Guidelines for offshore in situ testing and interpretation in deepwater soft clays

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Abstract: Offshore developments for hydrocarbon resources have now progressed to water depths approaching 3000 m, with geotechnical design increasingly focused on soft sediments in the upper 30 m or so of the seabed. Due to the difficulties and high cost in recovering high-quality samples from deepwater sites, there is increasing dependence on in situ testing techniques for determining the geotechnical design parameters. This paper summarizes the findings from a joint industry project, undertaken by the Norwegian Geotechnical Institute and the Centre for Offshore Foundation Systems at The University of Western Australia, on the use of in situ testing for the characterization of deepwater soft soils. The project focused on theoretical and empirical studies for the interpretation of piezocone, T-bar, and ball penetration test data, linking the penetration resistance to shear strengths determined from in situ vane tests and laboratory tests on high-quality samples. Guidelines are summarized for interpreting in situ test data, particularly for estimating intact and remoulded undrained shear strengths from the penetration resistance measured by each type of penetrometer. Suggestions are also given for future development of in situ testing techniques to maximize the potential of in situ testing in characterization of deepwater soft soils.

Key words: offshore soil investigations, soft clay, full flow penetrations, undrained shear strength.

Résumé : Les développements des ressources d'hydrocarbures en mer atteignent maintenant des profondeurs de près de 3000 m, et la conception géotechnique est de plus en plus centrée sur les sédiments mous des premiers 30 m du fond marin. En raison des difficultés et des coûts élevés associés à la récupération d'échantillons de qualité des sites profonds, les techniques in situ voient leur utilisation augmenter lors de la détermination des paramètres géotechniques de conception. Cet article résume les résultats obtenus lors d'un projet industriel conjoint, entrepris par l'Institut géotechnique norvégien et par le Centre for Offshore Foundation Systems à l'Université de Western Australia, sur l'utilisation de méthodes in situ pour la caractérisation des sols marins mous et profonds. Le projet s'est penché sur les études théoriques et empiriques de l'interprétation des données provenant d'essais au piézocône, au T-bar et au pénétromètre à bille, reliant la résistance à la pénétration aux résistances en cisaillement déterminées par des essais scissométriques in situ et en laboratoire effectués sur des échantillons de qualité élevée. Des directives pour interpréter les données d'essais in situ sont résumées, en particulier pour l'estimation de la résistance au cisaillement non drainée intacte et remaniée à partir des résistances à la pénétration obtenues pour chaque type de pénétromètre. Des suggestions sont aussi fournies pour le développement futur des techniques d'essai in situ afin de maximiser le potentiel de la caractérisation in situ des sols mous et profonds.

Mots-clés : études des sols marins, argile molle, pénétration en écoulement complet, résistance au cisaillement non drainée.

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Introduction

Geotechnical properties of near-surface seabed sediments are of increasing importance for deepwater hydrocarbon field developments, where offshore installations will typically comprise wellheads and subsea completions, pipelines, and shallow anchoring systems. In addition, geohazard evaluation (in particular submarine slides) has also proved to be of increasing importance at deepwater sites. In general, sediments underlying deepwater sites are soft, normally consolidated, fine-grained deposits, with low strengths (<20 kPa) at the surface and moderate strength increases with depth (1 to 2 kPa/m). This has resulted in increasing difficulties and cost in recovering high-quality soil samples, which in turn has led to increasing reliance on in situ testing for the determination of design parameters.

The accuracy of piezocone penetration test (CPTU) data in soft clays may decrease as the water depth increases. This is due partly to (*i*) reduced sensitivity of the load cell in measuring the small load increment from the penetration resistance in soft clays compared with the high ambient pressure at the seabed and (*ii*) uncertainty in corrections for the unequal area effect and contribution of overburden stress to the cone resistance. These equipment limitations can be reduced

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by using full-flow penetrometers, i.e., T-bar and ball penetrometers (Fig. 1) with projected areas an order of magnitude greater than the penetrometer shaft. Since the introduction of T-bar and ball penetrometers in 1996 and 2003, respectively (Randolph et al. 1998; Kelleher and Randolph 2005; Peuchen et al. 2005), full-flow penetrometers are now used in many offshore site investigations. However, clear guidelines on testing procedures and data interpretation are still emerging.

A joint industry project was undertaken by the Norwegian Geotechnical Institute (NGI) and the Centre for Offshore Foundation Systems (COFS) at The University of Western Australia to develop improved procedures for site investigation practice in deepwater soft clays. Extensive theoretical studies were carried out to investigate the effect of strain-softening, strain-rate dependency of strength, and strength anisotropy on the T-bar and ball penetration resistance (Randolph and Andersen 2006; Zhou and Randolph 2009*a*, 2009*b*). In addition, field and laboratory data from 11 offshore and three onshore sites were interpreted to form a worldwide database. With this database, results from CPTUs and T-bar and ball penetration tests were correlated to undrained shear strength determined from triaxial and direct simple shear tests on high-quality samples and from vane shear tests. From these studies, key soil characteristics that influence the relationship between undrained shear strength and penetration resistance were identified.

This paper summarizes the key outcomes of the joint industry project in terms of recommendations for the design of in situ tools and associated testing procedures with the aim of improving the accuracy, reliability, and consistency of the in situ test data. In addition, guidelines for interpretation of the penetration test data are provided, with particular focus on estimating intact and remoulded undrained shear strengths from the penetration resistance measured by the different penetrometers. To maximize the potential of in situ tools in determining design parameters for deepwater soft clays, further developments of the in situ tools and testing procedures are also proposed. Finally, guidance is provided on which type of in situ tool should be used for optimal characterization of deepwater soft clays, depending on soil conditions and the engineering problem under consideration. Note that the guidelines provided in this paper are proposed for offshore in situ testing in nonfissured deepwater soft clays. Nonetheless, the majority of guidelines provided may also be applicable for onshore in situ testing in nonfissured soft clays.

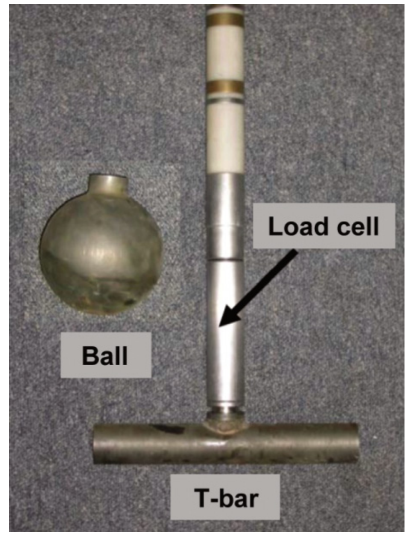
Equipment and testing procedures

In situ testing tool geometry

Piezocone

The equipment for CPTUs should be in accordance with internationally recognised guidelines and standards, notably the International Reference Test Procedure (IRTP) published by the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE 1999); NORSOK standard G-001 (Standards Norway 2004), American Society for Testing and Materials (ASTM) standard D5778-07 (ASTM 2007), and EN-ISO standard 22476-1 (ISO/CEN 2007). As equipment requirements for the piezocone are well documented in these references, they are not repeated here.

Fig. 1. T-bar and ball penetrometers.



