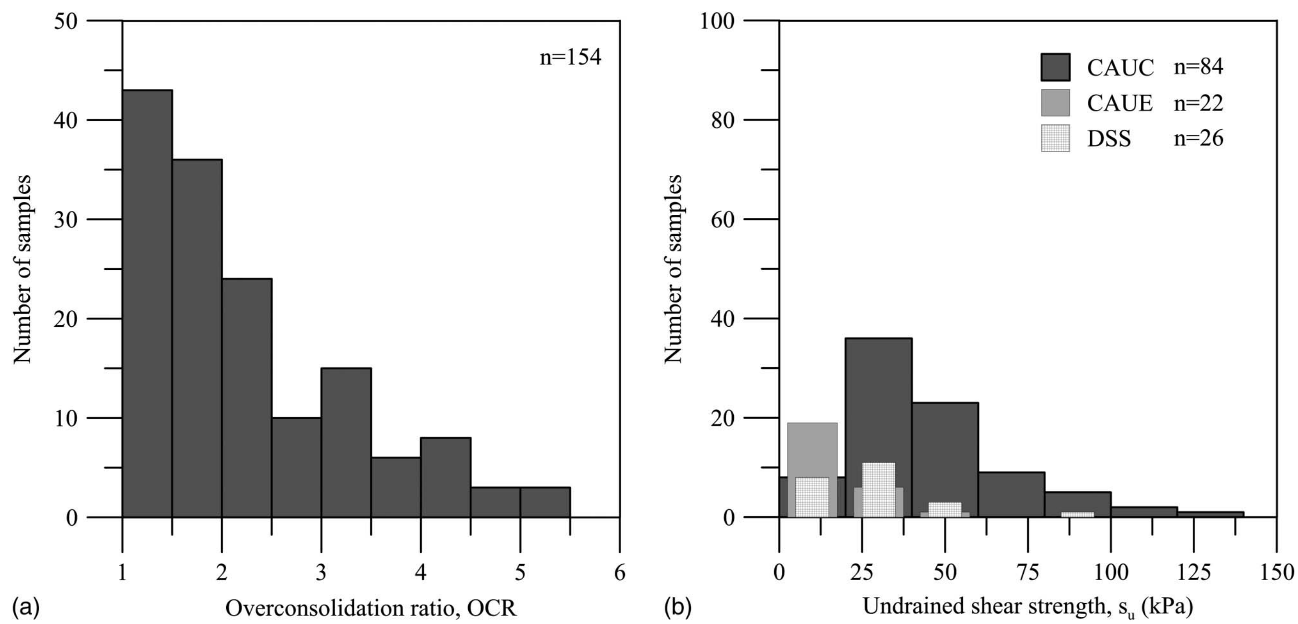

Fig. 5. (a) Overconsolidation ratio (OCR); (b) laboratory undrained shear strength for soils in the database

equation for the data gives a regression coefficient of 0.71. The linear relationship determined from the data in Fig. 9 is in the form

$$V_s = 1.11\sigma'_{v0} + 53.24 \tag{4}$$

where σ'_{v0} = vertical effective stress.

Most of the data fall within 90% of Eq. (4). The main reason for the large spread in the data is associated to uncertainties in the evaluations of σ'_{v0} in the field and to a lesser extent to intrinsic assessment of in situ V_s.

Long and Donohue (2007, 2010) and L'Heureux et al. (2013) have previously shown that the relationship described in Eq. (3) works well for Norwegian clays if S is taken to be in the range 500–700, $F(e) = 1/e^{1.3}$ (where e = void ratio), $K_0 = 0.5$ (where K_0 = coefficient of earth pressure at rest), and $n = 0.25$. Full details can be found in NGI (2015).

Norwegian practice often normalizes $G_{\max}$ with respect to the sum of the mean consolidation stress (σ'_m) and attraction (a) to obtain a dimensionless parameter that depends on friction only, e.g., Janbu (1985). This normalized small-strain shear modulus ($g_{\max}$) can be written as

$$g_{\max} = \frac{G_{\max}}{\sigma'_m + a} \tag{5}$$

$G_{\max}$ was calculated using the sample density and Eq. (1). A systematic variation of $g_{\max}$ against water content was found with

g_{max} decreasing with increasing water content, in a similar way to that proposed by Janbu (1985) for odometer moduli. Similarly, there was a reasonable correlation between g_{max} and plasticity index I_p.

The coefficients V_{sg} and m in Eq. (2) are plotted against average water content (w) and unit weight (γ) for each site (over the interval where V_s data are available) in Fig. 10. It can be seen that both parameters decrease with increasing w and increase with increasing

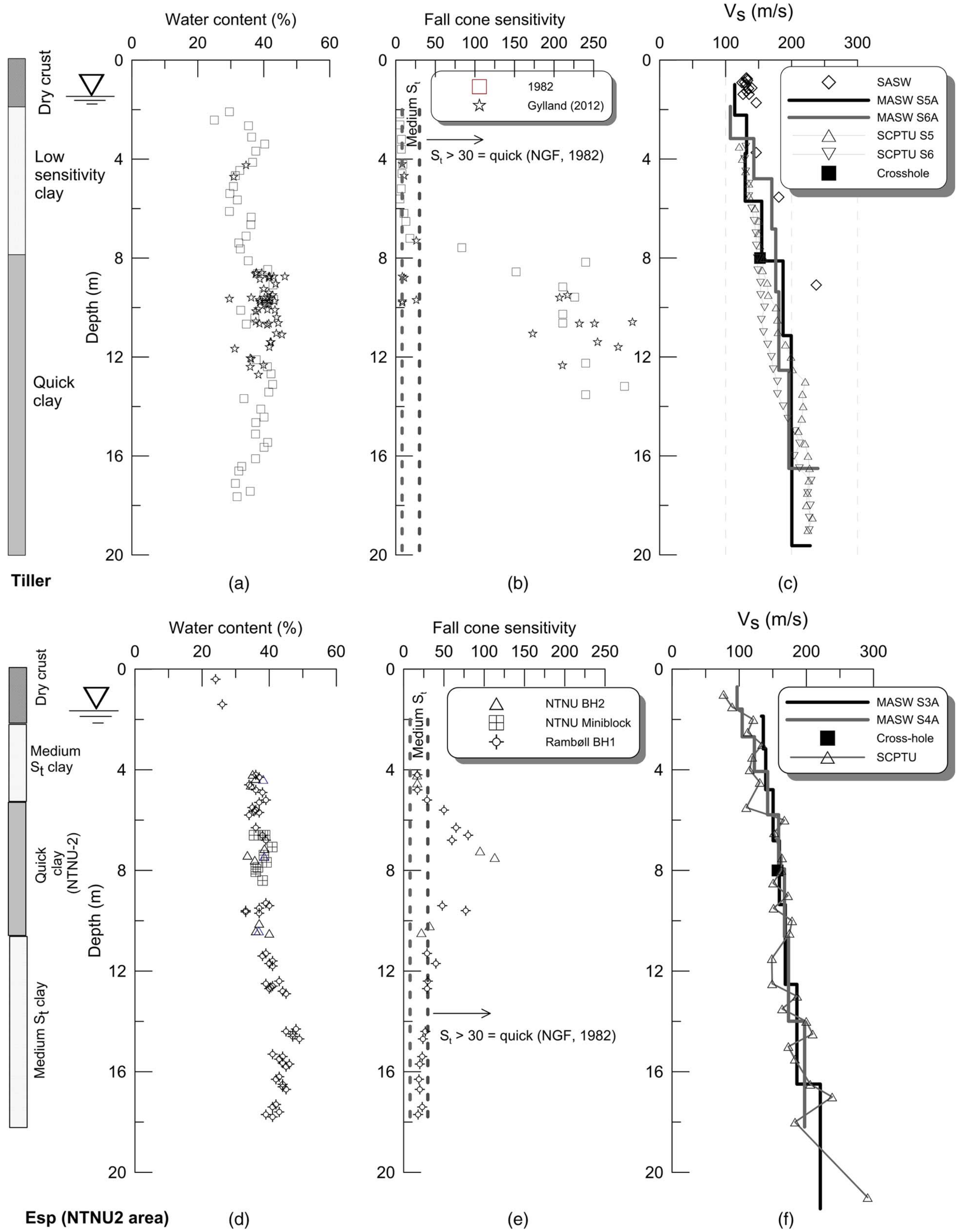

Fig. 6. Validation check for sites in the Trondheim area: (a–c) Tiller; (d–f) Esp; (g–i) Klett with each plot showing water content, sensitivity and V_s; note change in y-axis scale for Klett site

