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Session Report on Sampling and Laboratory Testing

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ABSTRACT: This report provides an overview of the reviewed papers presented at the Conference under the theme of sampling and laboratory testing. Two key themes namely 'sampling and sample quality assessment' and 'Testing' are reviewed.

1 INTRODUCTION

This Session Report reviews the topic of Sampling and Laboratory Testing based on 11 papers submitted to the 5th International Conference on Geotechnical and Geophysical Site Characterization, Gold Coast, Australia. The papers presented to this session describe a wide range of natural as well as artificial geomaterials, varying from sands to highly plastic clays and mudstone. The majority of the natural soils correspond to high plasticity clays and silts whereas artificial soil specimens (silica silt-kaolin mixtures) are used to represent the behaviour of the so-called *intermediate soils* (e.g., silty clays, clayey silts with PI typically between 0-10) (Figure 1). Two key themes, *Sampling and Sample Quality Assessment* as well as *Laboratory Testing*, are reviewed in the report, identifying the main findings of the studies reported.

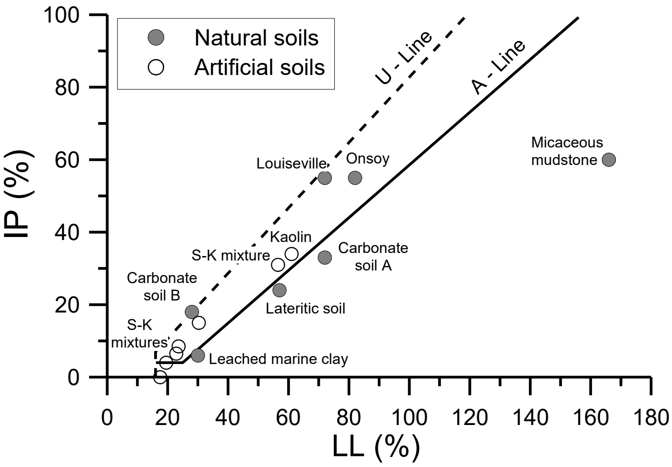


Figure 1.Plasticity of soils tested.

2 SAMPLING AND SAMPLE QUALITY ASSESSMENT

Available criteria for assessing sample quality in soils have been developed using laboratory results obtained primarily for marine clays (PI between 6 and 43) retrieved from relatively shallow depths (5-25 m). Sample quality is estimated in terms of the volumetric strain change (SQD, Andresen and Kolstad, 1979; Terzaghi et al., 1996) or the normalized voids ratio change ($\Delta e/e_0$, Lunne et al., 1997) caused during recompression to the *in situ* effective stress in laboratory tests. These methods are currently used to evaluate sample quality in a wide variety of natural soils without additional considerations. Although the influence of OCR is accounted for, no correction is considered for recompression to *in situ* stress in specimens with high overburden stresses which have been subjected to large stress relief due to sampling. The paper by *Krage et al.* explores this topic by using artificial silica silt-kaolin mixtures to prepare reconstituted specimens with PI ranging from 0 to 31. Specimens are subjected to a wide range of overburden stresses ($20 < \sigma'_{v0} < 500$ kPa) to establish depositional stress history. Two levels of disturbance are then induced as follows: 1D 'perfect sampling' (1DPS) and highly disturbed (HD) state. 1D 'Perfect sampling' condition is achieved via removal of deviatoric stress until reach K_0 of 1 whereas highly disturbed specimens are obtained by applying a freezing-thawing cycle under unstressed conditions. HD samples are then loaded beyond the preconsolidation stress followed by unloading until achieve K_0=1, as imposed to 1DPS specimens. Finally, both 1DPS and HD samples are loaded further to a vertical effective stress of 2500 kPa. Figure 2 shows HD specimens range from very good to excellent to poor sample quality. These results are inconsistent with level of disturbance induced to each specimen. There, the influence of the stress relief (overburden stress) on $\Delta e/e_0$ in HD specimens should be considered. *Krage et al.* suggest that the incorporation of the unloading-reloading stiffness would be useful to improve the assessment of sample quality in low plasticity soils subjected to large stress relief.