In offshore CPTUs, a wide range of cone penetrometer sizes are used. For downhole testing, the standard onshore size with a 1000 mm^2 tip area (35.7 mm diameter) is the most common, but 1500 mm^2 cone penetrometers (43.7 mm diameter) are used extensively for seabed mode testing. Studies have shown that cone penetration resistances measured by cone penetrometers with cross-sectional areas of 500 to 1500 mm^2 are very similar (De Ruiter 1982). As such, piezocone penetration tests carried out with 1000 and 1500 mm^2 cone penetrometers are acceptable, and EN-ISO 22476-1 (ISO/CEN 2007) allows for cones of 500 and 2000 mm^2 (25 and 50 mm diameter, respectively). Cone penetrometers with cross-sectional areas as small as 100 mm^2 (11.3 mm diameter) have also been used in conjunction with mini seabed frames (Lunne 2001), but these are not in accordance with the EN-ISO 22476-1 standard.

T-bar penetrometer

At present, the only standard that covers T-bar penetration testing is NORSOK G-001 (Standards Norway 2004). This standard recommends the use of a T-bar penetrometer of 40 mm diameter and 250 mm length, which gives a projected area of 10 000 mm^2 (i.e., 10 times the standard cone rod size). There are limited results on the effects of varying T-bar dimensions and geometry on net penetration resistance reported in the literature. For soft clays, the centrifuge test results reported by Chung and Randolph (2004) and field test results reported by Weemees et al. (2006) and Yafrate et al. (2007) showed no effect on the net penetration resistance for length to diameter ratios within a range of 4 to 10 (covering ratios of the projected T-bar area to that of the shaft of 6.4 to 15). Therefore, it is recommended that, if a T-bar penetrometer smaller than the NORSOK standard size is used, the length to diameter ratio should not be less than 4. However, it is also recommended that the cross-sectional area of the connecting shaft should be no more than 15% of the projected area of the T-bar, with a diameter not larger than that of the T-bar. On the basis of field tests that have shown at most 5% reduction in penetration resistance for T-bars with a machined smooth surface, it is recommended that the surface be lightly sand-blasted, as recommended in NORSOK G-001 (Standards Norway 2004).

Ball penetrometer

The ball penetrometer is presently not standardized and there are very few results reported regarding the effects of dimensions and probe material type on test results. A new International Standards Organization (ISO) standard that is expected to be published in 2011–2012 prescribes a ball penetrometer of diameter 113 mm, giving a projected area of 10 000 mm^2, directly attached to standard cone rods (Chung and Randolph 2004; Yafrate and DeJong 2006; Yafrate et al. 2007). The new standard will also allow for a ball with a smaller diameter, with 60 mm as a minimum (i.e., projected area of 2800 mm^2). The main criterion should be to maintain the ratio of the projected area for the connecting shaft behind the ball to the projected area for the ball below about 15%, as for the T-bar. Similar to the T-bar, it is recommended that the surface of the ball steel surface be lightly sandblasted.

Vane

The vane blade dimension for vane shear tests should comply with internationally accepted standards such as NORSOK G-001 (Standards Norway 2004) and ASTM D2573-08 (ASTM 2008). With these standards, the diameter of the vane blade should be in the range of 40 to 65 mm and the ratio of height to diameter should be 2. The thickness of the vane blade specified in these standards ranges from 1.6 to 3.2 mm. To minimize soil disturbance due to vane insertion, the vane blade should be kept as thin as possible (La Rochelle et al. 1973; Roy and Leblanc 1988; Cerato and Lutenegger 2004), preferably with a perimeter ratio (= $4e/\pi d_{\mathrm{vane}}$, where e is the vane blade thickness and d_{vane} is the vane diameter) no more than 3%. The selection of vane size depends on the strength of the soil to be tested. The ratio of vane diameter to push shaft diameter should be at least 3 to minimize the effect of soil consolidation around the shaft on the measured strength.

Data accuracy

Sensor calibration and temperature stability

Sensors (load cell and pore pressure transducer) for cone, T-bar, and ball penetrometers should be calibrated in accordance with international standards (e.g., NORSOK G-001 (Standards Norway 2004); ASTM D5778-07 (ASTM 2007); EN-ISO 22476 (ISO/CEN 2007)) for the cone penetrometer.

Shifts in reference readings can cause significant errors in penetration test measurements, especially for tests in soft clay. One of the main reasons for a shift in a reference reading is the shift in sensor output due to temperature change (Lunne et al. 1986). Therefore, it is recommended that the penetrometer sensors be designed appropriately to provide temperature compensation. Guidelines are given in the above-mentioned standards.

Data acquisition

Data acquisition requirements for the CPTU and vane shear test should be in accordance with international standards (NORSOK G-001 (Standards Norway 2004); ASTM D5778-07 (ASTM 2007); ISSMGE (1999); EN-ISO 22476-1 (ISO/CEN 2007); ASTM-D2573-08 (ASTM 2008). It is recommended that the data acquisition requirements for T-bar and ball penetration tests should be in accordance with the requirements for the piezocone penetration test, but with the important addition to log the resistance during extraction as well as penetration. The following outlines some important issues that require special attention for penetration testing, particularly for T-bar and ball penetration tests.

According to the international standards, the maximum data logging interval for the CPTU in soft clay is 20 mm. In (most) practice, however, the logging of cone parameters is more frequent than the required logging frequency. It is recommended that the data-logging frequency for T-bar and ball penetration tests be similar to that for the CPTU. However, as the recommended minimum strokes for cyclic T-bar and ball penetration tests are ±0.15 and ±0.20 m, respectively, a maximum measurement interval of 10 mm is recommended during cyclic T-bar and ball penetration tests. This is to ensure sufficient data points are acquired for interpretation of the cyclic penetration test result.

Testing procedure

Penetration tests

It is important to minimize errors when taking the reference readings of the sensors before the start of the penetration test to reduce uncertainties in the tool measurements. All the sensors must be allowed to stabilize at the local temperature before taking the reference readings for a penetration test (either seabed mode or downhole mode testing). In addition, any pre-embedment of the penetrometer into the soil before taking the reference readings at the beginning of a penetration test should be avoided. For high-quality testing in soft clay, it is essential that the reference readings be recorded and documented as outlined later.

Monotonic CPTUs and T-bar penetration tests should be carried out in accordance with international standards such as NORSOK G-001 (Standards Norway 2004). Although, at present, there is no standard for the ball penetration test, it is recommended that the test be carried out in accordance with the NORSOK G-001 standard for the T-bar penetration test. The monotonic penetration and extraction T-bar and ball penetration testing should be carried out at a steady rate of about 20 mm/s, or 0.5 diameters per second for the T-bar and about 0.25 diameters per second for the ball. For penetrometers of different sizes, it is preferable to maintain the same rate in terms of diameters per second, resulting in the same average shear strain rates in the soil. It is recommended that both penetration and extraction resistances be measured during T-bar and ball penetration tests. Although it is not specified in the international standards, recording (and reporting) of piezocone data during extraction of the piezocone is also recommended as this may help in quality control of the measurements.

While cyclic T-bar and ball penetration tests may be carried out to estimate remoulded undrained shear strength, it is recommended that at least one cyclic test be carried out for every test location to provide additional input for checking the reference readings of the sensors. It is also recommended that 10 cycles of penetrating and extracting the T-bar and ball penetrometer through a minimum stroke of ±0.15 m for the T-bar and ±0.20 m or ±3 diameters (whichever is the greater) for the ball penetrometer be undertaken. The cyclic test should be performed during the penetration phase of the test because partial consolidation of soils around the push rod will result in higher extraction and remoulded resistances being measured if the cyclic penetration test is carried out during the extraction phase of the test. The penetration and extraction rate for the cyclic penetration test should be the same as for the monotonic penetration stage, at least during the final cycle where the remoulded penetration resistance is assessed.

