Geotechnical and Geophysical Site Characterisation 5 – Lehane, Acosta-Martínez & Kelly (Eds)

Use of shear wave velocity to estimate stress history and undrained shear strength of clays

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ABSTRACT: A quality database of downhole shear wave velocity profiles (V_{sVH}) from 64 worldwide well-documented clay sites has been compiled to provide a first-order estimate of stress history of clays. The database primarily includes soft-to-firm normally consolidated (NC) to lightly overconsolidated where OCR < 2, as well as some stiff to hard overconsolidated (OC) and fissured clays. The stress history is expressed in terms of effective yield or preconsolidation stress (σ_p') which was measured in the laboratory by oedometer and/or consolidation tests on undisturbed samples. The in-situ V_{sVH} were measured by downhole tests (DHT), mostly using seismic piezocones (SCPTu) or seismic dilatometers (SDMT). The estimated stress history is applied to provide an evaluation of undrained shear strength (s_u) of intact clays using a SHANSEP type approach which considers different shearing modes. The suggested approach can be used as an independent estimate of stress history and undrained shear strength of clays reliant on only the in-situ measured shear wave velocity.

1 INTRODUCTION

1.1 *Stress History*

The stress history of a soil can be used to delineate the geological processes which have occurred over the many years and can be considered the focal point for soil behavior in terms of strength, flow, and compressibility. The preconsolidation stress (σ_p') can be defined as the maximum effective overburden stress experienced by the soil during its stress history. The overconsolidation ratio (OCR) is a convenient normalized and dimensionless parameter based on σ_p' and current effective vertical stress (σ_{v0}') such that: OCR = σ_p'/σ_{v0}'. More recently, the terms yield stress (σ_{vy}') and yield stress ratio (YSR = $\sigma_{vy}'/\sigma_{vo}'$) are used to also include effects of diagenesis, bonding, and ageing.

1.2 *Stress History Evaluation Methods*

The most basic and conventional means to determine stress history is a laboratory one-dimensional consolidation test using a consolidometer (ASTM D2435). The specimen is subjected to constrained compression in either a mechanical oedometer, pneumatic or hydraulic consolidometer, or automated constant rate of strain (CRS) device. On the basis of the oedometer test, many methods have been proposed to evaluate σ_p' from the compression measurements. However, the results are dependent on the plotting methods and curve-fitting procedures. Casagrande analyzed the stress history and established the first and most common method to obtain σ_p' using a graphical method. Afterwards, others attempted to improve and develop new methods in order to determine the preconsolidation stress more definitively such as the Schmertmann (1955) reconstruction method, Janbu (1969) stress-strain and modulus-strain method, Butterfield (1979) approach with a bilogarithmic representation, and Becker et al. (1987) work-energy method. At least 28 methods are available for these purposes (Ku and Mayne 2013).

1.3 *Stress History Evaluation Problems*

Laboratory based techniques are associated with many issues: disturbance which can be attributed to sampling process, specimen handling, disturbance, and stress relief due to bringing the sample from depths to the ground surface with possible swelling. Other issues include: lack of information on sampling effective stress before testing, specimen trimming method, load application duration, secondary consolidation consideration, temperature, salt concentration in pore fluid, friction between specimen ring and the soil, load increment ratio and schedule, lack of proper saturation, specimen slenderness, and capacity of loading frame (Germaine & Germaine, 2009).

To overcome issues associated with the laboratory methods, σ_p' can be estimated using empirical correlations or analytical solutions with in-situ test measurements that avert the issues of sample disturbance and these are also faster and more economical than laboratory tests. Many relationships for stress history evaluation have been proposed for various in-situ tests such as cone penetration test (CPT), flat dilatometer test (DMT), standard penetration test (SPT), vane shear test (VST) and pressuremeter test (PMT), as discussed for instance by Mayne (1995).

Herein, a direct relationship between σ_p' and V_s was sought following the concepts and logic previously established by Mayne et al. (1998) and Mayne (2005) on smaller datasets.