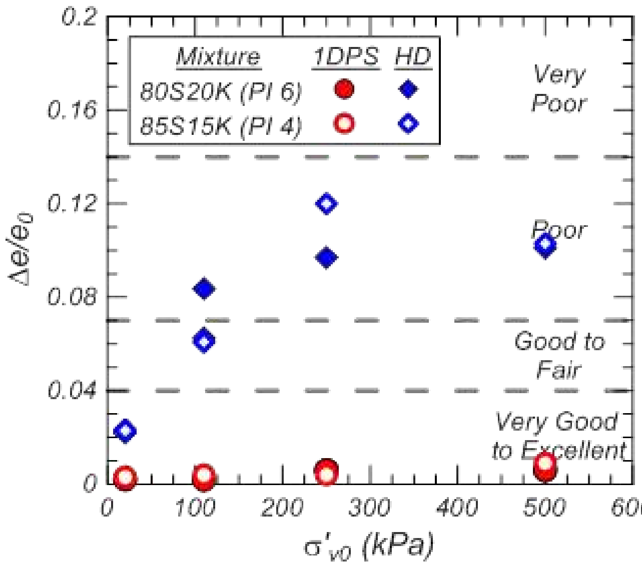
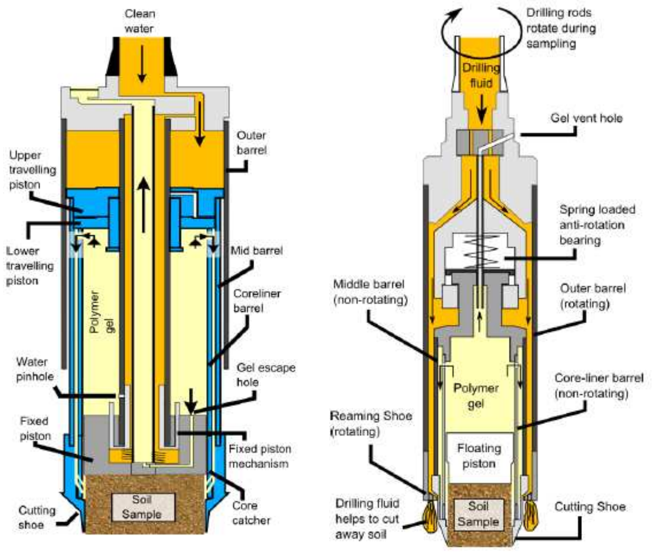


Figure 2. Sample quality assessment for 1DPS and HD specimens (4 in-situ stress levels and 2 mixtures: PI=7 and PI=4) (from *Krage et al.*).

Emerging tube sampling techniques are nowadays getting attention from practitioners due to the well-recognized issues of standard sampling techniques for obtaining undisturbed specimens in granular soils (e.g., clean and silty sands) as well as the cost-prohibitive use of the freezing technique. This is the case of geotechnical projects where the liquefaction potential has to be evaluated. There, high-quality soil specimens are required to carry out laboratory tests. *Stringer et al.* describe the use of the Gel-Push tube sampling technique for obtaining undisturbed specimens of silty soils, micaceous silts and clean sands in New Zealand. The Gel-Push (GP) sampling technique (e.g., Lee et al., 2012), which keeps the same operational principle as the Osterberg fixed-piston sampler, assumes that the main source of soil disturbance is due to sidewall friction as the soil enters the tube sampler. To overcome this problem, a low friction polymer gel is injected which acts as lubricant. Three different versions of Gel-Push sampler are available (GP-S, GP-Tr and GP-D) depending on the system employed to deliver the gel to the base of the sampler. *Stringer et al.* used the GP-S sampler in silty soils and silts whereas an attempt was made with the GP-Tr version in clean sands (Figure 3). Sample quality is assessed by visual inspection after soil extrusion as well as from the comparison between in situ and laboratory shear wave velocity measurements. *Stringer et al.* report a successful trial using the GP-S sampler in silty clays and silty sands. They also provide some comments about particular operational aspects for further application of the sampler in similar soils. Large amount of swelling, which led to poor sample quality, is reported for the GP-S in micaceous silts. The trials using the GP-Tr sampler indicate that further improvements are required to obtain undisturbed specimens, at least in the case of clean sands. Overall, the GP sampling technique appears to be very promising for obtaining high-quality specimens in complex natural soils deposits.