Vane shear tests

Vane shear tests should be carried out in accordance with internationally accepted standards such as NORSOK G-001 (Standards Norway 2004) and ASTM-D2573-08 (ASTM 2008). The recommended rotation rate for initial rotation to peak torque (or intact undrained shear strength) should be in the range of 0.1° to 0.2° per second. NORSOK G-001 specifies that the time from the instant when the desired testing depth has been reached to the beginning of the test (waiting time) should be 2 to 5 min. After the peak torque is measured, and if the remoulded undrained shear strength is required, NORSOK G-001 specifies that the remoulded undrained shear strength should be measured after at least 10 rotations at a rate faster than 4 revolutions/min (24°/s) and until a constant torque over 45° continuous rotation has been reached. At the end of the rapid rotations the remoulded undrained shear strength is to be measured without delay at the same rate as for the intact undrained shear strength.

Existing vane shear apparatus available for offshore vane shear testing have not been able to conduct quick rotations and hence it has been impractical to do even 10 quick rotations.

Typically, during offshore vane shear testing, residual undrained shear strength is measured after one rotation at a rotation rate of 1°/s. Some engineers use this as the remoulded shear strength, but the value is likely to be an overestimation of the true remoulded shear strength. The extent to which the residual vane strength reflects development of discrete shear surfaces with reduced frictional characteristics is unclear, but evidence suggests that this is not the case and the residual shear strength recorded after a single rotation of the vane is generally greater than fully remoulded strengths obtained in the laboratory or in cyclic penetrometer tests in the field.

Therefore, to measure remoulded undrained shear strengths offshore reliably (similar to onshore practice), offshore geotechnical contractors are encouraged to develop equipment that can conduct 20 rotations in say 5 min, but still allow rotation rates of 0.1° to 0.2° per second for the intact and remoulded undrained shear strength measurements.

Offshore deployment of in situ tools using seabed mode

Offshore in situ testing can be carried out using downhole (i.e., at the base of a drill string) and seabed modes (i.e., from a frame placed on the seabed). For deepwater sites where shallow anchoring systems are anticipated, in situ tests are normally carried out in seabed mode.

There are several issues for the deployment of seabed frames that need to be considered to improve the reliability of the subsequent test data. During touch-down, the seabed frame may sink into soft surficial sediments due to its self-weight. As such, careful control of depths for the in situ tests is extremely important to avoid any pre-embedment of in situ tools into the soil before the start of the test, which can lead to errors in the recorded reference readings for the sensors. In addition, when measurement of soil properties of the upper 1 to 2 m of the seabed is of interest, it is very important to ensure that the seating of the seabed frame on the seabed does not disturb the soil in the vicinity of the in situ test, and that the bearing stresses imposed by the seabed frame do not affect the test data.

The effect of the seabed frame on the test results may be reduced by careful consideration of the following:

1. The seabed frame should be designed so that its self-weight is sufficient to provide the reaction force required for carrying out the in situ test, but not so large that it disturbs the soft seabed.
2. Skirts should be used on the periphery of the seabed frame to transfer weight of the seabed frame to stiffer soil, reducing penetration of the seabed frame into the seabed.
3. The contact area (footprint) of the seabed frame should be designed to include a sufficiently large opening where the in situ tool is pushed into the seabed, or for the bearing areas of multi-foot seabed frames to be far from the centreline of the in situ test.

To evaluate the effects of the seabed frame on the in situ measurements, it is recommended that the touch-down of the frame on the sea floor or any penetration of the frame into the seabed soils be monitored. One way to achieve this is to mount video cameras on the frame, as has already been adopted on some commercial seabed frames.

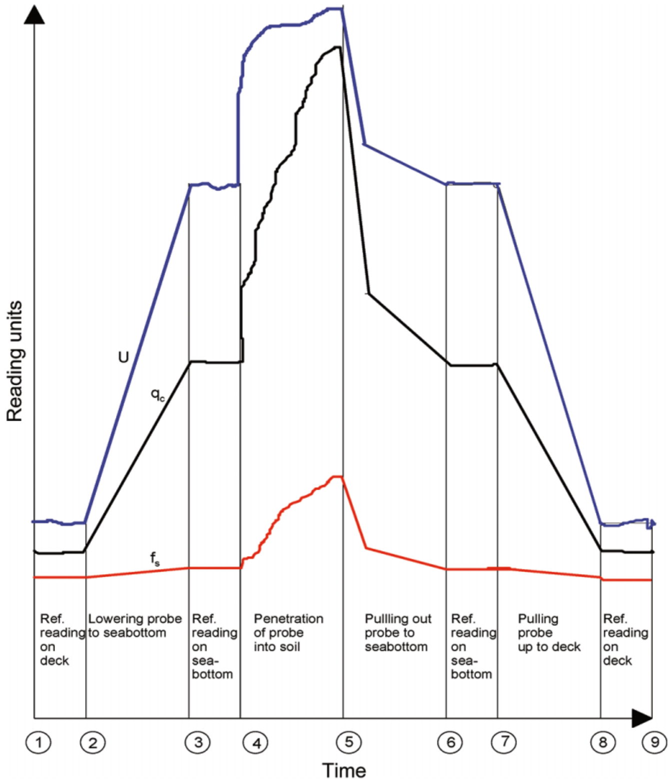
Recommendation for measurement and documentation of reference readings

For in situ testing in very soft soils and when high-accuracy measurement is of special concern, or class 1 accuracy in accordance with NORSOK G-001 (Standards Norway 2004) is required, it is recommended that the data during all stages of a piezocone, T-bar or ball penetrometer deployment and testing be recorded for the assessment of test quality. Recommendations for the measurement and documentation of reference readings at stages 1 to 9 shown in Fig. 2 are applicable for seabed mode testing and will be included in a new ISO standard for marine soil investigation that is expected to be published in 2011–2012. A similar scheme will also be included for downhole testing.

The data recorded for stages 1 to 9 shown in Fig. 2 should be presented together with the "standard" presentation of the measured (e.g., q_c, u, f_s) and the derived parameters (e.g., q_t, q_{net}, $F_r = f_s/q_{net}$, and $B_q = \Delta u/q_{net}$) as required by the international standards. The data should be presented in plots of all sensor readings versus time as shown in Fig. 2 and in a table with all sensor reference readings in engineering units recorded at stages 2 to 9 shown in Fig. 2. The recorded data can then be used to scrutinize the test results. As the total in situ vertical stress, σ_{v0}, is needed to compute q_{net}, the basis for the estimation of σ_{v0} should be given. In addition, the pore pressure and tip resistance data should be recorded during lowering of the penetrometer to the seabed and used to confirm the calibrated net area ratio for the piezocone.

For high-quality testing, the difference between the refer-

Fig. 2. Scheme for taking reference readings for seabed in situ testing (stage numbers are indicated along the time axis). Ref., reference (ISO/CEN 2007).



ence readings recorded at stages 7 and 4 and stages 9 and 2 should be small. As a provisional suggestion, the recommended limiting values for the difference between the reference readings recorded at stages 7 and 4 and stages 9 and 2 for each of the piezocone sensors are (with all maximum readings taken relative to the seabed)

> q_c — The larger of 35 kPa or 5% of the maximum reading in the layer being tested.
> u — The larger of 10 kPa or 2% of the maximum reading in the layer being tested.
> f_s — The larger of 5 kPa or 10% of the maximum reading in the layer being tested.