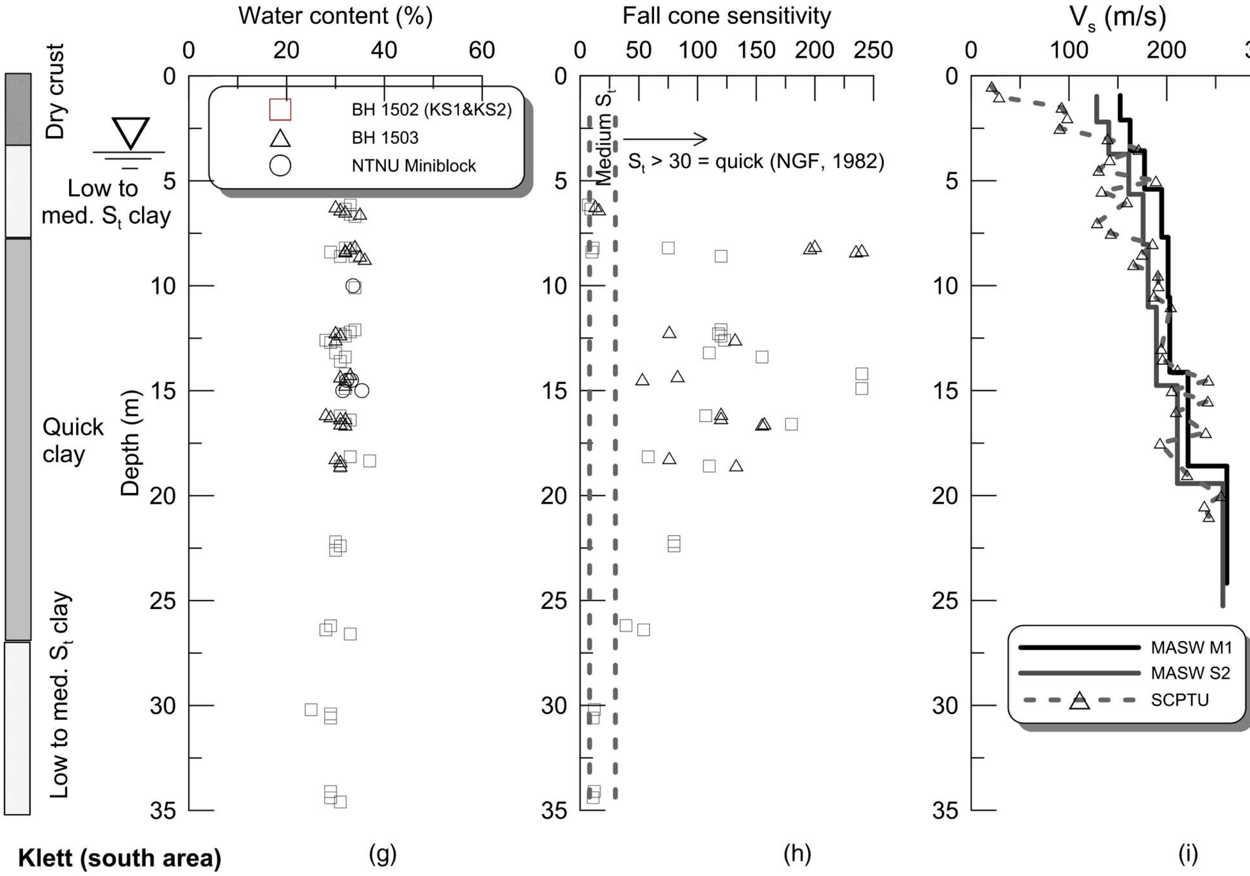

Fig. 6. (*Continued.*)

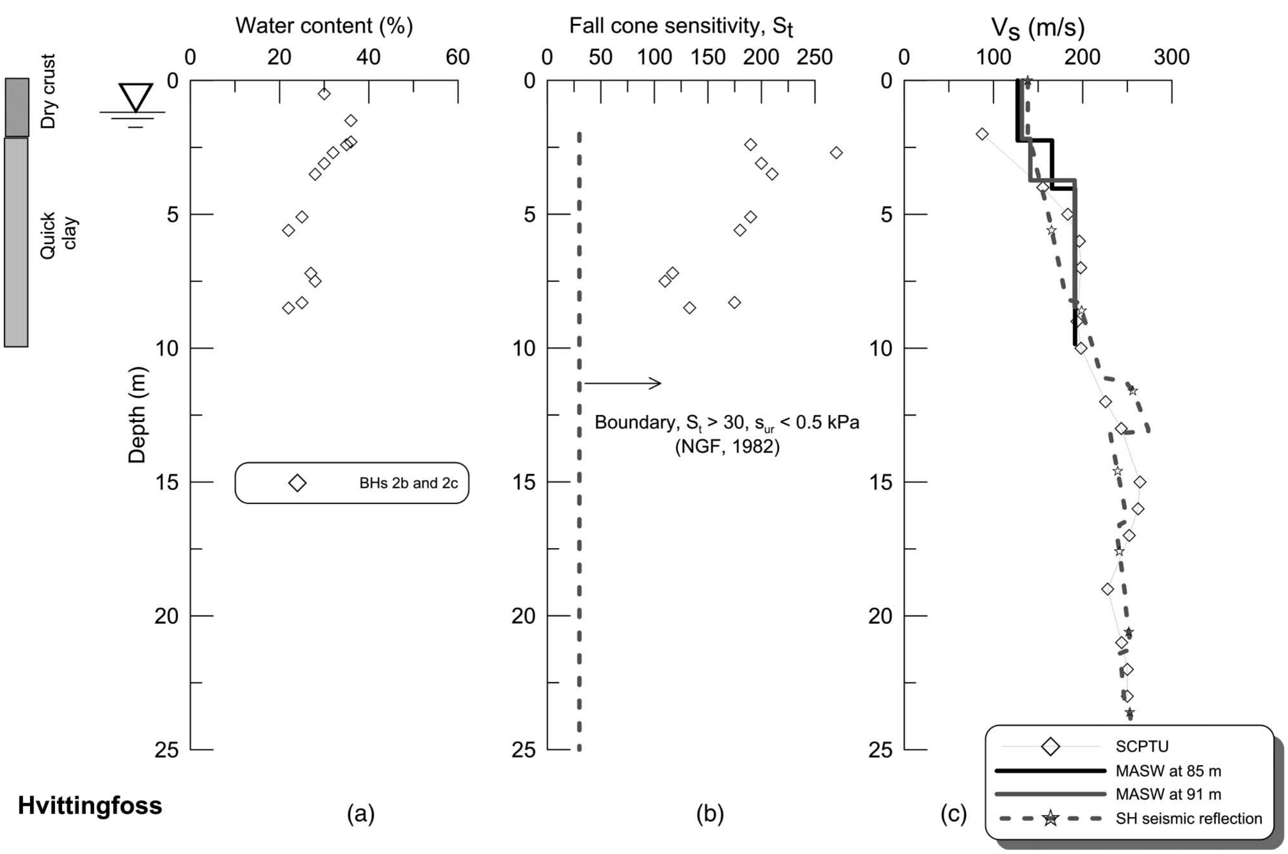

Fig. 7. Validation check for southern Norway site at Hvittingfoss with plot showing (a) water content; (b) sensitivity; (c) V_s