(a) (b)

Figure 3.Schematic view of the GP sampler. (a) GP-S sampler. (b) GP-Tr sampler (from Stringer et al.)

Soil sampling in offshore projects put forward additional challenges which make difficult to obtain undisturbed specimens for laboratory testing. Gravity samplers are commonly employed in offshore geotechnics, mainly for characterization purposes, due to its operational simplicity and economical cost. A common observation is the larger sampler penetration regarding to sampler recovery. The contribution by *Ramsey* proposes an approach for predicting sampler penetration and sample recovery by using CPT data. It is assumed that the soil recovery is less than the sampler penetration due to a temporary tube 'plugging' at one or more elevations during penetration. The phenomenon of tube 'plugging' occurs when the friction resistance exceeds the bearing capacity of the soil. Analytical expressions are provided in the paper to estimate 'plug' as well as 'unplug' resistances (F_{plug} and F_{unplug}) used to predict the elevations at which tube 'plugging' may occur: $F_{unplug}/F_{plug}>1$.

The applicability of the proposed methodology is demonstrated by using a case study where CPT data are available (Figure 4). Tube 'plugging' is predicted to occur at three different levels which is in agreement with the sampler inspection. Maximum variations of -15% in tube penetration and +/- 10% in sample recovery are reported by *Ramsey* for the case study described in the paper. Further improvements to the proposed technique are discussed by the Author in order to minimize uncertainty of the predicted sample recovery.

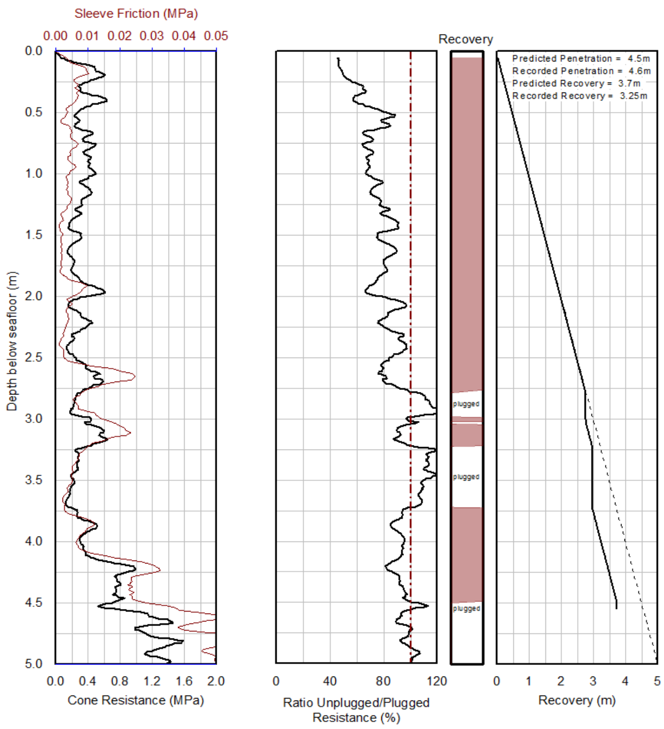


Figure 4.Assessment of sampler recovery (from *Ramsey*).