For T-bar or ball penetration tests, the recommended limiting difference for $q_{\text{T-bar}}$ or q_{ball} is the larger of 10 kPa or 5% of the maximum reading in the layer being tested.

The limiting values recommended above for piezocone measurements are in accordance with the requirements for application class 1 of the new European standard EN-ISO 22476-1 (ISO/CEN 2007). If the difference between reference readings at stages 7 and 4 and stages 9 and 2 exceeds the recommended limiting maximum value for any sensor, it is recommended that a comment be included on each plot of the test results, stating the magnitude of the differences.

The above recommended procedure is valid for seabed mode testing where the seabed frame is recovered to deck for each test. The procedure has to be modified if the seabed frame is moved from one testing location to another without being recovered to deck. Although the above recommendations are for the penetration tests, similar concepts should also be applied for vane shear tests conducted from a seabed frame. For some projects, it may be necessary to make more strict requirements than those suggested above.

Site investigation contractors are encouraged to develop in situ tools and data acquisition systems that minimize the shift in zero reference readings to values lower than those recommended above. As shown by Randolph et al. (2007), with the T-bar and ball cyclic penetration tests, the relative symmetry of the penetration and extraction resistance profile about the zero line can provide additional input for checking the reference readings of penetrometer sensors. This is one of the reasons for the recommendation that at least one cyclic penetration test be carried out at each test location.

Presentation of data

The presentation of results from piezocone and T-bar monotonic penetration tests and vane shear tests should be in accordance with international standards (ISSMGE (1999); NORSOK G-001 (Standards Norway 2004); ASTM D5778-07 (ASTM 2007); EN-ISO 22476-1 ISO/CEN 2007; ASTM-D2573-08 (ASTM 2008)). It is recommended that results for the monotonic ball penetration test be presented in accordance with NORSOK G-001 (Standards Norway 2004) for the T-bar penetration test. For T-bar and ball penetration tests, in addition to the penetration resistance profile, profiles of extraction resistance and ratio of extraction to penetration resistance should also be presented.

Cyclic T-bar and ball penetration test results should be presented in plots of resistance profile and degradation factor against cycle number as shown in Fig. 3. It is suggested that the cycle number for the initial penetration should be taken as 0.25 and initial extraction taken as 0.75 and so forth (Randolph et al. 2007). The degradation factor is calculated by dividing the average (absolute) net resistance measured at each half-cycle (either penetration or extraction) by the net penetration resistance measured during initial penetration. The average net resistance for each half-cycle should be taken at the central part of each cyclic stroke to avoid the influence of conditions at the extremes of the cyclic zone. The net resistance is obtained by correcting the measured resistances for the overburden pressure and pore pressure effects as will be discussed in the next section.

Correction of measured penetration resistance

Before measured penetration resistances are used for the estimation of soil properties, they have to be corrected appropriately for the unequal pore pressure and overburden pressure effects. The measured piezocone resistance is corrected to total tip resistance, q_t using the following relationship (Lunne et al. 1997):

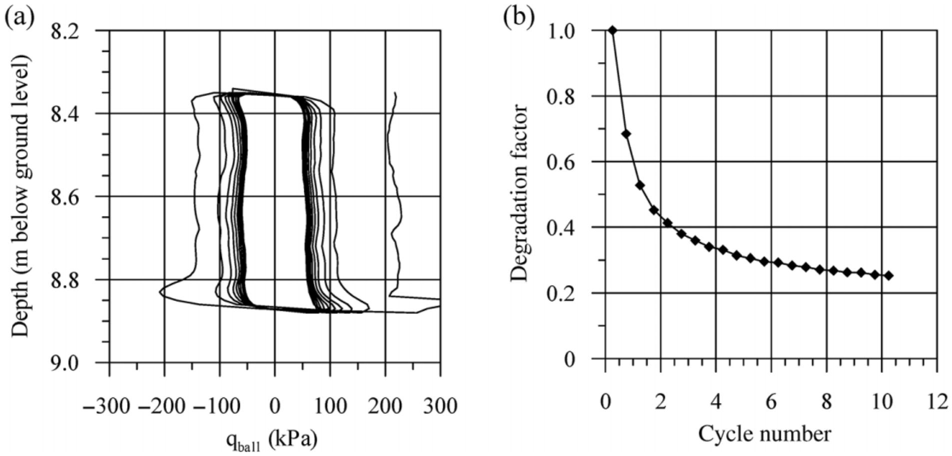
$$[1] \qquad q_t = q_c + u_2(1 - \alpha)$$

where u_2 is the measured pore pressure at the shoulder of the cone and α is the net area ratio. Typical α values for piezocones used offshore range from 0.6 to 0.8. The net piezocone penetration resistance is then calculated as

$$[2] \qquad q_{net} = q_t - \sigma_{v0}$$

where σ_{v0} is the in situ total overburden stress (obtained by

Fig. 3. Example for presentation of cyclic penetration test results: (a) depth versus ball penetration resistance; (b) degradation factor versus cycle number.



integrating γ_{bulk} with depth, where γ_{bulk} is total unit weight of the soil).

Similarly, the T-bar and ball penetration resistances measured during the initial penetration and cyclic penetration tests should also be corrected for the unequal pore pressure and overburden pressure effects using the following simplified expression (Chung and Randolph 2004):

$$[3] \qquad q_{\text{T-bar}} \text{ or } q_{\text{ball}} = q_m - [\sigma_{v0} - u_0(1-\alpha)]A_s/A_p$$

where $q_{\text{T-bar}}$ and q_{ball} are the net penetration resistances for T-bar and ball penetrometer, respectively; q_m is the measured resistance; u_0 is the hydrostatic water pressure; α is the net area ratio (as defined above and generally ranges from 0.6 to close to unity); A_s is the cross-sectional area of the connecting shaft; A_p is the projected area of the penetrometer in a plane normal to the shaft. A slightly more refined version of eq. [3] was presented by Randolph et al. (2007), but the difference was estimated to be less than 3% during penetration and eq. [3] avoids the need for accurate measurement of u_2 during T-bar and ball penetration tests. The net remoulded T-bar and ball penetration resistances will be denoted as $q_{\text{T-bar,rem}}$ and $q_{\text{ball,rem}}$, respectively, in this paper.

Interpretation in terms of intact undrained shear strength

The worldwide database established from the joint industry project formed the basis of a correlation study between the penetration test measurements (i.e., net penetration resistance and pore pressure) measured during the initial penetration of the penetrometer and intact undrained shear strength (s_u) (Low et al. 2010). This study indicated that the cone N_{kt} (= q_{net}/s_u) and $N_{\Delta u}$ (= $(u_2 - u_0)/s_u$) factors are influenced by the rigidity index of the soil. In contrast, full-flow penetrometer $N_{\text{T-bar}}$ (= $q_{\text{T-bar}}/s_u$) and N_{ball} (= q_{ball}/s_u) factors based on the average of triaxial and simple shear undrained shear strengths (s_{uave}) and vane shear strengths (s_{uvane}) are relatively independent of secondary soil characteristics, apart from a slight effect of strength anisotropy, at least for soil with a strength sensitivity $\leq$ 8. The overall statistics showed similar levels of variability of the resistance factors, with low coefficients of variation (ranging from 0.10 to 0.20), for all three types of penetrometer. However, due to its high variation (with coefficient of variation ranging from 0.20 to 0.35) and strong dependency on the rigidity index, $N_{\Delta u}$ is not recommended for estimating s_u without site-specific correlation.