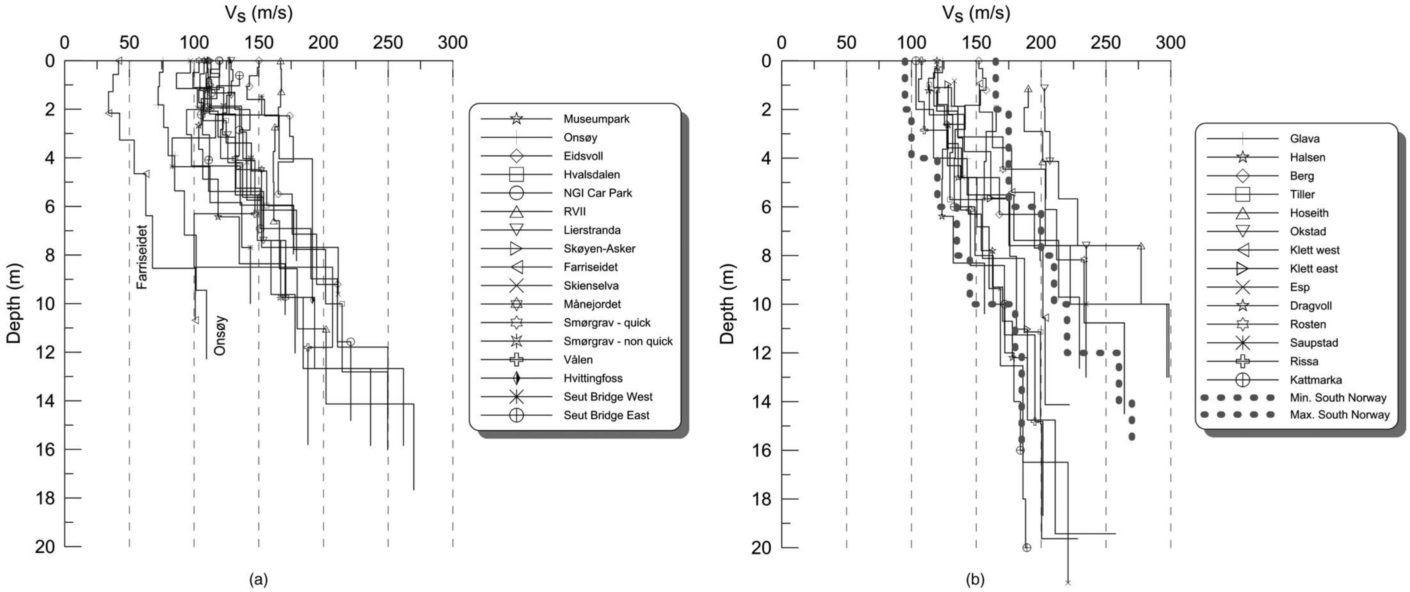

Fig. 8. Summary of all MASW data for (a) sites in southeastern Norway; (b) sites in Trondheim and mid-Norway

γ as would be expected. The trend between the parameters is reasonably good and these relationships could therefore be used for first-order estimates of V_s or for controlling site measurements. A reasonable fit would have been expected here as both w and γ are amongst the parameters, which most strongly influence V_s as has been discussed previously. Teachavorasinskun and Lukkunaprasit (2004) found that for Bangkok clay V_{sg} varied between 45 and 64 m/s (i.e., at the lower end of the values recorded here) and that m varied between 3.3 and 8.8 m/s/m. They found a correlation between decreasing m and increasing plasticity index. Attempts were also made to correlate V_{sg} and m against plasticity index for the Norwegian data. Broad trends exist but there was significant scatter in the data.

Correlations with CPTU

The piezocone (CPTU) test is widely used in Scandinavia for characterizing soft clays and there is widespread confidence amongst geotechnical engineers in its use (e.g., Lunne et al. 1997; Karlsrud et al. 2005). Therefore, it is important to relate the V_s values to the various CPTU parameters so that the two techniques can be used in a complementary fashion. Various researchers have studied relationships between CPTU parameters and V_s in clayey soils. These studies have explored relationships between in situ V_s and various parameters such as CPTU tip resistance (q_c), corrected tip resistance (q_t), cone net resistance (q_{net}), sleeve friction (f_s), pore pressure parameter (B_q), effective stress (σ'_v), water content (w), and void ratio (e).

An overview of the V_s prediction equations found in the literature for clays is presented in Table 2. For consistency, some of the equations have been modified to use of SI units: q_c, q_t, q_{net}, f_s, and (σ'_v) are in kPa and depth (D) is in meters. The number of points used to develop each correlation equation is presented as well as the coefficient of determination (R^2). Some conclusions on the range of equations available are as follows:

- Relationships of a similar form have been found to work successfully worldwide;
- Even if the form of the equation is the same from place to place it is necessary to have different factors in the equation in order to get a good fit for the local soils;

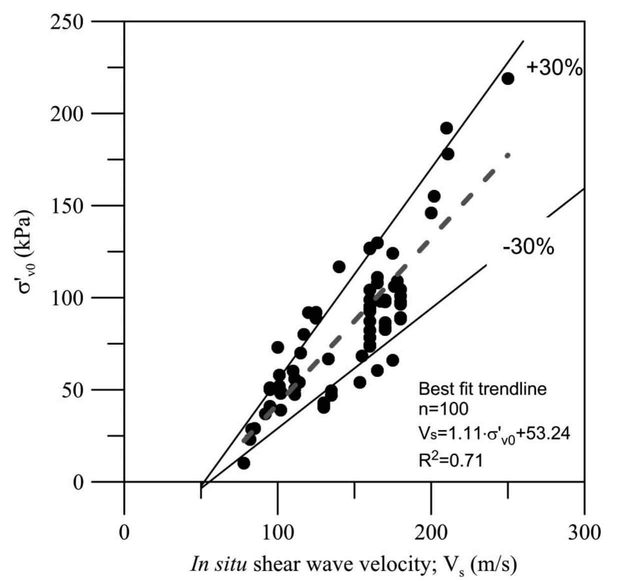

Fig. 9. In situ shear-wave velocity against vertical effective stress for all sites in the database

- An improvement in the fit of the data can be found if q_t is used instead of q_c;
- The introduction of a soil property, as measured in a laboratory test (e.g., e_0 or w) can improve the efficiency of the equation; and
- However, such an approach is then dependent on having laboratory test data as well as CPTU results and a better approach may be to use B_q instead of the soil index property.

Furthermore based on work at NGI, e.g., Powell and Lunne (2005) and on experience in Ireland and the United Kingdom, Long (2008) suggested that CPTU sleeve friction (f_s) measurements are less reliable than cone resistance (q_t), which in turn are less reliable than pore pressure (u_2). It follows then that the most reliable correlations between V_s and CPTU parameters are likely to involve q_t, q_{net}, u_2, or B_q and that use of f_s readings could be unreliable.