3 LABORATORY TESTING

3.1 *Undrained shear strength and soil sensitivity*

Amundsen et al. describe the geotechnical characterization of Rissa clay, a lightly overconsolidated low plasticity (OCR≈2; PI≈8.5) leached marine clay from Norway. The soil profile at Rissa site, estimated using electrical resistivity tomography (ERT), shows a complex geological environment formed by the indentation between two mountain ridges filled with marine clay as well as sand and gravel deposits (see Figure 5a). The shallow upper 9 m correspond to leached marine clay with salt content ranging between 2.0 to 9.5 g/l. High-quality block specimens, retrieved from 4 m depth using the Sherbrooke sampler, are used to characterize the mechanical behaviour of the clay. Additional specimens retrieved using piston samplers (54 mm and 73 mm in diameter) allow the Authors to assess sample quality based on the results from one-dimensional and triaxial compression tests. The comparison shows that, as would be expected, block specimens produce the highest sample quality followed by the 73 mm piston sampler. *Amundsen et al.* explores the rate dependency of the undrained shear strength in Rissa clay by means of CAUC triaxial tests. The undrained shear strength increases around 20% with the strain rate from 0.1%/h to 4.5%/h (typical strain rates in Norway varies from 0.7%/h to 3.0%/h). This behaviour is in agreement with the results presented by Lunne and Andersen (2007) for NC and OC Norwegian clays (Figure 5b). It is shown that the failure envelope of Rissa clay is not affected by rate effects. It means that the shear strain rate only affects the induced excess pore pressure without modifying the effective cohesion or the friction angle of the soil. The influence of the strain rate on the preconsolidation stress of Rissa clay (one-dimensional loading) is also explored by comparing estimations of σ'_{prec} obtained from Incremental Loading (24h load steps) and Constant Rate of Strain (1.5%/h) tests. An increase in σ'_{prec} of 16 % is reported for the case of the CRS test. Overall, the mechanical behaviour of Rissa clay is consistent with the response of other Norwegian low plasticity clays previously reported in the literature.

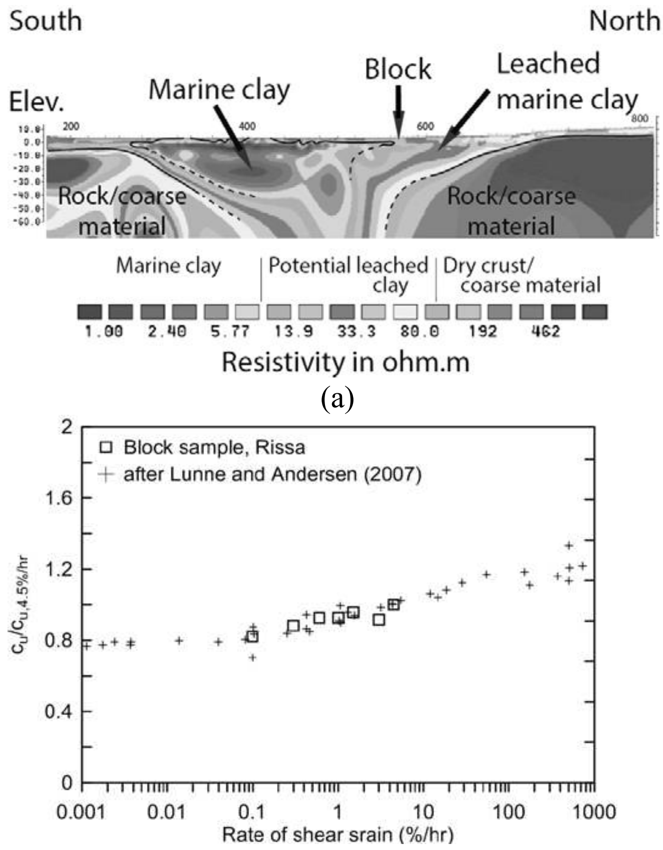


Figure 5.(a) Geophysical profile at Rissa site. (b) Normalized undrained shear strength of Rissa clay as a function of the strain rate (from *Amundsen et al.*).

The paper by *Hirabayashi et al.* describes the engineering properties of three natural high-plasticity soft clays estimated from in situ and laboratory tests: Onsoy clay (Norway), Louisville clay (EEUU) and Mexico City clay (Mexico). Particular emphasis is given here to the estimation of the cone factor N_{kt}, required to compute the undrained shear strength, by comparing CPTu data against in situ (Field Vane

Test, FVT) as well as laboratory tests results (constant-volume Direct Shear Tests, DST, and Unconfirmed Compression Test, UCT). As would be expected, N_{kt} varies depending on the testing method with no appreciable influence of the soil type. The cone factor N_{kt} seems insensitive to changes in plasticity index irrespective of the clay type (see Figure 6). Mean values of N_{kt} are: 12.5 (FVT), 13.4 (UCT_ and 11.5 (DST). *Hirabayashi et al.* claim further clarification in the Standards to select the testing technique for estimating the undrained shear strength (and therefore N_{kt}) in soft clays.