Table 1 summarizes the *N*-factors recommended by Low et al. (2010) for the estimation of s_u from penetration resistances. Specific *N*-factors for the Gulf of Guinea were also recommended, as six of the offshore sites considered in the project were from that region. The recommendations given in Table 1 should only be used for the estimation of s_u for soil with a strength sensitivity $\leq$ 8 and may be updated in light of local experience. However, extreme caution should be exercised if the correlations for a new site fall outside the ranges given, as this may indicate questionable data. For the ranges given in Table 1, the lower value should be used to compute s_u when it is conservative to adopt high shear strength and the higher value used when it is conservative to adopt low shear strength.

From the comparison between the ball and T-bar penetration resistances (both initial and remoulded), Low et al. (2010) found that the ball penetration resistance may be approximately 5% higher than the T-bar penetration resistance. Due to limited ball penetrometer data available in the database, they proposed adopting $N_{\text{ball}} = N_{\text{T-bar}}$, but as more data become available it may prove appropriate to distinguish between N_{ball} and $N_{\text{T-bar}}$.

It should be noted that, unlike in onshore practice, vane shear strengths are rarely adjusted by any correction factors in offshore practice (Kolk et al. 1988). This is based on correlations with other measurements of shear strength, and may arise (at least partly) from increased soil disturbance during vane insertion for offshore conditions.

Interpretation in terms of remoulded undrained shear strength

The remoulded undrained shear strength (s_{ur}) can be determined from cyclic T-bar and ball penetration tests. The "remoulded" penetration (or extraction) resistance, $q_{\text{T-bar,rem}}$ and $q_{\text{ball,rem}}$, measured at the end of the cyclic penetration test (normally 10 cycles) may be used to estimate s_{ur} using an appropriate remoulded *N*-factor (denoted as N_{rem} in this paper).

Table 1. Recommended *N*-factors (Low et al. 2010).

		Recommended *N*-factor			
		All data		Gulf of Guinea	
N-factor or N_{rem}-factor	Definition	Mean	Range	Mean	Range
$N_{kt,suc}$	q_{net}/s_{uc}	12.0	10.0–14.0	12.5	10.5–14.5
$N_{kt,suave}$	q_{net}/s_{uave} or q_{net}/s_{udss}[a]	13.5	11.5–15.5	13.5	11.5–15.5
$N_{T\text{-}bar,suc}$	$q_{T\text{-}bar}/s_{uc}$	10.5	8.5–12.5	10.5	8.5–12.5
$N_{T\text{-}bar,suave}$	$q_{T\text{-}bar}/s_{uave}$ or $q_{T\text{-}bar}/s_{udss}$[a]	12.0	10.0–14.0	12.0	10.0–14.0
$N_{T\text{-}bar,rem,UU}$	$q_{T\text{-}bar,rem}/s_{ur,UU}$	20.0	13.0–27.0	—	—
$N_{T\text{-}bar,rem,fc}$	$q_{T\text{-}bar,rem}/s_{ur,fc}$	14.5	12.5–16.5	—	—
$N_{T\text{-}bar,rem,vane}$	$q_{T\text{-}bar,rem}/s_{ur,vane}$	14.0	12.0–16.0	—	—

Note: Shear strengths: s_{uc}, triaxial compression; s_{uave}, average of triaxial compression and extension and simple shear; s_{udss}, simple shear; $s_{ur,UU}$, $s_{ur,fc}$, and $s_{ur,vane}$, remoulded UU, fall cone, and vane, respectively.

[a]When triaxial extension strength (s_{ue}) is not available.

Although the soil may not be fully remoulded at the end of the 10th cycle of the test, s_{ur} can still be estimated from $q_{T\text{-}bar,rem}$ and $q_{ball,rem}$ as long as the N_{rem}-factor is calibrated with $q_{T\text{-}bar,rem}$ and $q_{ball,rem}$ measured at the 10th cycle, which represents a practical length of test. In the correlation between s_{ur} and the average of remoulded penetration and extraction resistances measured during the 10th cycle of the cyclic penetration test, Low et al. (2010) found that the N_{rem}-factors were higher than those for intact strength (s_u) and showed a slight increase with increasing strength sensitivity, but no consistent trend with index properties. This finding confirms theoretical arguments and results from numerical analysis (Randolph and Andersen 2006; Zhou and Randolph 2009*b*), which show that the ratios $q_{T\text{-}bar}/q_{T\text{-}bar,rem}$ and $q_{ball}/q_{ball,rem}$ (resistance sensitivity) will always be less than the shear strength sensitivity (s_u/s_{ur}).

The recommended N_{rem}-factors for estimating s_{ur} from the remoulded resistance are summarized in Table 1. Just as for the interpretation of s_u, the recommended N_{rem}-factors should only be used to estimate s_{ur} for soil with a strength sensitivity ≤ 8 for which the proposal of $N_{rem,ball} = N_{rem,T\text{-}bar}$ is still valid. Note that the recommended N_{rem}-factors depend on the way in which s_{ur} has been measured (Randolph and Andersen 2006).

Interpretation in terms of other soil parameters

Low et al. (2011) showed that comparison of the penetration resistance measured from the different penetrometers may give some indication of the rigidity index of the soil. In addition, Low et al. (2007, 2008*a*) showed that, by modifying the testing procedures and fitting the penetrometers with pore-water pressure sensors, full-flow penetrometers have excellent potential in determining parameters for consolidation and the strain rate dependency of soil strength.

Evaluation of rigidity index

The ratio of net T-bar or ball penetration resistance to cone penetration resistance (= $q_{T\text{-}bar}/q_{net}$ or q_{ball}/q_{net}) has been found to follow the theoretical trends for the cone resistance to increase with the rigidity index, G/s_u (Low et al. 2011). The best quantitative agreement was obtained using the small-strain rigidity index, G_0/s_{uave}, with small strain stiffness, G_0, measured by in situ seismic cone tests (Fig. 4). Based on this observation, they suggested that, in the absence of accurate shear modulus data, the small-strain rigidity index, G_0/s_{uave}, may be estimated in the range 200 to 300 for $q_{T\text{-}bar}/q_{net}$ of unity, increasing to ~1000 for $q_{T\text{-}bar}/q_{net}$ of 0.75. They also suggested that seismic shear wave data can be used as a check for the measured ratios of cone and T-bar penetration resistances.

Evaluation of strength dependency on strain rate

The undrained shear strength of soil is affected by the applied shear strain rate. Therefore, the strain rate dependency of soil strength is an important issue, both in interpretation of test data and in the choice of what shear strength is appropriate for different design applications. Varying the penetration rate during a penetration test is one way to assess the influence of strain rate dependency of soil strength in situ (Chung et al. 2006; Low et al. 2008*a*). Alternatively, the variable-rate vane shear test, as suggested by Peuchen and Mayne (2007), may be used for the same purpose. The strain rate dependency of soil strength (or the rate coefficient) may then be evaluated by fitting the variable-rate penetration test data or variable-rate vane shear test data to a rate function such as the semi-logarithmic or hyperbolic sine strain rate law shown in Table 2. Alternatively, the data may be fitted using a power law relationship (Lehane et al. 2009).