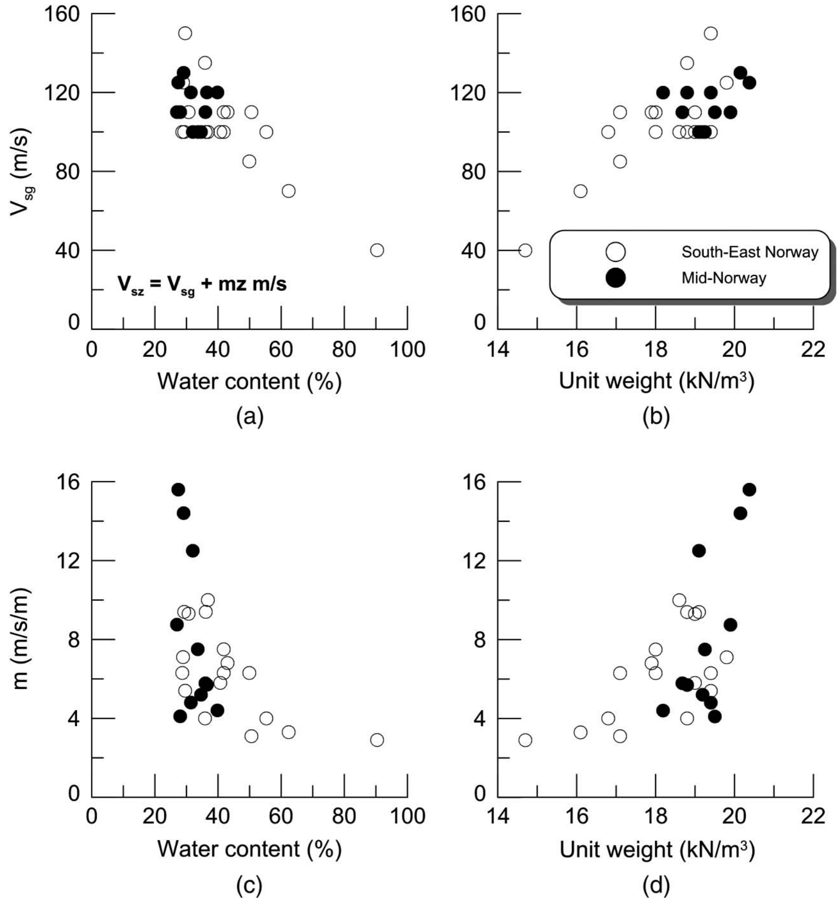

Fig. 10. Coefficients V_{sg} and m in equation $V_{sz} = V_{sg} + mz$: (a) V_{sg} against water content; (b) V_{sg} against unit weight; (c) m against water content; (d) m against unit weight

Table 2. Examples of Available CPTU-V_s Correlations for Clays

Study/reference	Clays from	Number of data pairs	R^2	V_s (m/s) or $G_{\max}$ (kPa)
Jaime and Romo (1988)	Mexico City	3 sites	—	$V_s \approx 0.1q_c$
Bouckovalas et al. (1989)	Greece	35	0.94	$G_{\max} = 2.8q_c^{1.4}$
Mayne and Rix (1993)	Worldwide	481	0.713	$G_{\max} = 2.78q_c^{1.335}$
Mayne and Rix (1993)	Worldwide	418	0.901	$G_{\max} = 406q_c^{0.695}/e_0^{1.13}$, $G_{\max} = 99.5p_a^{0.305}q_c^{0.695}/e_0^{1.13}$
Tanaka et al. (1994) and Leroueil and Hight (2003)	Japan, Canada	—	—	$G_{\max} = 50 \cdot (q_t - \sigma_{v0})$
Hegazy and Mayne (1995)	Worldwide	406	0.890	$V_s = 14.13 \cdot (q_c)^{0.359} \cdot (e_0)^{-0.473}$
Hegazy and Mayne (1995)	Worldwide	229	0.780	$V_s = 3.18 \cdot (q_c)^{0.549} \cdot (f_s)^{0.025}$
Mayne and Rix (1995)	Worldwide	339	0.830	$V_s = 9.44 \cdot (q_c)^{0.435} \cdot (e_0)^{-0.532}$
Mayne and Rix (1995)	Worldwide	481	0.740	$V_s = 1.75 \cdot (q_c)^{0.627}$
Simonini and Cola (2000)	Venice	87	0.628	$G_{\max} = 21.5q_c^{0.79}(1 + B_q)^{4.59}$
Piratheepan (2002)	United States	20	0.910	$V_s = 11.9 \cdot (q_c)^{0.269} \cdot (f_s)^{0.108} \cdot D^{0.127}$
Anagnostopoulos et al. (2003)	Greece	152	0.85	$G_{\max} = 58q_c^{1.17}$
Mayne (2006)	Worldwide	161	0.820	$V_s = 118.8 \log(f_s) + 18.5$
Long and Donohue (2010)	Norway	35	0.613	$V_s = 2.944 \cdot (q_t)^{0.613}$
Long and Donohue (2010)	Norway	35	0.758	$V_s = 65 \cdot (q_t)^{0.15} \cdot (e_0)^{-0.714}$
Long and Donohue (2010)	Norway	—	0.777	$V_s = 1.961 \cdot (q_t)^{0.579} \cdot (1 + B_q)^{1.202}$
Taboada et al. (2013)	Gulf of Mexico	274	0.94	$V_s = 14.4 \cdot (q_{\text{net}})^{0.265} \cdot (\sigma'_{v0})^{0.137}$
Taboada et al. (2013)	Gulf of Mexico	274	0.948	$V_s = 16.3 \cdot (q_{\text{net}})^{0.209} \cdot (\sigma'_{v0}/w)^{0.165}$
Cai et al. (2014)	Jiangsu, China	35 (7 sites)	0.631	$V_s = 7.95(q_t)^{0.403}$
Cai et al. (2014)	Jiangsu, China	35 (7 sites)	0.794	$V_s = 90 \cdot (q_t)^{0.101} \cdot (e_0)^{-0.663}$
Cai et al. (2014)	Jiangsu, China	35 (7 sites)	0.825	$V_s = 4.541 \cdot (q_t)^{0.487} \cdot (1 + B_q)^{0.337}$

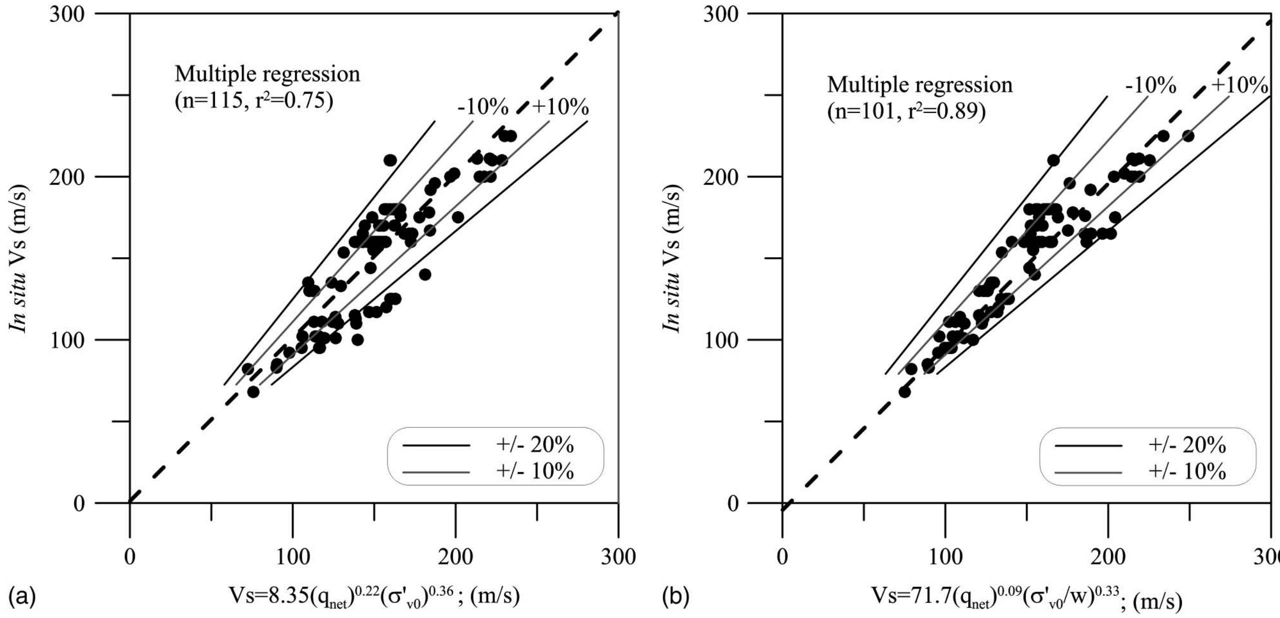

Fig. 11. Comparison of measured and predicted V_s as a function of (a) net cone resistance (q_{net}) and effective stress (σ'_{v0}); (b) net cone resistance (q_{net}) and effective stress (σ'_{v0}) normalized by water content (w)