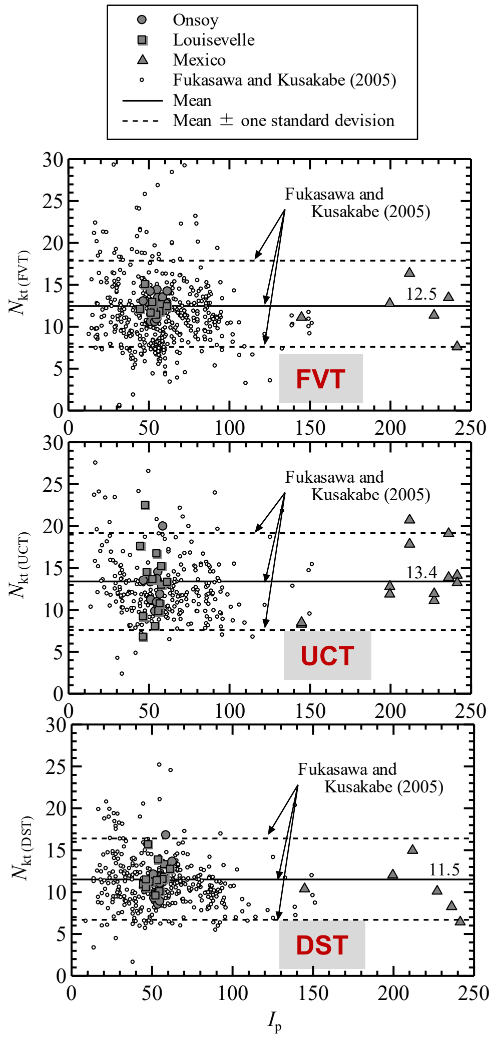


Figure 6.Estimated values of N_{kt} as a function of PI (from *Hirabayashi et al.*).

The contribution by *Arsalan et al.* reports a laboratory investigation carried out to estimate the undrained shear strength as well as the anisotropic yield surface of a natural diatomaceous mudstone from Japan. A comprehensive experimental program that included CID as well as CK_0U triaxial tests is used to study the yielding behaviour of this complex naturally cemented soft rock. CID tests results are used to map the initial yield surface of the mudstone which seems to be well-represented by the Original Cam Clay (OCC) model (see Figure 7a). The stress-strain response observed in CK_0U compression tests shows an increase in rock brittleness with overconsolidation state: from 25% in NC samples up to 40% in OC specimens. Anisotropic consolidation leads to the enlargement of the yield locus with a variation in shape so that the effective stress ratio at maximum deviatoric stress differs from the stress ratio at critical state (see Figure 7b). The OCC model is therefore unable to represent the new yield locus. *Arsaland et al.* propose a modification of an existing yield function to properly capture the (macroscopic) anisotropic response of the mudstone. As indicated by a solid black line in Figure 7b, good agreement between the experimental results and the model simulations is achieved by using the modified anisotropic yield locus.