2 SHEAR WAVE VELOCITY

Shear wave velocity (V_s) can be measured using either invasive and/or non-invasive geophysics, as well as obtained on small lab specimens. Shear waves can be measured in all geomaterials where it serves as an excellent reference benchmark in comparing stiffness and stress states. The measured V_s profile is fundamentally applicable to both static and dynamic geotechnical analyses as it provides the small-strain shear modulus.

The magnitude of shear modulus can be measured from small specimens in the laboratory using resonant column, ultrasonics, bender elements, and/or triaxial tests with local strain sensors, however, these methods have several issues: sampling disturbance difficulties, loss of ageing and diagenesis effects in addition to stress relief. Hence, V_s is best measured in-situ rather than in the laboratory.

Field methods for V_s measurement can be either invasive or non-invasive. Invasive methods include cased borehole methods such as: crosshole test (CHT), downhole test (DHT), uphole test (UHT), and P-S suspension logger, as well as direct push methods: seismic cone penetration test (SCPT) and seismic flat dilatometer test (SDMT) which are efficient versions of the DHT mode that measures a vertically-propagating horizontally-polarized shear wave velocity, or V_{sVH} mode. Non-invasive methods include refraction survey, reflection survey, and surface wave methods using either active sources to measure Rayleigh waves: spectral analyses of surface waves (SASW), multi-channel analyses of surface waves (MASW), and continuous surface wave method (CSW), or passive source techniques, such as passive surface waves (PSW) or reflection microseism (ReMi).

3 COMPILED DATABASE

A comprehensive database was prepared from a total of 64 well-documented worldwide geotechnical test sites. The sites primarily include soft to firm normally-consolidated young to aged clays, as well as a few sites from stiff to hard overconsolidated clays and fissured fine-grained soils. Only high-end laboratory tests were included with a focus on one-dimension consolidation tests for stress history profiles measured using standard 1D consolidation tests with a constant load increment duration of 24 hr (IL_{24}) and successive load increments applied at end-of-primary consolidation tests (IL_{EOP}), and constant rate of strain (CRS) consolidation tests.

Regarding V_s data, only in-situ results were compiled from downhole shear wave velocity (V_{sVH}) which can be measured using DHT, SCPT, and SDMT. At each of the studied sites, complementary field and lab data were also often available such as: Atterberg limits and unit weight for evaluating the effective vertical overburden pressure.

In the events where no undisturbed samples or natural water contents were available, unit weight was estimated using indirect methods such (Mayne 2005):

$$\gamma_t\ (\mathrm{kN/m^3}) = 8.63\log(V_s) - 1.18\log(z) - 0.53 \quad (1)$$

where V_s (m/s) and depth z reported in meters.

The compiled database has a total pairing of 790 one-dimension consolidation tests along with their corresponding V_{sVH} values which can be classified into 3 main groups: (a) normally consolidated (NC) intact to lightly overconsolidated (LOC) intact clays; (b) overconsolidated (OC) intact clays; and (c) highly overconsolidated (HOC) fissured clays.

A total of 36 different NC-LOC intact clays with a total of 486 preconsolidation effective stress (σ_p') measurements were collected, as presented in Table 1. Whereas, the OC clays generally exhibit 2 < OCRs < 5 and include intact and lightly cemented clays from 19 test sites with a total of 204 σ_p' measurements, as presented in Table 2. Finally, the HOC clays group includes fine-grained soils with OCRs > 5 and a total of 9 hard to fissured clay sites with 100 σ_p' measurements, as listed in Table 3. The reference sources of data and information in the presented tables are given in an abbreviated form (i.e., CEPNS = Characterization & Engineering Properties of Natural Soils, Taylor & Francis, London) due to the need to conserve space in allocated pages.