Multiple regression analyses were conducted on the Norwegian clay database to provide power function expressions for in situ V_s in terms of q_{net}. The relationship with the highest coefficient of correlation using q_{net}, and one additional parameter was a power function similar to those listed on Table 2

$$V_s = 8.35 \cdot (q_{\text{net}})^{0.22} \cdot (\sigma'_{v0})^{0.357} \tag{6}$$

The coefficient of determination R^2 is 0.73 and a total of 115 datasets were used in the analysis [Fig. 11(a)]. The figure shows most of the predicted values of V_s are within 20% of the measured V_s. The prediction given by Eq. (6) can be improved when the water content is introduced giving rise to the following expression:

$$V_s = 71.7 \cdot (q_{\text{net}})^{0.09} \cdot \left(\frac{\sigma'_{v0}}{w}\right)^{0.33} \tag{7}$$

The coefficient of determination R^2 is 0.89 and a total of 101 datasets were used in the analyses. When using Eq. (7) most of the predicted values of V_s are within 10–15% of the measured V_s [Fig. 11(b)].

The usefulness of the equations involving q_t and e_0 and q_t and B_q, respectively, for Norwegian marine clays presented by Long and Donohue (2010) (Table 2) and Eq. (7) is investigated for all sites in the database in NGI (2015). In conclusion, it was found that

- All three equations predict V_s values that are numerically close to those measured; and
- The equations involving q_t and B_q and q_{net}, σ'_{v0}, and w better capture the profile of increased V_s with depth.

Two examples of these comparisons are shown in Fig. 12 for the Vålen site in southern Norway and the Dragvoll site in Trondheim. Neither of these sites was included in the original study by Long and Donohue (2010) and therefore they represent an independent evaluation of the approach. In addition, the Dragvoll site is underlain by unusually soft clay.

For the Vålen site a single MASW trace is available and it can be seen that this matches well with all the data derived from the CPTU test. Although all three CPTU techniques give similar results it seems that the approaches q_t and B_q and q_{net}, σ'_{v0} and w better capture the profile of increased V_s with depth. For Dragvoll two MASW tests and an independent surface wave inversion procedure by Pasquet et al. (2014) give similar results and in this case all three CPTU based methods match well with the measured data. It would seem then that satisfactory predictions of V_s can be made from independent CPTU data for Norwegian clays and that V_s and CPTU data can be used as a cross check of one another.

Correlations with Undrained Shear Strength

As discussed previously, $G_{\max}$ and V_s of cohesive soils primarily depend on void ratio, effective stress, and stress history. Therefore $G_{\max}$ (or V_s) has been frequently related to undrained shear strength (s_u) since both properties depend on common parameters.

An overview of some $G_{\max}$ (or V_s) relationships with s_u for clays used in Scandinavia and internationally is presented in Table 3. Many of the expressions summarized on Table 3 are of the same format with different coefficients. This is largely due to the fact that the value of s_u depends on the testing method used. It is therefore important to recognize the origin of the data from which such conclusions are made. This is especially true for low-plastic clays where it can be difficult to obtain consistent values of s_u

Many of the relationships follow the same format, i.e.

$$V_s = a s_u^b \tag{8}$$

In Norway it is common practice to carry out triaxial testing after the sample has been first consolidated anisotropically to the best estimate of its in situ stress. Shearing can subsequently be by compression (CAUC tests) or by extension (CAUE tests). The s_u values obtained from CAUC and CAUE triaxial tests on high-quality samples of Norwegian clay are plotted against in situ shear-wave velocity in Figs. 13(a and b), respectively. In both cases the results show an increase in s_u with increasing V_s. For the CAUC tests there is a good relationship between the two sets of data. The best fit relationship is given by the following equation, which is generally of the same format as suggested for soft clays worldwide [Table 3 and Eq. (8)]. This equation can also be used to

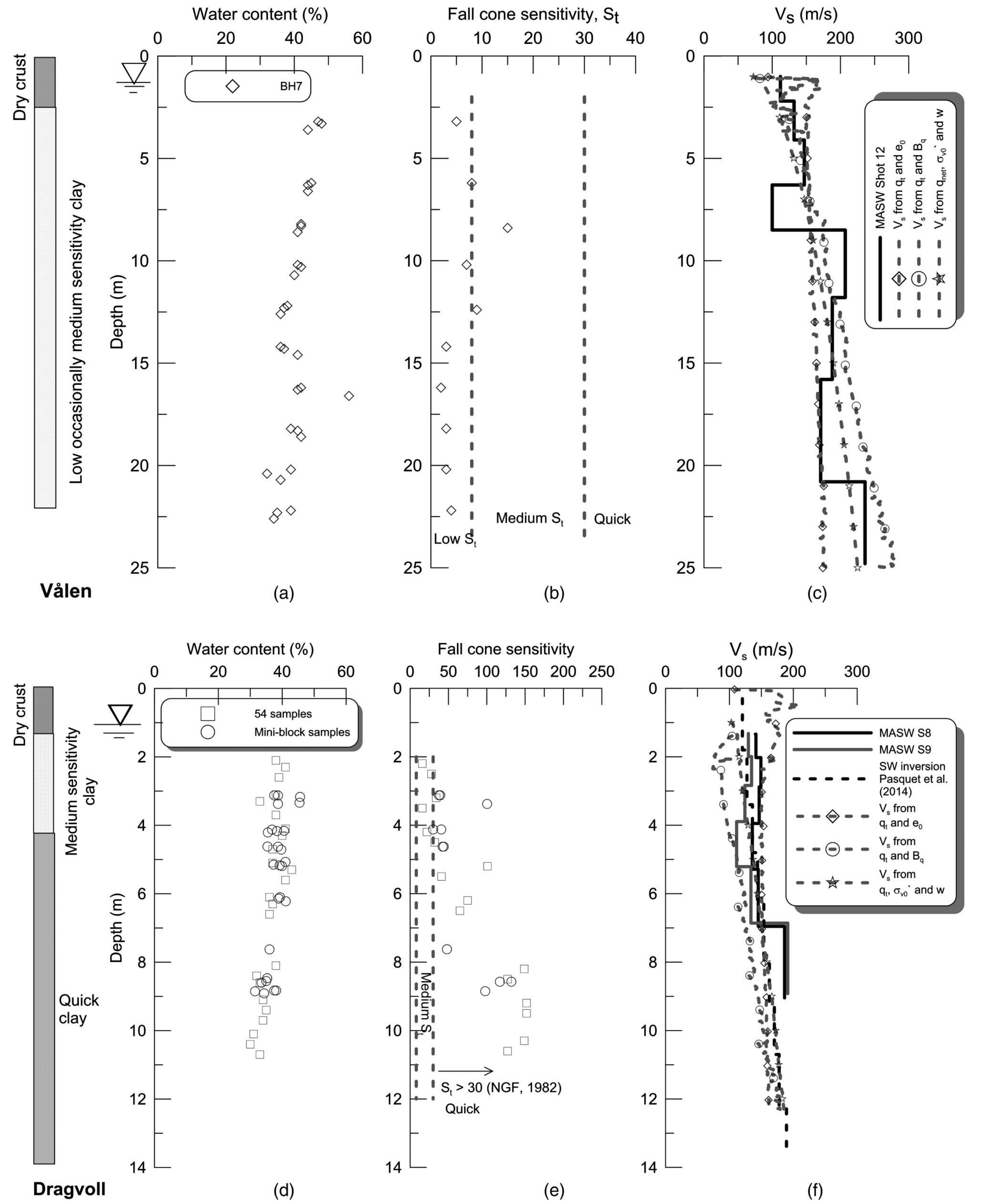

Fig. 12. Comparison between measured and predicted V_s for (a–c) Vålen site in southern Norway; and (d–f) Dragvoll site in Trondheim with each plot showing water content, sensitivity, and V_s; note change in x-axis scale for S_t at Vålen

assess undrained shear strength from V_s measurements by rewriting the relationships and solving for s_u as follows:

$$V_s = 12.72 s_{u,\mathrm{CAUC}}^{0.66} \quad \text{or} \quad s_{u,\mathrm{CAUC}} = 0.021 V_s^{1.52} \quad \text{with} \quad R^2 = 0.85 \tag{9}$$

For the CAUE tests the best fit relationship gives R^2 of 0.6, which is not considered sufficiently high for practical use of the equation. For both s_u (CAUC) and s_u (CAUE) the scatter in the data increases for increasing V_s and the greatest variation is for the highly overconsolidated Eidsvoll and Hvalsdalen clays.