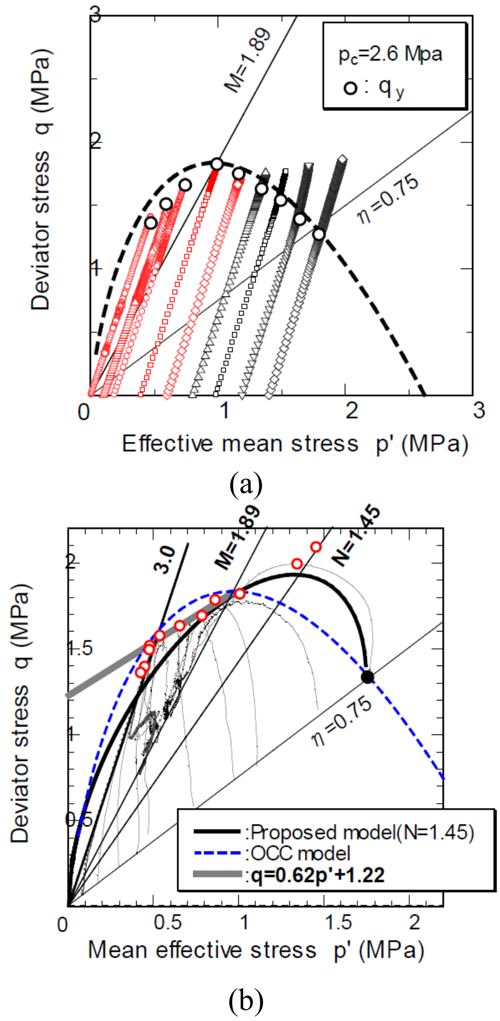


Figure 7. (a) Stress paths and yield surface from CID triaxial tests. (b) Stress paths and anisotropic yield surface from CK_0U triaxial tests (from *Arsalan et al.*)

The paper by *Boukpeti and Lehane* explores the use of simple laboratory techniques for the estimation of the soil sensitivity in two natural carbonate soils (Soil A and Soil B) retrieved using Shelby tubes from the North West Shelf in Australia. Although both soils display similar mineralogical compositions (mainly calcite and aragonite) Soil A classifies as a well-graded clayey silt whereas Soil B is a well-graded silty sand. Fines content/clay fraction are equal to 85/28 and 35/10 for Soil A and Soil B, respectively. Soil sensitivity (S_t) is evaluated by means of three different laboratory tests as follows: fall cone (apex angle of 30, cone mass of 80g and cone factor K=2), hand vane (rotation speed of 1 r.p.m) as well as miniature T-bar penetrometer (resistance factor N_p=10.5). Values of soil sensitivity show important differences depending on the testing method. In Soil A, S_t increases with depth for the fall cone (4-33) and T-bar (9-18) but it remains approximately constant for the hand vane (~ 4) (see Figure 8a). Soil B shows lower soil sensitivity than Soil A which is indicative of differences in soil structure. S_t ranges around 4-18 for the fall cone and remains almost uniform for the hand vane (~ 6) (T-bar was not used in Soil B). Tests carried out on reconstituted Soil A show that St reduced, around 2 times less the value measured in undisturbed specimens. This behaviour remarks the importance of the natural soil structure which may not be created by reconstitution methods. The use of available correlations for soil sensitivity and liquidity index (LI) predict large variation in S_t for specimens with similar liquidity indices (LI). *Boukpeti and Lehane* conclude that LI is not an adequate parameter to assess the sensitivity of carbonate soils. The Authors use results obtained from one-dimensional compression tests carried out on Soil A to estimate soil sensitivity following the sensitivity framework proposed by Cotecchia and Chandler (2000). There, sensitivity prediction is based on the distance of the yield point in compression to the Intrinsic Compression Line, ICL: $S_t = \sigma'_{vy} / \sigma'_{ey}$ (Figure 8b). Estimated values of St are slightly lower than sensitivities measured on specimens of similar depths. This discrepancy is attributed partly to disturbance caused by tube sampling.

3.2 *Cyclic undrained shear strength*

The common assumption in offshore design of a unified failure criterion for normally consolidated clays under symmetrical and non-symmetrical undrained cyclic loading conditions is studied in the paper by *Zografou et al.* The results of symmetrical and non-symmetrical Cyclic Direct Simple Shear (CDSS) tests carried out on normally consolidated kaolin are discussed. The CDSS tests were carried using stacked rings to restrict the lateral deformation. The vertical load was adjusted during the cyclic shearing stage in order to ensure constant volume conditions. Samples used in symmetrical cyclic loading tests were incrementally consolidated to a maximum vertical effective stress of 150 kPa. This value reduced to 70 kPa in non-symmetrical tests. The cyclic loading was applied at a frequency of 0.1 Hz and it was maintained for N=1000 cycles unless failure was achieved earlier. Low stress levels were used by *Zografou et al.* during CDSS tests in order to simulate similar conditions to those occurring in offshore subsea structures. Results from symmetrical loading tests show that failure is achieved by cyclic degradation at a maximum shear strain of 4-5% (vertical dashed line in Figure 9a). On the other hand, shear strain accumulation is the failure mechanism observed in non-symmetrical tests (Figure 9b). The maximum shear strain is in this case much higher than achieved in symmetrical tests. It means that independent failure criteria should be used in design depending on the loading type. Shear strain contour diagram for symmetrical cyclic loading (only) is provided in the paper which could be used in design. In the case of non-symmetrical tests, high tolerances to movements (larges shear strains) should be considered for the definition of the failure criterion.