Table 1. Soft/firm clays (1<OCRs < 2) with paired σ_p' and V_{sVH}

Site	Location	Reference(s)
AIT	Bangkok	Watabe et al. (2004), 2nd ISC (2), Porto: 1765-1772.
Amherst, MA	USA	DeGroot & Lutenegger (2003), CEPNS (1): 695 - 724.
Ariake	Japan	Tanaka et al. (2001), Canadian Geot. J. 38 (2): 378-400.
Bäckebol	Sweden	Larsson & Mulabdić (1991), SGI (40)
Belfast	Ireland	Lehane (2003), Geot. Engrg. 156 (1) ICE, London: 17-26.
Bothkennar	UK	Hight et al. (1992) Géotechnique Vol. 42 (2): 303-347
Burswood	Australia	Low et al (2011) Géotechnique Vol. 61(7): 575-591

Busan	Korea	Chung et al. (2012), KSCE J. Civil Engrg. 16(3): 341-50
Drammen	Norway	Long & Donohue (2007), Canadian Geotechnical J. 44 (5): 533 – 544.
Evanston, IL	USA	Finno et al. (2000), National Geotech Sites, ASCE GSP 93: 130-159
Fucino	Italy	Burghignoli et al. (1991), 10th ECSMFE (1), Florence: 27-40.
Hachirougata	Japan	Tanaka (2006), CEPNS (3): 1831-1852
Higashi	Japan	Shibuya et al. (1995), Earthquake Geotechnical Eng. (1): 77-82
Kobe	Japan	Nakase et al. (1988), J. Geotech. Eng. 114(7): 844-858.
Kurihama	Japan	Shibuya & Tanaka (1996), Soils & Foundations 36(4): 45-55.
Lianyungang	China	Liu (2008), Marine Georesources & Geotechnology 26(3): 189-210.
Lierstranda	Norway	Long & Donohue (2007), Canadian Geotechnical J. 44 (5): 533 – 544.
Lilla Mellösa	Sweden	Larsson & Mulabdić (1991), Swedish Geot. Institute Report 42.
Munkedal	Sweden	Larsson & Mulabdić (1991), Swedish Geot. Inst. Report 42.
Museumpark	Norway	Long & Donohue (2007), Canadian Geot. J. 44(5): 533-544.
Norrköping	Sweden	Larsson & Mulabdić (1991), Swedish Geot. Institute Report 42.
Onsøy	Norway	Lunne et al. (2003), CEPNS (1): 395-428
Pentre	UK	Lambson et al. (1992), Large-scale Pile Tests in Clay, ICE:134-196
Perniö	Finland	Lehtonen (2015), PhD Thesis, Tampere University: 213 p
Sarapui	Brazil	Almeida & Marques (2003), CEPN (1): 477-504.
Sarapui II	Brazil	Jannuzzi et al. (2015), Eng. Geology (190): 77-86.
Saro Rd 6-900	Sweden	Larsson & Mulabdić (1991) Swedish Geot. Institute Report 42.
Saro Rd 7-600	Sweden	Larsson & Mulabdić (1991) Swedish Geot. Institute Report 42.
Singapore	Singapore	Tanaka et al. (2001), Canadian Geot. J. 38(2): 378-400
Skä Edeby	Sweden	Larsson & Mulabdić (1991) Swedish Geot. Institute Report 42.
S. Gloucester ON	Canada	Bozozuk (1972), PhD Thesis, Purdue Univ.: 208p.
Sutthisan	Bangkok	Shibuya & Tamrakar (2003), CEPNS, (2): 645-692.
Taipei	Taiwan	Chin et al. (2007), CEPNS (4): 1755-1803
Troll Upper	North Sea	Lunne et al. (2007), CEPNS (4): 1939-1972
Tuve	Sweden	Larsson & Mulabdić (1991) Swedish Geot. Institute Report 42.
Valen	Sweden	Larsson & Mulabdić (1991) Swedish Geot. Institute Report 42.