Table 3. Examples of Available Correlations between the Undrained Shear Strength of Clays and V_s or G_{max}

Study/reference	Type of clays	V_s (m/s) or G_{max} (kPa)	s_u determined from
Larsson and Mulabdic (1991)	Swedish (10) and Norwegian (4) sites. Medium-high plasticity	$G_{max} = \left(\frac{208}{I_p} + 250\right) s_u$	Unspecified
Larsson and Mulabdic (1991)	Swedish (10) and Norwegian (4) sites. Low-plastic clays to high-plastic clayey organic soils	$G_{max} = 504 \cdot s_u/w_L$	Unspecified
Dickenson (1994)	San Francisco Bay clay	$V_s = 23 s_u^{0.475}$	Fall cone tests
Blake and Gilbert (1997)	Offshore NW United States (55 tests)	$s_u = 1.87 V_s^{1.12}$	Triaxial
Ashford et al. (1997)	Bangkok clays (13 sites)	$V_s = 23 s_u^{0.475}$	Unspecified
Likitlersuang and Kyaw (2010) and Likitlersuang et al. (2013)	Bangkok clays (3 sites) based on downhole and MASW, respectively	$V_s = 187(\frac{s_u}{p_a})^{0.372}$, $V_s = 228(\frac{s_u}{p_a})^{0.510}$	Unspecified
Andersen (2004)	Normally consolidated clays	$\frac{G_{max}}{s_u^{DSS}} = 325 + 55/(\frac{I_p}{100})^2$	DSS
Andersen (2004)	Sensitive and quick clays (remolded strength; $s_{ur} < 0.5$ kPa)	$\frac{G_{max}}{s_u^{DSS}} = 800$ to 900	DSS
Yun et al. (2006)	Gulf of Mexico (38 tests)	$V_s = 19.4 s_u^{0.36}$	Unspecified
Kulkarni et al. (2010)	Indian coastal soils (130 tests, $R^2 = 0.82$)	$s_u = 5 \times 10^{-4} V_s^{2.5}$	Unconsolidated undrained triaxial
Taboada et al. (2013)	Bay of Campeche clay	$V_s = 31 s_u^{0.414}$	Unconsolidated undrained triaxial and in situ vane tests
Baxter et al. (2015), Baffer (2013)	Gulf of Mexico clay, Presumpscot clay (Gulf of Maine), and organic silt	Follows same relationship with I_p as proposed by Andersen (2004)	DSS
Agaiby and Mayne (2015)	Worldwide soils (360 tests, $R^2 = 0.76$)	$s_u = 0.152 V_s^{1.142}$	Triaxial compression
Agaiby and Mayne (2015)	Worldwide soils (362 tests, $R^2 = 0.87$	$s_u = 0.038 V_s^{1.063} I_p^{0.14} \mathrm{OCR}^{0.31} e_0^{0.07} \sigma_{v0}'^{0.23}$	Triaxial compression
Andersen (2015)	Worldwide soils I_p in range 10–100%	$\frac{G_{max}}{s_u^{DSS}} = \left[30 + \frac{300}{(\frac{I_p}{100}+0.03)}\right] \mathrm{OCR}^{-0.25}$	DSS

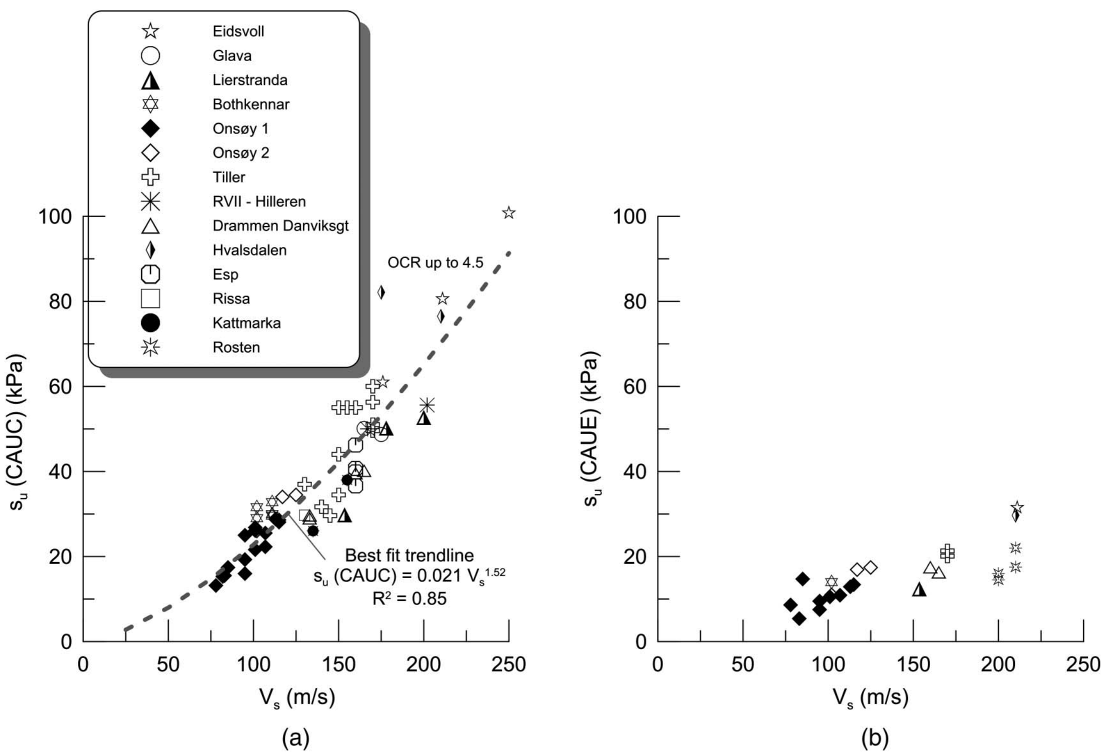

Fig. 13. V_s versus s_u from (a) CAUC triaxial tests; (b) CAUE triaxial tests on high-quality samples

A similar plot for V_s against undrained shear strength from DSS is presented in Fig. 14(a). There seems to be a particularly good fit between V_s and s_{uDSS}. Perhaps this is not surprising given that the mode of deformation is the same in the two sets of tests. The best fit relationship is given by

$$V_s = 13.32 s_{u,\mathrm{DSS}}^{0.72} \quad \text{or} \quad s_{u,\mathrm{DSS}} = 0.027 V_s^{1.39} \quad \text{with} \quad R^2 = 0.87 \tag{10}$$

The data in Fig. 14(a) are compared to the relationships proposed by Andersen (2004) (i.e., $G_{max}/s_{u,DSS} = 800$–900 (Table 3). This latter relationship is that currently used in Norwegian design practice for choice of G_{max} based on DSS test results. To compare with the relationships proposed by Andersen (2004) density has been assumed to vary between 1.6 and 1.9 $\mathrm{Mg/m^3}$ and the empirical factor to vary between 800 and 900. Fig. 14 shows the two extreme lines from the Andersen (2004) relationship. The fit is