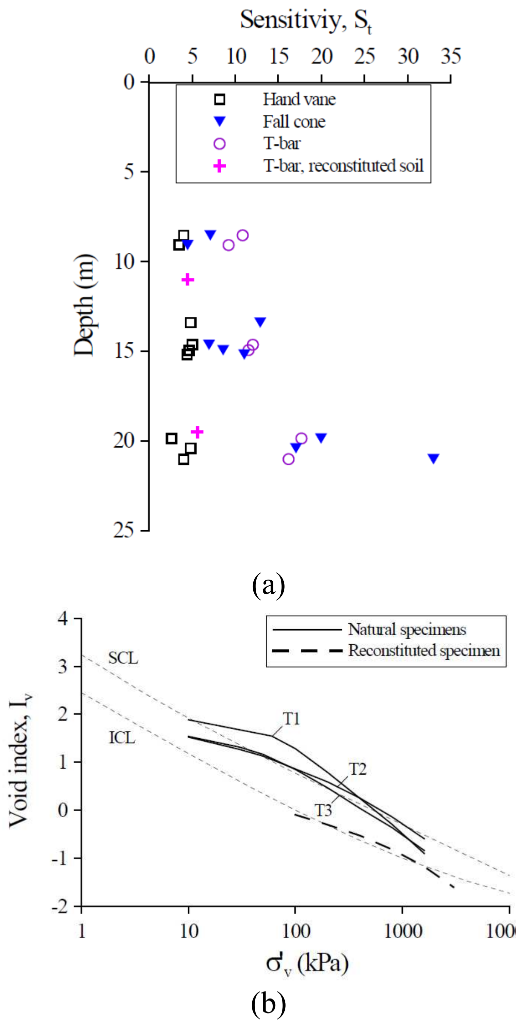
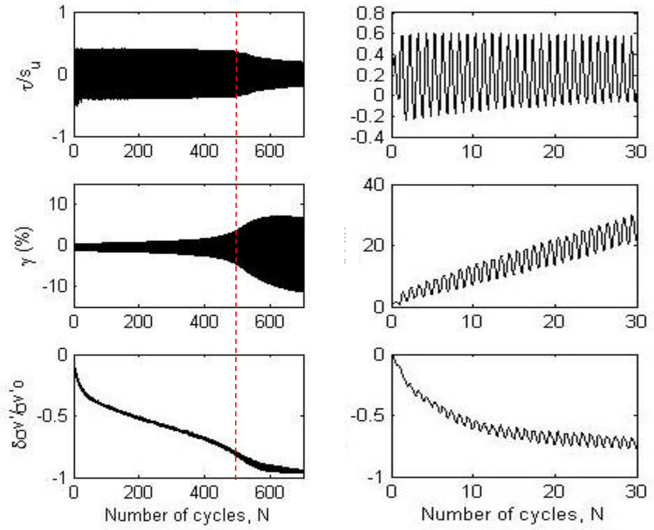


Figure 8.(a) Variation of St with depth for Soil A. (b) Estimation of St from oedometer test results (from *Boukpeti and Lehane*).

A simple methodology for the assessment of the cyclic softening in clays from Matsyapuri (India) is described by *Raskar-Phule et al.* The term 'cyclic softening' refers to the temporal reduction in clay strength due to the increase in excess pore pressure during undrained cyclic loading (e.g. earthquakes). The soil profile under study is composed by an upper zone fill layer (clayey sand + gravel) overlaying high plasticity cohesive layer of around 6.7 m thickness. The cyclic softening is assessed at five locations by evaluating the cyclic resistance ratio (CRR) and the cyclic stress ratio (CSR). A factor of safety (FoS=CRR/CSR) of 1.3 is adopted as threshold value to establish the occurrence of cyclic softening of Matsyapuri clays. CSR and $CRR_{7.5}$ are estimated following the proposals by Idriss and Boulanger (2008) and Idriss and Boulanger (2007). In the absence of laboratory as well as in situ estimations, values of undrained shear strength are computed using an empirical correlation expressed in terms of N_{SPT}, water content, liquid limit and plasticity index. Earthquakes of moment magnitudes 6.5 and 7.5 are considered in combination with maximum accelerations of 0.16 g and 0.3 g. For such scenarios, the results of this rather simple methodology show that clays at depths between 3.75 m to 12 m may be subjected to cyclic softening.