Table 2. Overconsolidated Clays with 2.0 < OCR < 5.0

Site	Location	Reference(s)
Beaumont, TX	USA	Mahar & O'Neill (1983), J. Geotech Engrg. 109(1): 56 -71.
Bonneville, UT	USA	Gardner (2007) MS Thesis, Brigham Young Univ: 142p.
Cooper Marl, SC	USA	Camp (2004), *GeoSupport* GSP 124, ASCE Reston VA: 1-18.
Hai-Phong	Vietnam	Watabe et al. (2004), 2nd ISC (2), Porto.
Hamilton, CA	USA	Nguyen (2007) MS Thesis, MIT: 366p.
Hilleren	Norway	Long et al. (2009), J. Geotech. and Geoenvir. Engrg. 135(2): 185-198.
I-395, ME	USA	Hardison (2013) MS Thesis. Univ. Maine
Louiseville, Quebec	Canada	Leroueil & Hamouche (2003), CEPNS (2): 363-393.
Montalto di Castro	Italy	Jamiolkowski & LoPresti (1994), 13th ICSMGE, (5):51-55.
Newbury, MA	USA	Landon (2007), PhD Thesis, U.Mass. Amherst: 701p.
Newport, OR	USA	Ferguson (2015), Seismic Evaluation Big Creek Dams: 561p.
Oseberg	North Sea	Rutledal et al. (2000), SEG: 570-573
Pisa	Italy	LoPresti et al. (2003), CEPNS (2): 909-946.
Port of Anchorage, AK	USA	Mayne & Pearce (2005), Frontiers Offshore Geot, Perth: 951-955.
Route 197 Bridge, ME	USA	Hardison (2013) MS Thesis, Univ. Maine: 394p.
Saint Alban, Quebec	Canada	Levesque et al. (2007), CEPNS (4): 2645-2677.
Troll Lower	North Sea	Lunne et al. (2007), CEPNS (4).
Lower 232 St., BC	Canada	Sully (1991) PhD Thesis, Univ. British Columbia: 485p.
200th St., BC	Canada	Sully (1991) PhD Thesis, UBC: 485p

Table 3. HOC and Fissured Clays with OCRs > 5

Site	Country	Reference(s)
AGS, NJ	USA	Stoll et al. (1988), J. Acoustical Society of America 83(1): 93-102.
Brent Cross	UK	Hight et al.(2003),CEPNS(2):851–907
Chattenden	UK	Butcher & Powell (1995), 11th ECSMFE (1), Copenhagen: 27-36
Cowden	UK	Hight et al.(2003),CEPNS(2):851–907
Heathrow	UK	Hight et al.(2003),CEPNS(2):851–907
Madingley	UK	Butcher & Powell(1995),11thECSMFE, Copenhagen
Martin's Point Bridge, ME	USA	Hardison (2013) M.Sc. Thesis Univ. Maine: 394p.
Oxford	UK	Bates & Phillips (2000), J. Applied Geophysics (44): 257-273
Tornhill	Sweden	Larsson (1991), SGI Rept 59: 169p.

4 ESTIMATING STRESS HISTORY FROM SHEAR WAVE VELOCITY

Using multiple regression analyses and investigating the compiled database from 64 clay sites given in Tables 1 to 3, a relationship between the preconsolidation stress, σ_p' (measured in kPa), the downhole shear wave velocity, V_{sVH} (measured in m/sec), and the effective vertical overburden stress, σ_{v0}' (measured in kPa) was developed, as shown in Figure 1, andcan be expressed:

$$\sigma_P' = 0.18 \cdot (V_{sVH})^{1.14} \cdot (\sigma_{vo}')^{0.26} \qquad (2)$$

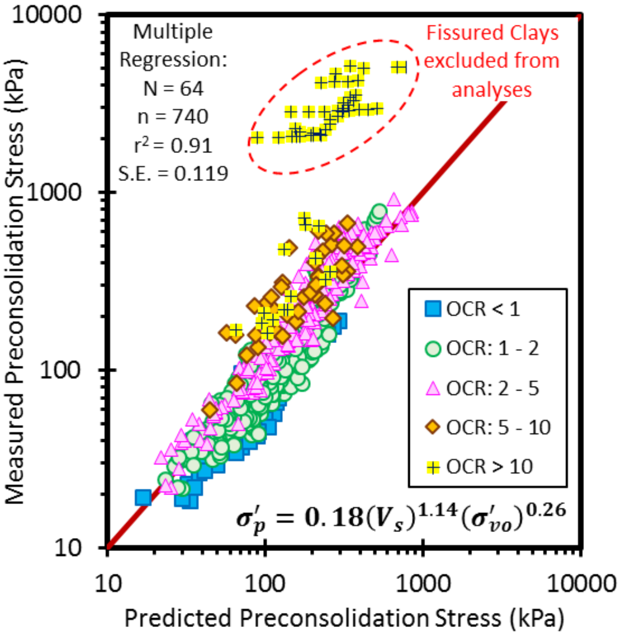

Figure 1. Measured versus predicted preconsolidation stress in terms of downhole shear wave velocity and effective vertical overburden stress for different OCR ranges.