Figure 5 shows an example of the effect of penetration rate on the penetration resistance obtained from a series of variable-rate T-bar and ball penetration tests and variable-rate cyclic T-bar and ball penetration tests performed at a soft clay site located in Western Australia (Low et al. 2008*a*). The site comprises lightly overconsolidated, soft silty clay with plasticity index and strength sensitivity ranging from 40% to 70% and 3.5 to 4.5, respectively. The yield stress ratio of the clay is approximately 1.4.

It may be noted in Fig. 5 that, provided undrained conditions are maintained, $q_{T\text{-}bar}$ and q_{ball} decrease compared with that for a standard rate test (penetration rate of 20 mm/s) as the penetration rate is reduced. Low et al. (2008*a*) found that the rate coefficients deduced from in situ T-bar and ball penetration tests ranged between 0.10 and 0.21. The rate coefficients depend slightly on the rate function and reference penetration rate used to fit the data, but the rate coefficients for the ball resistance, q_{ball}, always tended to be lower than

Fig. 4. Variation of (*a*) $q_{T\text{-bar}}/q_{net}$ and (*b*) q_{ball}/q_{net} with rigidity index (I_r) (after Low et al. 2011). Δ, in situ normalized shear stress (= $(\sigma_{v0} - \sigma_{h0})/2s_u$); α_s, interface friction ratio. (The data based on G_0/s_{uave} are circled in the plot and the data for "poor" quality data are bracketed.)

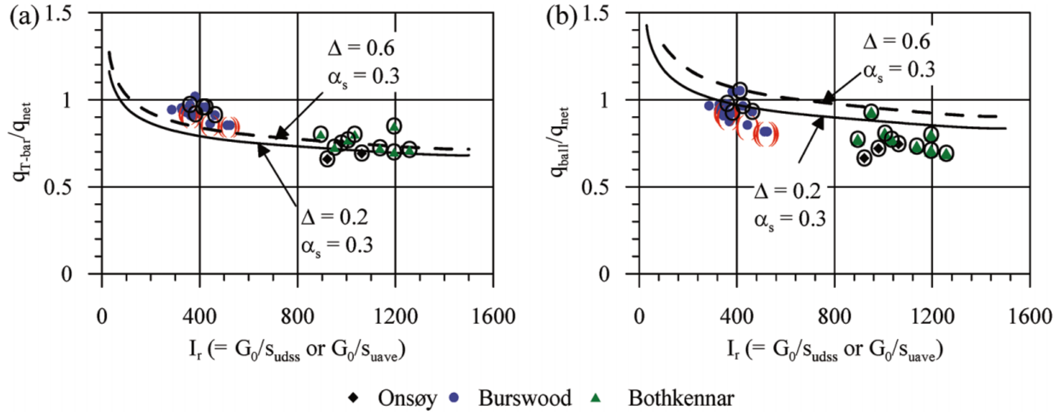


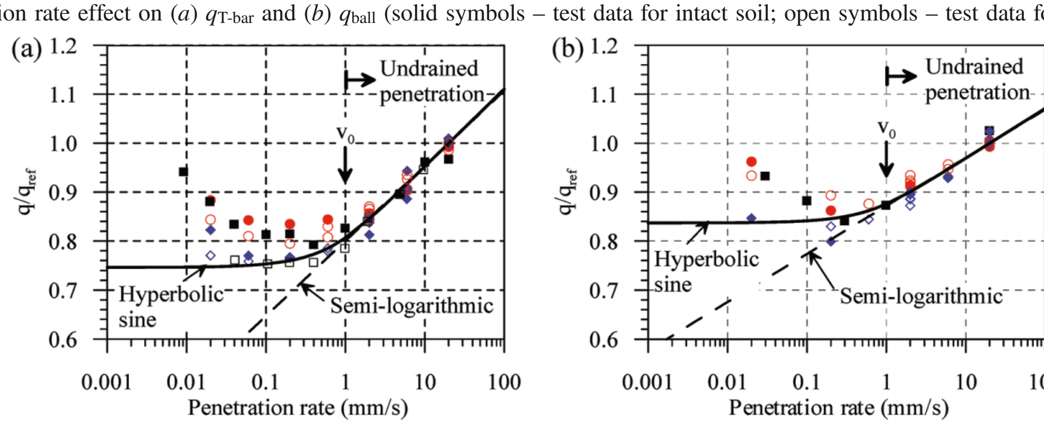
Table 2. Best-fit hyperbolic sine and semi-logarithmic rate coefficient (Low et al. 2008*a*).

Penetrometer	Hyperbolic sine rate coefficient, μ $\frac{q}{q_{ref}} = \frac{1 + \{[\mu/\ln(10)]\}[\sinh^{-1}(v/v_0)]}{1 + \{[\mu/\ln(10)]\}[\sinh^{-1}(v_{ref}/v_0)]}$	Semi-logarithmic rate coefficient, μ $\frac{q}{q_{ref}} = 1 + \mu \log\left(\frac{v}{v_{ref}}\right)$
T-bar (intact)	0.21	0.15 (0.19)[a]
T-bar (remoulded)	0.21	0.15 (0.19)[a]
Ball (intact)	0.12	0.10 (0.11)[a]
Ball (remoulded)	0.12	0.10 (0.11)[a]

Note: q_{ref} was taken as penetration resistance measured at the penetration rate of 20 mm/s; v_{ref} was taken as penetration rate of 20 mm/s; μ is the rate coefficient that quantifies the change in strength with an order of magnitude change in rate.

[a]Values in brackets were obtained by replacing v_{ref} with v_0 (= 1 mm/s) and q_{ref} with q measured at the penetration rate of v_0. v_0 is the penetration rate at which the undrained strain rate effect start to decay towards zero.

Fig. 5. Penetration rate effect on (*a*) $q_{T\text{-bar}}$ and (*b*) q_{ball} (solid symbols – test data for intact soil; open symbols – test data for remoulded soil).



those for the T-bar resistance, $q_{T\text{-bar}}$ (Table 2). They also observed no difference in the rate coefficients for intact and remoulded conditions, for either $q_{T\text{-bar}}$ or q_{ball}. This similarity in rate effects for both intact and remoulded conditions suggests that performing variable-rate penetration tests in remoulded soil may be advantageous because it is much easier to measure a consistent resistance profile for both standard-rate and variable-rate tests in remoulded soil.

Variable penetration test data may allow the penetration resistance to be used directly in design for different applications, depending on the shearing rates imposed (Randolph et al. 2007). However, it has been shown experimentally that the strain rate dependency of soil strength tends to decrease with decreasing strain rate (e.g., Lunne and Andersen 2007; Peuchen and Mayne 2007). Therefore, caution should be exercised when extrapolating the penetration resistance for direct application in design where this involves extrapolation of shear strength over several orders of magnitude difference in strain rate. This is particularly true if the semi-logarithmic strain rate law is used to model the strain rate dependency.

Characterization of near-seabed surface sediment

The strength profile in the upper 1 to 2 m of the seabed is critical for pipeline, flowline, and riser design, and yet is the most difficult to assess by means of in situ testing and soil sampling. From a review of some existing approaches for the strength characterization of seabed surficial sediments, Low et al. (2008*b*) concluded that performing in situ strength tests within box core samples is the most reliable means of characterizing the undrained shear strength of soft surficial sediments. While a miniature vane shear test can be used to determine the intact and remoulded undrained shear strength profiles in a box core sample, miniature penetration tests take much less time and provide a continuous strength profile throughout the depth (Low et al. 2008*b*; Low and Randolph 2010). In addition, penetration testing should provide excellent definition of any crustal features. The intact and remoulded undrained shear strength can be estimated from the penetration resistance using the *N*-factors recommended in Table 1.