(a) (b)

Figure 9.(a) Symmetrical CDSS test results for τ_{max}/s_u=0.40. (b) Non-symmetrical CDSS test results for τ_{max}/s_u=0.60 and τ_{ave}/τ_{cyc}=0.5) (from *Zografou et al.*)

3.3 *Small strain stiffness*

The contribution by *Décourt et al.* compares estimates of small-strain shear modulus (G_0) from *in situ* and laboratory tests for a Brazilian lateritic soil. As expected, very good agreement is observed between *in situ* seismic tests (cross-hole and SDMT). *Décourt et al.* correlate values of G_0 (cross-hole) with N_{SPT} values obtained from SPT tests and compare them against available relationships from the literature for non-cemented soils. Site-dependent empirical correlations for lateritic soil deposits are also considered. It is shown the relationship between G_0 and N_{SPT}, q_c, p′ (and e) is not properly captured by using general expressions for non-cemented soils (including temperate zone soils). Lateritic soils display a much higher small strain stiffness than the predicted by using expressions for non-cemented soils. The reason for such higher soil stiffness in lateritic soils is attributed by the Authors to chemical bonding.

3.4 *K_0 estimation in granular materials*

Lee et al. revisit the expression proposed by Jaky (1944) for the estimation of the coefficient of earth pressure at rest (K_0=1-sinϕ′) in granular materials. By using oedometer tests results carried out on sand, glass beads and etched glass beads it is shown that the use of the critical state friction angle ϕ'_c in the original Jaky's equation is not able to capture both the behaviour of uniform and irregularly angular particles. Good agreement is only obtained in the former case. By adopting an inter-particle strength model, the following modified expression for K_0 is proposed in terms of the critical state friction angle ϕ'_c which incorporates a new parameter β to account for the particle interlocking:

$$K_0 = \frac{1-\sin(\beta \cdot \phi'_c)}{1+\sin(\beta \cdot \phi'_c)} \qquad (1)$$

Equation (1) is expanded further to consider the influence of relative density (D_R) on particle interlocking by incorporating an experimentally-based relationship between β and D_R, given by: $\beta=a[D_R(\%)]^b$, where a and b are correlation parameters.

4 CONCLUDING REMARKS

The papers submitted to the technical session on *Sampling and Laboratory Testing* at the 5th International Conference on Geotechnical and Geophysical Site Characterization provide a useful snapshot of the current state of practice. The following general observations can be inferred.

The effects of sampling depth on sample quality are not well quantified using current clay-based quality criteria. Consideration of overburden stress and stress history is crucial for the correct assessment of sample quality in intermediate soils.

Emerging sampling technologies like the GP sampler are currently getting attention due to their potential use in complex soil deposits to provide high-quality specimens for geotechnical characterization. Despite being a global problem, the answers to the sampling issues are local, because they need to be grounded in the local practice of drilling and sounding and be adapted to suit the local geological and geochemical conditions. Further research should be devoted to understand the effects of tube sampling on the soil fabric as it controls the mechanical behaviour of the soil.

The estimation of the static but also cyclic undrained shear strength has been the main topic of the papers devoted to Laboratory Testing. Aspects like the failure criteria for symmetrical and non-symmetrical undrained cyclic loading conditions (crucial in offshore design), the strain rate dependency of the undrained shear strength and preconsolidation pressure as well as the effects of natural leaching in sensitive clays have been discussed in some detail. However, in the Reporter´s opinion, little attention is given in current practice to the presence of cations/ions in natural soils and the application and/or control of geochemical variables (e.g. pore fluid conductivity) during index and mechanical tests.

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