Notably, the highly overconsolidated and fissured clays were found to behave differently and did not fit any of the developed expressions, therefore were excluded from the regression analyses.

By considering the results plotted in Figure 1, an good agreement between the measured and the predicted σ_p' values is achieved for intact clays. Hence, the proposed expression can be used as a first order estimate of stress history where only shear wave velocity data are available prior any other geotechnical investigations. Exception is again noted to the data from fissured overconsolidated clays that have discontinuities and joints. By considering the complementary compiled data, it was interesting to investigate the effect of the plasticity characteristics of the studied clays expressed in terms of plasticity index (PI), as shown in Figure 2. The clays can be divided into 4 subgroups based on their plasticity ranges where the higher the plasticity the lower the preconsolidation stress. Note here that the majority of data are from low OCR clays in the NC-LOC range.

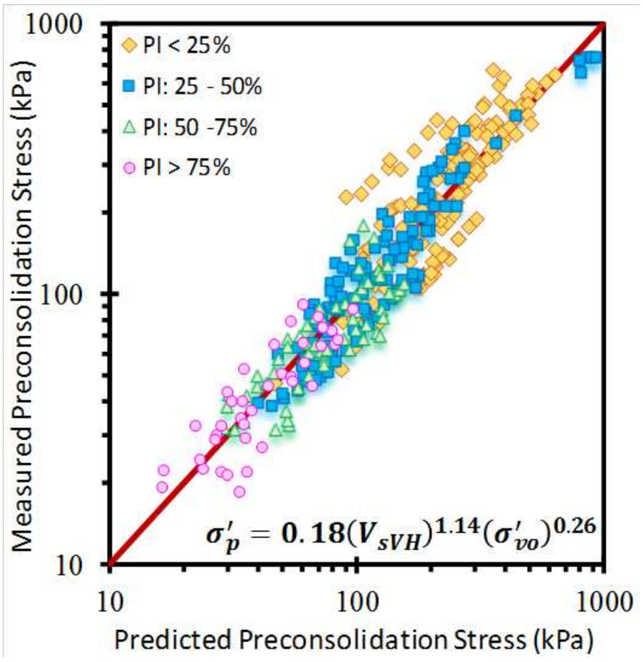

Figure 2. Measured versus predicted preconsolidation stress in terms of downhole shear wave velocity and effective vertical overburden stress for different PI values.

5 UNDRAINED SHEAR STRENGTH AND CASE STUDY APPLICATIONS

Different lab and field testing techniques can be used to measure undrained shear strength and provide a reference profile of s_u with depth. An alternate means to directly evaluating the magnitude of s_u in clays involves the utilization of stress history (i.e., OCR) of the clay deposit to profile a family of undrained strength ratios:

$$(s_u/\sigma_{vo}') = S \cdot OCR^m \qquad (3)$$

where the coefficient S and exponent m can be found using SHANSEP (Stress History And Normalized Soil Engineering Parameters), as detailed by Ladd (1991) and Ladd & DeGroot (2003). The value of *S* = $(s_u/\sigma_{vo}')_{NC}$ is found experimentally by extensive laboratory testing with companion series of plane strain compression (PSC), simple shear (SS), and plane strain extension (PSE) tests on the soils at varied OCRs, or by series of triaxial compression (TC), simple shear (DSS), and triaxial extension (TE) tests. Representative S values as suggested by Ladd (1991) are 0.30 for TC, 0.21 for DSS, and 0.15 for TE. The exponent *m* can be determined experimentally and has been generally found to be on the order of 0.8 ± 0.1.