At the moment, commercially available box corers are capable of recovering soil samples from depths up to 0.5 m below the seabed (e.g., Borel et al. 2005). In view of the excellent potential of box corers in recovering intact samples of very soft surficial sediments, development of box corers capable of taking deeper samples without increasing the level of disturbance should be encouraged. The use of a remotely operated vehicle (ROV) is also a very interesting alternative for deploying in situ tools to undertake in situ tests to 1 to 2 m below the seabed without disturbing the seabed soil and is already commercially available (e.g., Newson et al. 2004).

Future development of in situ tools and testing techniques

A number of future developments of in situ tools and testing techniques are suggested to maximize the potential and reliability of in situ tools, particularly full-flow penetrometers, in the characterization of deepwater soft clays. The suggestions include incorporating additional sensors to existing in situ tools, improving on existing sensors, and improving current testing equipment.

Incorporation of pore pressure sensor on full-flow penetrometer

The recent development of full flow penetrometers has involved fitting pore pressure sensors to obtain parameters in addition to the penetration resistance, thus enhancing the capability of full-flow penetrometers for estimating geotechnical parameters other than undrained shear strength. Kelleher and Randolph (2005) and Peuchen et al. (2005) showed the excellent potential of the T-bar and ball penetrometer with pore pressure measurement (i.e., piezo T-bar and piezoball) for assessing soil stratigraphy. Kelleher and Randolph (2005) measured pore pressure at the mid-height of the ball while Peuchen et al. (2005) measured the pore pressure along the axis of the T-bar (with one sensor at the centre and one at the edge) and at the tip of the ball. In the characterization of peaty soil, Boylan and Long (2006) also showed that the pore pressure data measured at a location of one-third the ball diameter from the tip of the ball appeared to be useful in identifying the relative humification within a peat deposit. Low et al. (2007) and DeJong et al. (2008) showed that, as for the piezocone, piezoball dissipation tests (with pore pressure measurement at the mid-height of the ball) may be used to estimate the consolidation parameters for a soil. Theoretically, the pore pressure distribution around the T-bar and ball penetrometer should be largely independent of the soil rigidity index. This may provide an advantage of the piezo T-bar or piezoball over the piezocone in estimating the in situ coefficient of consolidation from dissipation tests.

The existing international standard such as the NORSOK G-001 standard (Standards Norway 2004) does not mention pore pressure measurement in connection with T-bar (or ball) penetration testing. As the experience in pore pressure measurement for T-bar and ball penetration tests is still very limited, it is recommended that tests be carried out at well-documented test sites with different pore pressure measuring locations to find the optimal measuring location and allow standardization of pore pressure measurements on full-flow penetrometers.

Sensor compensated for ambient pressure

In deep water, the load cell and pressure sensors for penetrometers are preloaded by the high ambient water pressure at the seabed and consume a significant portion of the measurement range. As a result, high-capacity sensors are required and this limits the sensitivity in respect to measuring the very small incremental resistance from the soil during penetration in soft clays. To increase the accuracy or sensitivity of the measurements, geotechnical contractors should be encouraged to develop and use sensors that measure differential pressure or resistance relative to ambient water pressure. Although this is somewhat less important for the T-bar and ball penetrometer compared with the cone, because of the 10-fold greater projected area relative to the connecting shaft, sensors compensated for ambient pressure would further improve accuracy of the tests. Pressure-compensated cone penetrometers are available commercially (Meunier et al. 2004; Boggess and Robertson 2010).

Variable-rate penetration test

In view of the benefits of the variable-rate penetration test in evaluating strain rate dependency of soil strength in situ and consolidation conditions during the penetration test, it is recommended that the industry move towards incorporating intervals of varying rate penetration tests through appropriate design of the test control software. In some cases this will require associated advances in equipment to increase the range of penetration rates at which controlled testing can be carried out and to increase the data-logging rate. A provisional target would be to vary the penetration rate in steps of between 1 and 2 orders of magnitude, with a minimum advance at each step of 0.1 m for cone or T-bar and 2 diameters for ball penetrometer. A possible sequence is given in Table 3, which would be completed within a depth range of 0.5 m or 10 ball diameters in 2 to 3 min. The proposed variable-rate penetration test may be particularly useful and important for the interpretation of penetration tests and predicting foundation behaviour in intermediate soils such as silts, as noted by Erbrich (2005).

Remoulded undrained shear strength

A number of design situations in offshore engineering require estimation of the remoulded undrained shear strength of the soil. A common application is for estimation of the shaft friction during installation of suction anchors. Design approaches usually equate this to the remoulded undrained

Table 3. Suggested sequence of penetration rates for evaluation of rate effects.

Step	Rate (mm/s)	Comment
0	20	Standard rate
1	60	Increased rate for 0.1 m or 2d
2	20	Standard rate for 0.1 m or 2d
3	6	Decreased rate for 0.1 m or 2d
4	2	Decreased rate for 0.1 m or 2d
5	6	Increased rate for 0.1 m or 2d
6	20	Revert to standard rate

Note: d, penetrometer diameter.

shear strength of the soil. Logically, the cone friction sleeve could provide a suitable estimate of the installation shaft friction, but there is considerable uncertainty in friction sleeve measurement and presently the data are not considered reliable (Lunne and Andersen 2007). Improved measurement of the cone sleeve friction remains a challenge for the future, particularly in very soft soils where the friction can be extremely small. Due to the effects of pore pressures at the end of the friction sleeve, improvement in results may be achieved by adding a pore pressure sensor at the upper end of the sleeve.

Conclusions and guidance on when to use the different tests

Based on the findings and experience obtained from a joint industry project, a number of recommendations on the design of in situ tools and testing procedures in soft offshore sediments have been suggested to improve the accuracy and reliability of the test results and the consistency in results obtained by different operators. Guidelines are also provided for the interpretation of intact and remoulded undrained shear strength from the penetration resistance measured by different penetrometers. The main focus has been on lightly overconsolidated clays, with strengths less than 100 kPa.

Preliminary studies have shown the potential of full-flow penetrometers, particularly if fitted with pore-water pressure sensors, for determining strain rate dependency of soil strength, soil stratigraphy, and consolidation parameters, by varying the penetration rate during a penetration test. Therefore, some suggestions on future developments for the in situ tools (especially penetrometers) and associated equipment were also recommended to maximize their potential in characterization of deepwater soft clays.

Recommendations on which of the in situ tools (cone, T-bar, ball penetrometer or vane) should be used for a site investigation will depend on the project requirements, the soil conditions that are likely to be encountered, and the geotechnical problem(s) to be solved. Table 4 summarizes a number of geotechnical problems relevant to deepwater field developments and the various soil parameters that can be interpreted from in situ testing (mainly the undrained shear strengths) and their reliability. Table 4 is meant to be used as a guide for when to use the different in situ tests. The T-bar and ball penetrometer are grouped in the same category because their measured resistances are very similar.