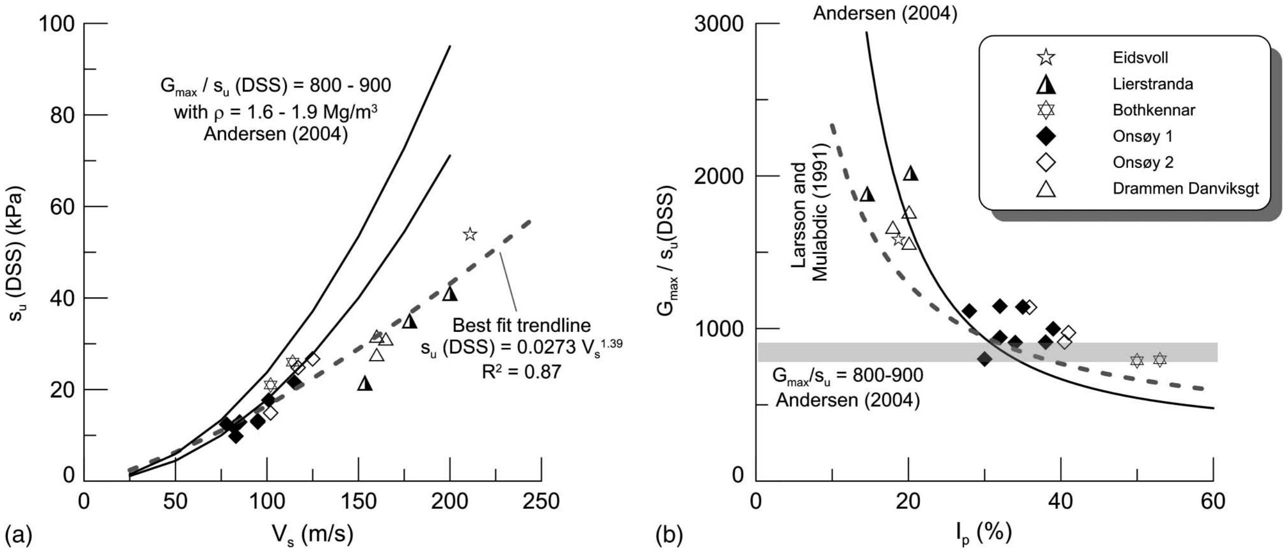

Fig. 14. (a) V_s versus s_u from DSS tests; (b) $G_{\max}/s_u$ (DSS) versus I_p

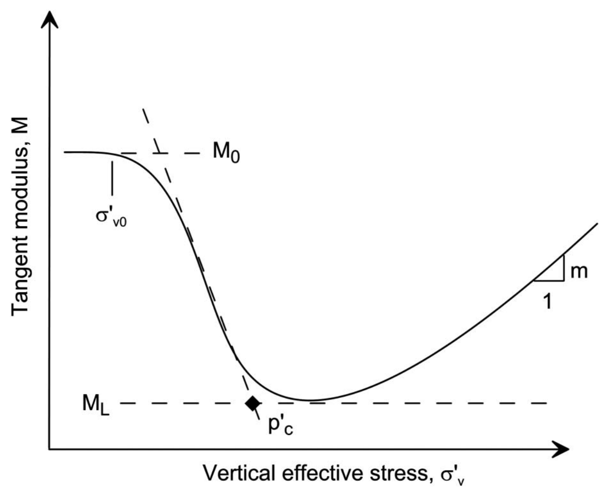

Fig. 15. NGI interpretation of classical Janbu tangent modulus versus stress model

good at low V_s value, but large difference arise for higher V_s results. The reason for these differences may come from the fact that the relationships proposed by Andersen (2004) are based on laboratory measurements of V_s and $G_{\max}$, whereas in situ V_s data are used in this study. It would seem that current Norwegian practice for a choice of $G_{\max}$ based on s_{uDSS} is conservative but there is great potential for optimization of the approach.

Similarly the ratio of $G_{\max}$ (determined from V_s) and s_u (DSS) is plotted against I_p in Fig. 14(b) and is compared to the relationships proposed by Andersen (2004) and Larsson and Mulabdić (1991) (Table 3). The fit between the data and the two relationships is reasonable if a little conservative for I_p greater than 30%.

Correlations with 1D Compression Parameters

In this section in situ shear-wave velocity measurements are compared to the classical 1D compression parameters published by Janbu (1963, 1969). The classical Janbu plot of 1D compression stiffness against stress is shown in Fig. 15. Janbu (1963) used the resistance concept to interpret 1D consolidation in an odometer test. He defined the tangent modulus (or the constrained modulus), M, as the ratio of the change in stress ($\delta\sigma'$) to the change in strain ($\delta\varepsilon$) for a particular load in increment, i.e.

$$M = \frac{d\sigma'}{d\varepsilon} \tag{11}$$

For a low stress level, around the in situ vertical effective stress (σ'_{v0}), the resistance against deformation (M_0) is large. When the stress increases this high resistance decreases appreciably owing to partial collapse of the grain skeleton. Resistance reaches a minimum (M_n) around the preconsolidation stress (p'_c). Subsequently when the effective stress is increased beyond p'_c the resistance increases linearly with increasing effective stress. In the overconsolidated range M_1 (the average between M_0 and M_n) is often used in design. The minimum value of the tangent modulus is M_L. The ratio between M_0 and M_L was proposed by Karlsrud and Hernandez-Martinez (2013) as an index for assessing sample disturbance in soft clays.

Behavior in the normal consolidation stress range can be approximated by a linear odometer modulus M. Hence, for $\sigma' > p'_c$

$$M = m(\sigma' - \sigma'_r) \tag{12}$$

where m = modulus number and σ'_r = intercept on the σ' axis and is the reference stress.

Here odometer test data were obtained from tests on high-quality Sherbrooke block samples or miniblock samples only was used. The relationship between M_0 and M_L and V_s is shown in Fig. 16. Correlations would be expected here as V_s is a function of the current state of stress. There is a clear trend of both M_0 and M_L increase with increasing V_s as expected. The scatter in the data increases for increasing V_s and the greatest variation is for the highly overconsolidated Eidsvoll and Hvalsdalen clay. The best-fit power trend lines shown give reasonable R^2 values of 0.78 and 0.8 for M_0 and M_L, respectively. A similar relationship for M_1 gives R^2 of 0.69.

Values of the preconsolidation stress (p'_c as determined by the Janbu procedure) are plotted against V_s in Fig. 17. Again a reasonable correlation would be expected here as the shear-wave velocity is strongly dependent on the maximum past stress experienced by the clay. The relationship between p'_c and V_s is satisfactory and the