5.1 *Bothkennar Clay*

Bothkennar is a soft silty estuarine clay that is located on the south side of the River Forth, between Edinburgh and Glasgow in Scotland (Hight et al., 2003). The clay has the following average index parameters and soil properties: e_0 = 1.69, w_n = 61%, LL = 72.6%, PI = 41.8%, clay fraction of 30%, G_s = 2.65, and bulk density (ρ) = 1.607 Mg/m^3. The behavior in one-dimensional compression was investigated by Hight et al. (1992) using three types of consolidation tests: conventional incremental load tests, continuous load tests, and restricted flow tests as presented in Figure 3b. These give an average OCR = 1.54 with depth. Results of downhole V_{sVH} profiles from SCPT were measured by Hepton (1988) and presented in Figure 3a. These data along with the profile of the effective vertical stress are used to provide an estimate for σ_p' using Equation (2) showing a good agreement as shown in Figure 3b.

The evaluated stress history profile is used to estimate s_{uDSS}, s_{uTC}, and s_{uTE} profiles for Bothkennar clay via Equation (3), with adopted S values of 0.3 for TC, 0.21 for DSS, and 0.15 for TE and an exponent of m = 0.8, as presented in Figure 3c. Laboratory CK_0UC, DSS, and CK_0UE tests on undisturbed samples from the site are reported by Hight et al. (2003). Comparison of results show good agreement between measured and estimated profiles.

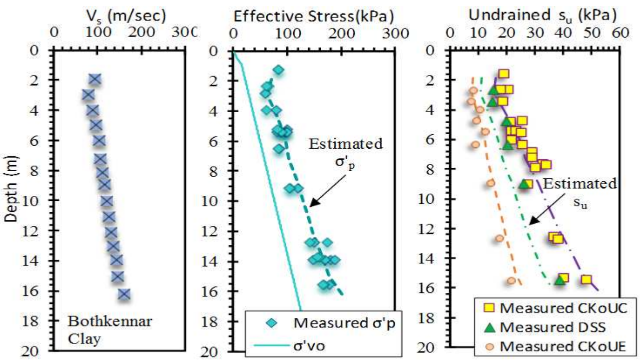

Figure 3. Profiles at Bothkennar soft clay: (a) Measured downhole shear wave velocity profile; (b) Estimated preconsolidation stress compared to lab-measured values; (c) Lab-measured and V_s-estimated TC, DSS, and TE strengths.

5.2 *Busan Clay*

Busan (or Pusan) is a soft clay that covers the Nakdong River delta in South Korea (Locat and Tanaka 1999). The clay has the following average index parameters and soil properties: e_0 = 1.59, w_n = 58.4%, LL = 61.5 %, PI = 27.2%, clay fraction of 27%, G_s = 2.71, and bulk density (ρ) = 1.60 Mg/m^3 (Chung et al., 2011). The behavior in one-dimensional compression was investigated by Singh & Chung (2015) using three test types: standard 1D consolidation tests with a constant load increment duration of 24 h (IL_{24}) and a successive load increments applied after 100% primary consolidation (end-of-primary consolidation, IL_{EOP}); and a constant rate of strain (CRS) consolidation test.

Results of downhole V_{sVH} profiles from SCPT are reported by Chung and Kweon (2013) and presented in Figure 4a. These data along with the profile of the effective vertical stress are used to provide an estimate for σ_p' using Equation (2) showing a good agreement as shown in Figure 4b. The estimated stress history profile is used to evaluate the OCR profile to estimate s_{uDSS}, s_{uTC}, and s_{uTE} with adopted S values of 0.30 for TC, 0.21 for DSS, and 0.15 for TE and an exponent m = 0.8 using Equation (3) as presented in Figure 4c. Laboratory CK_0UC and CK_0UE tests on undisturbed samples are reported by Chung et al. (2012). By comparing the undrained shear strength profiles, the estimated values compare well with lab data.