The undrained shear strength estimated from a piezocone penetration test is rated as lower reliability in the case of backfilled material as compared with that in the original seabed soil. This is because very low measured cone resistance and pore pressure are expected in this type of material. For the characterization of soft clay at very shallow depths, the T-bar (or ball penetrometer) and the vane should be capable of estimating the undrained shear strength with sufficient accuracy, if the tests are performed with extreme care. However, the T-bar (or ball) penetration test is much quicker than the vane shear test and gives a continuous profile of undrained shear strength. In addition, vane shear test results typically show more scatter due to the varying amounts of soil disturbance and consolidation, as a result of the vane insertion, before the vane shear test is conducted. As such, it is recommended that the T-bar (or ball penetrometer) is viewed as the primary tool, with the vane test as a supplementary test to increase reliability of the measured undrained shear strength. T-bar and ball penetration tests should also be performed in box core samples for assessing the shear strength profile in the upper 0.5 m of the seabed.

In natural deposits, where the stratigraphy and knowledge of material type is required, it is recommended that the piezocone be used as the primary investigation tool because there is wide experience in deducing the material type from the piezocone parameters. However, for the estimation of undrained shear strength, particularly in relatively soft material, the T-bar (or ball) penetrometer should be considered as a supplementary tool. This is because the T-bar (or ball) penetrometer is potentially more reliable than the piezocone (particularly when $q_{\text{T-bar}}$ and q_{ball} are correlated to s_{uave} and s_{uvane}), and the deduced undrained shear strength from the T-bar (or ball) penetration resistance appears to offer a good predictive basis for the capacity of foundation elements (e.g., Watson 1999).

At this stage there is insufficient experience to assess the relative merits of the T-bar or ball penetrometer. The T-bar penetrometer, by its nature, is more susceptible to bending moments being induced in the load cell. These may result in spurious changes in the load cell measurements because it is difficult to achieve complete independence of the load cell from the effects of bending. However, the T-bar penetrometer may be viewed as a model pipeline element, and thus provides direct information for pipeline and riser design.

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Table 4. Applicability–reliability of interpreted soil parameters.

				Applicability–reliability		
Geotechnical problem	Depth below seabed (m)	Comment	Soil parameters required[a,f]	Piezocone	T-bar, ball	Vane
Backfilled trenches: upheaval buckling	0–1	Extremely soft material may be encountered	Soil profile	1–2	3	—
			Classification	2	—	—
			Soil density	2–3	—	—
			Undrained shear strength	2–3	1–2	2–3
Pipeline–riser soil interaction	0–3	Very soft material may be encountered	Soil profile	1–2	3	—
			Classification	2	—	—
			Undrained shear strength	2	1–2	2–3
			Remoulded shear strength	5	1–2[b]	2–3[e]
Skirted foundations: penetration, bearing capacity	0–15/40	—	Soil profile	1–2	3	—
			Classification	2	—	—
			Undrained shear strength	2	1–2	2–3
			Remoulded shear strength	5	1–2[b]	2–3[e]
Seabed templates, penetration, stability, settlements	0–10	—	Soil profile	1–2	3	—
			Classification	2	—	—
			Undrained shear strength	2	1–2	2–3
			Remoulded shear strength	5	1–2[b]	2–3[e]
			Settlements	(3–4)[d]	(—)[d]	(—)[d]
Geohazards; slope stability	0–10/100[c]	Use of T-bar–ball and vane may be limited to 40 m depth	Soil profile	1–2	3	—
			Classification	2	—	—
			Undrained shear strength	2	1–2	2–3
			Remoulded shear strength	5	1–2[b]	2–3[e]

[a]Scale of relative applicability–reliability: 1, high; 5, very low; —, no applicability. The values indicate to some extent NGI and COFS's view on the potential tool to derive a certain parameter.
[b]Require cyclic T-bar (or ball) penetration tests.
[c]This study mainly covers the interpretation of parameters at depths up to say 30 m below seabed.
[d]Settlement parameters have not been covered in this study.
[e]Requires at least 10 quick rotations.
[f]Parameters for evaluating cyclic behaviour not included in the above table.

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Symbol list

A_p	projected area of the penetrometer in a plane normal to the shaft
A_s	cross-sectional area of the connecting shaft for T-bar and ball penetrometer
B_q	pore pressure ratio
d	penetrometer diameter
d_{vane}	vane diameter
e	vane blade thickness
F_r	normalised friction ratio
f_s	measured sleeve friction
G/s_u	rigidity index
G_0	small strain stiffness
I_r	rigidity index
N_{ball}	ball factor, q_{ball}/s_u
N_{kt}	cone resistance factor, q_{net}/s_u
$N_{kt,suave}$	cone factor relative to average or simple shear strength (= q_{net}/s_{uave} or q_{net}/s_{udss})
$N_{kt,suc}$	cone factor relative to triaxial compression shear strength (= q_{net}/s_{uc})
N_{rem}	remoulded resistance factor
$N_{rem,ball}$	remoulded ball factor, $q_{ball,rem}/s_{ur}$
$N_{rem,T\text{-}bar}$	remoulded T-bar factor, $q_{T\text{-}bar,rem}/s_{ur}$
$N_{T\text{-}bar}$	T-bar factor, $q_{T\text{-}bar}/s_u$
$N_{T\text{-}bar,rem,fc}$	T-bar factor relative to remoulded fall cone strength (= $q_{T\text{-}bar,rem}/s_{ur,fc}$)
$N_{T\text{-}bar,rem,UU}$	T-bar factor relative to remoulded UU strength (= $q_{T\text{-}bar,rem}/s_{ur,UU}$)
$N_{T\text{-}bar,rem,vane}$	T-bar factor relative to remoulded vane strength (= $q_{T\text{-}bar,rem}/s_{ur,vane}$)
$N_{T\text{-}bar,suave}$	T-bar factor relative to average or simple shear strength (= $q_{T\text{-}bar}/s_{uave}$ or $q_{T\text{-}bar}/s_{udss}$)
$N_{T\text{-}bar,suc}$	T-bar factor relative to triaxial compression strength (= $q_{T\text{-}bar}/s_{uc}$)
$N_{\Delta u}$	cone pore pressure factor, $(u_2\text{-}u_0)/s_u$
q	penetration resistance
q_{ball}	ball penetration resistance
$q_{ball,rem}$	remoulded ball penetration resistance
q_c	measured cone penetration resistance
q_m	measured penetration resistance
q_{net}	net cone penetration resistance
q_{ref}	penetration resistance measured at the reference penetration rate i.e. 20 mm/s
q_t	corrected cone penetration resistance
$q_{T\text{-}bar}$	T-bar penetration resistance
$q_{T\text{-}bar,rem}$	remoulded T-bar penetration resistance

s_u undrained shear strength
s_{uave} average of triaxial and simple shear undrained shear strength
s_{uc} triaxial compression undrained shear strength
s_{udss} simple shear undrained shear strength
s_{ue} triaxial extension undrained shear strength
s_{ur} remoulded undrained shear strength
$s_{ur,fc}$ remoulded fall cone undrained shear strength
$s_{ur,UU}$ remoulded unconsolidated undrained shear strength
$s_{ur,vane}$ remoulded vane shear strength
s_{uvane} vane shear strength
u measured pore pressure
u_0 hydrostatic water pressure
u_2 pore pressure measured at the shoulder of the cone
v penetration rate
v_0 penetration rate at which the undrained strain rate effect start to decay towards zero
v_{ref} reference penetration rate i.e. 20 mm/s
α net area ratio
α_s interface friction ratio
γ_{bulk} total unit weight of soil
Δ in situ normalized shear stress ($= (\sigma_{v0} - \sigma_{h0})/2s_u$)
μ rate coefficient
σ_{v0} in situ total overburden stress
σ_{h0} in situ total horizontal stress