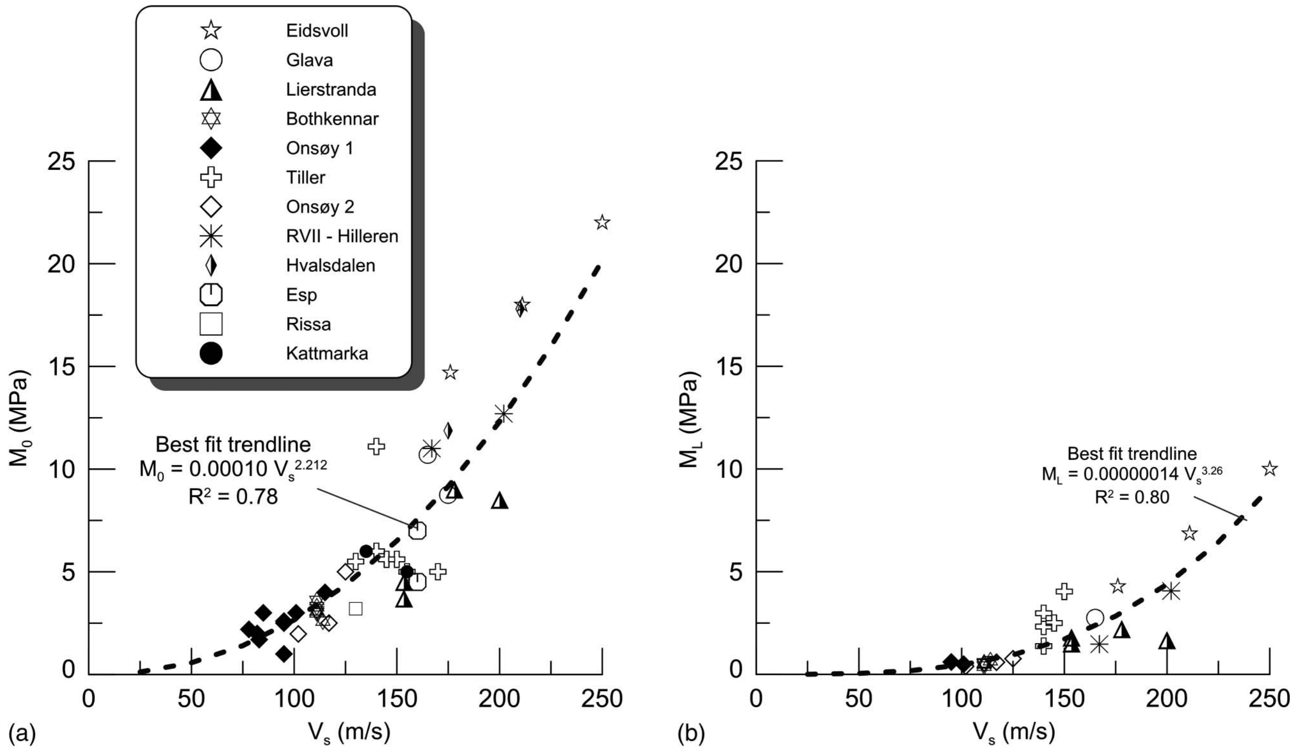

Fig. 16. Janbu tangent moduli: (a) M_0; (b) M_L versus V_s

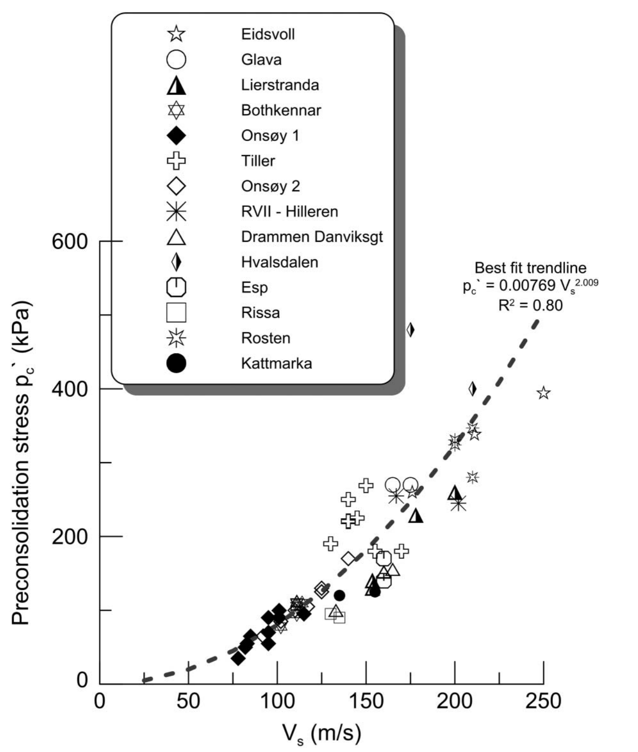

Fig. 17. Preconsolidation stress (p_c') versus V_s

best-fit power function has an R^2 value of 0.8. This is an important finding given the sensitivity of settlement calculations to the p_c' value. However, the fit is not good for OCR.

The variation in the modulus number m versus shear-wave velocity has also been explored (NGI 2015). There is a clear tendency for an increase in m with increasing V_s. However, the fit is not as good for M_0, M_1, and p_c'. This is not surprising as one would expect V_s to represent the current state of stress not at some arbitrary higher stress stiffness.

Conclusions

The purpose of this study was to present guidelines and correlations to assist geotechnical engineers in estimating V_s profiles in Norwegian clays in the absence of site-specific data. Additionally, the study aimed to highlight relationships that can be used by practicing engineers to give first-order estimates of soil properties. To achieve this, a database of in situ V_s measurements and standard geotechnical engineering material properties for Norwegian clays from 28 sites has been established. Data from high-quality Sherbrooke block or miniblock samples only were used. It was found that

- Reliable measurements of V_s can be obtained from a variety of techniques such as SCPTU, downhole tests or surface wave (principally MASW) testing. Intrinsic differences of the order of 20% or less can be expected between the various methods;
- For surface wave testing, survey design needs to be carried out carefully on a site-by-site basis and the inversion process needs to be carefully controlled;
- There are some small differences between the clays from southern and eastern Norway and from mid-Norway. However V_s values show similar trend with depth but differ mainly by the value of V_s at the surface;
- The link between the V_s measurements and index data for the Norwegian clays fit well with established relationships for clays worldwide;
- CPTU can be used to give reliable estimates of V_s in Norwegian clays. Relationships that involve the input of an index property such as the water content (w) and the in situ effective stress

(σ'_{v0}), or which rely on the CPTU-measured data only (q_t and B_q) both work well;

- V_s correlates satisfactorily with CAUC and DSS derived s_u values. These relationships can be used either to evaluate V_s from a given soil property, or the way around to evaluate soil properties from V_s; and
- There appears to be a good link between V_s and preconsolidation stress (p'_c). Useful relationships also exist between V_s and the tangent moduli M_0 and M_1.

As there is an intrinsic uncertainty associated with all geophysical techniques, it is recommended that engineers consider all available data including available relationships, in situ measured V_s profiles, and site-specific geotechnical data. The use of correlations in geotechnical engineering should be limited to the conditions for which they were developed and calibrated.

Acknowledgments

This work is funded by the Norwegian Geotechnical Society (NGF) through NGF stipend 2014-2015 and through the NFR strategic research project SP8-GEODIP at NGI. The authors would like to thank their many colleagues at NGI, NTNU, Multiconsult, SVV, APEX, and UCD for help in assembling the large dataset. The authors gratefully acknowledge their colleagues' assistance during field-data acquisition as well as their numerous helpful discussions.

Notation

The following symbols are used in this paper:

A = attraction;
B_q = piezocone pore water pressure coefficient = $(u_2 - u_0)/q_{\mathrm{net}}$;
e/e_0 = void ratio/initial void ratio;
f_s = piezocone sleeve friction;
$G_{\max}$ = small strain shear modulus;
$g_{\max}$ = normalized small strain shear modulus;
I_p = plasticity index;
M = constrained modulus in odometer test = $\delta\sigma'_v/\delta\varepsilon$;
m = modulus number;
OCR = overconsolidation ratio;
p_a = atmospheric pressure/reference stress;
p'_c = preconsolidation pressure;
q_t = corrected piezocone cone end resistance;
q_{net} = piezocone net end resistance = $q_t - \sigma_{v0}$;
S_t = sensitivity;
s_u = undrained shear strength;
s_{ur} = remolded undrained shear strength;
u = pore pressure;
u_0 = in situ pore water pressure;
u_2 = pore pressure measured by piezocone;
V_s = shear-wave velocity;
w = natural water content;
γ_b = bulk unit weight;
ρ = density;
σ'_a = axial effective stress in triaxial test;
σ'_d = deviator stress = $\sigma'_a - \sigma'_r$;
σ'_h = horizontal effective stress;
σ'_m = mean effective stress;
σ'_v = vertical effective stress; and
σ'_{v0} = in situ vertical effective stress.

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