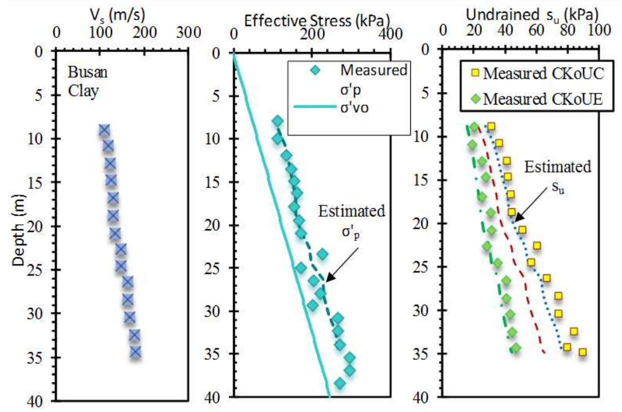

Figure 4. Profiles in soft Busan clay: (a) Measured downhole shear wave velocity; (b) Estimated preconsolidation stress compared to lab-measured stress history profile; (c) Lab-measured and V_s-estimated TC, DSS, and TE strengths.

5.3 *Onsøy Clay*

Onsøy is a soft marine clay that is located in Norway, southeast of Oslo (Lunne et al., 2003). The soil profile at Onsøy consists of a one meter thick weathered crust followed by an 8-m thick soft clay layer underlain by a soft medium plastic clay layer over the remaining thickness of 36 m over bedrock. The clay has the following average index parameters and soil properties: clay fraction of 53 %, e_0 = 1.75, w_n= 64%, LL= 68%, PI= 35%, G_s= 2.71, and bulk density (ρ) = 1.587 Mg/m^3. The profile of preconsolidation stress from one-dimensional oedometer tests has been measured in several investigations (e.g. Lunne et al., 2001). These give an average OCR = 1.69. The profile of the field measured V_{sVH} from seismic cone test is presented in Figure 5a. These data along with the profile of the effective vertical stress are used to provide an estimate for σ_p' using Equation (2) showing a good agreement as shown in Figure 5b.

The estimated stress history profile is used to evaluate the OCR profile to estimate s_{uDSS}, s_{uTC}, and s_{uTE} for Onsøy clay with typical S and m values as in Bothkennar and Busan clays as presented in Figure 5c. Laboratory s_u from DSS, CK_0UC, and CK_0UE tests have been obtained and reported by Lunne et al. (2006). Reasonable agreement is evident between measured and estimated values.

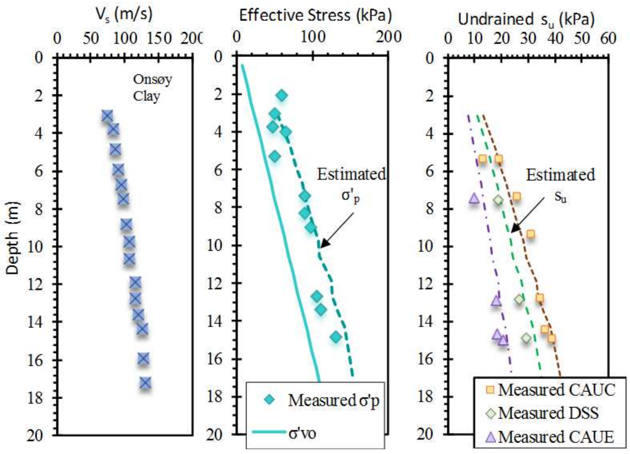

Figure 5. Profiles in soft clay at Onsøy, Norway: (a) Measured downhole shear wave velocity; (b) Estimated preconsolidation stress compared to lab measured stress history profile; (c) Measured and estimated DSS, TC, and TE strengths.

6 SUMMARY AND CONCLUSIONS

A compiled database study was statistically analyzed to produce a general expression for σ_p' from downhole shear wave velocity (V_{sVH}) and effective vertical overburden stress (σ_{v0}'). The data were collected from 64 worldwide well-documented natural soils primarily covering primarily NC to LOC clays, as well as some OC and HOC fissured clays. Case studies were used to verify the ability of the proposed expression in estimating stress history in comparison with reference values from consolidation tests on undisturbed samples.

The derived stress history profiles were used to estimate the undrained shear strengths under three different shearing modes: triaxial compression, direct simple shear, and triaxial extension using the SHANSEP approach.

7 ACKNOWLEDGMENTS

The authors appreciate the support of ConeTec Investigations of Richmond, BC and Design House Engineering Consultancy of New York, NY in support of this research effort.

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