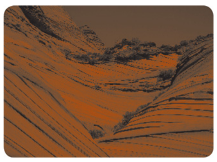

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Reference

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ABSTRACT

This paper presents the design and use of an interface direct shear testing device called the Cam-shear apparatus. This device permits the testing of very soft marine clays against pipeline interface material, and can be utilised to determine the interface friction value and the soil strength. The results from a series of interface and soil–soil tests using reconstituted, reconsolidated marine clays are presented. The Cam-shear device is shown to be a useful tool for pipeline design, particularly where very low normal stress levels are required, with stresses ranging from 1 to 4.5 kPa presented herein. The results demonstrate that the peak interface friction value is influenced by the preconsolidation pressure (overconsolidation ratio), the rate of shearing (governed by the drainage condition) and the interface roughness (rough or smooth). The rate at which peak strength reduces is strongly dependent on both the rate of shearing and the interface roughness.

Keywords

marine clay, sediment, laboratory tests, shear, interface, pipelines

Introduction

Characterisation of interface shear behaviour for deepwater hot oil pipelines requires site-specific testing of soil samples. These pipelines are often permitted to self-embed into the seabed sediments without prior trenching or burial, although recent observations highlight the potential for remoulding of surface sediments during the pipe laying process (White et al. 2012). During operation, hot oil passes through the initially cold pipe causing hundreds of thermal expansion and contraction cycles. Expansion in the axial direction may result in movements of several metres, causing lateral buckling of five to twenty pipe diameters (Bruton et al. 2007; White and Cheuk 2008) equivalent to many metres on each occasion. One design approach for these unburied pipelines is "controlled buckling," where designers attempt to limit lateral bucking to safe levels over the lifetime of the pipeline. Control of lateral movements is reliant on the short and long term resistance to pipeline relative movement provided by the soil in the axial direction in the adjacent anchor zones that feed pipe into the buckle. Resistance is quantified using the peak and residual shear strengths both of the soil, and the soil–-pipeline interface. However, it is the axial soil–pipeline coefficient of friction (μ) that is more difficult to quantify.

A recent study using a long-pipe setup to investigate axial pipe–soil interaction on sediments from West Africa (White et al. 2011) highlighted the importance of obtaining an improved understanding of the complicated interplay between speed of sliding, pipeline interface roughness, and soil stress state. To better understand the potential variability of μ subject to these factors, the use of a shearing device capable of testing very soft clays at different shearing speeds and against interfaces of various roughnesses, all at very low effective confining pressures, is required. This paper presents the results of a series of tests using an interface shear apparatus called the "Cam-shear," which is capable of these determinations. This paper describes its use with reconstituted clays, including disturbed samples from offshore west Africa, which were taken through an overconsolidation cycle in the laboratory to achieve soil strengths representative of the ocean bed. The use of remoulded clay permits improved repeatability and limits the difficulties in interpreting complex data obtained when testing highly structured natural sediments.

EXISTING SHEARING DEVICES

Only a limited number of commonly-available laboratory devices are capable of assessing the soil-structure interface of offshore pipelines on soft marine clays. These devices include the ring shear and tilt table apparatus. A number of studies have considered the use of these devices, particularly for analysis of residual strengths, as summarised in **Table 1**. Early investigations focused on the internal strength of reconstituted soils rather than interfaces, as outlined below.

Lupini et al. (1981) considered the drained residual strength of clay and clay–sand mixtures during testing of samples with a ring-shear device and identified the presence of three main shearing behaviours: turbulent, transitional, and sliding. These phases were found to be dependent on particle shape and angularity, which controls the inter-particle friction.

Skempton (1985) considered the residual strength of clay–sand mixtures present in landslides, and compared their behaviour to tests reported by Lupini et al. (1981). Lemos and Vaughan (2000) undertook interface shear tests with ring shear and conventional direct-shear boxes. Interfaces of varying roughness were used to investigate the influence of interface roughness on interface shear strength. It was concluded that samples with a high clay content produced residual interface strengths that were independent of the interface roughness, and comparable with soil–soil residual strengths.

Stark and Contreras (1996) and Stark and Eid (1993,1994) describe the use of a modified Bromhead ring shear apparatus to complete the undrained and drained shear strength testing of a large number of clays and shales. Both peak (Stark and Contreras, 1996) and residual (Stark and Eid, 1994) strengths were analysed. Applied normal stresses were relatively large (between 60 and 850 kPa) and related to the application of terrestrial landslides and dam failures.

Najjar et al. (2007) describes the design and use of a tilt-table test device to measure the drained residual shear strength of pipeline-clay interfaces subject to low (1.7 to 5.8 kPa) normal stresses. The clay is reconstituted, smeared over the model interface, loaded by a deadweight, and is then permitted to

TABLE 1 Summary of existing interface shearing devices.

Shearing device	Sample condition	Interface	Stress level (kPa)	Comments / Limitations	Reference
Ring shear	Remoulded	Rough and smooth	20 to 60	- Only remoulded samples can be tested due to nature of sample preparation - Apparatus internal friction limits the minimum achievable stress	Colliat et al. (2011)
	Remoulded	Rough (coarse sand blasting) and smooth (glass)	50 to 400		Lemos and Vaughan (2000)
	Remoulded	Rough and smooth	10 to 50		Fugro (2010)
Tilt table	Remoulded	Smeared	5 to 30	- Only remoulded samples can be tested due to nature of sample preparation - The stress distribution on the interface varies depending on the angle of the table - Shearing displacement is limited to size of table	Pederson et al. (2003)
Low stress shearbox (UWA)	Remoulded	Rough and smooth	> 2.5 kPa	- Only remoulded samples have been tested - Shearing displacement is limited to size of interface material	White et al. (2012), Hill et al. (2012)

drain, coming into a normally consolidated condition before being tilted so that the deadweight creates shear stress. Due to the limitations of the method utilised to mobilise the failure mechanism, each shearing episode was only permitted to travel 13 mm, which was repeated to achieve a cumulative total shearing distance of about 100 mm. A range of interface materials were used with estimated roughnesses of 10 and 50 μm.

Colliat et al. (2011) present a large data-set of ring-shear test results relating to the interface strength of soft clays against rusty steel ("rough") and coated ("smooth") interfaces. These tests were undertaken at normal stress levels applicable to the design of suction caissons; therefore at levels higher than those experienced by self-embedded pipelines.

As shown in **Table 1**, there are a number of limitations inherent to these devices. Key limitations include the inability to test natural core samples, high internal friction values, and difficulties in maintaining constant normal stresses at the sheared interface. Neither the ring-shear nor the tilt table device can therefore fully satisfy the requirements of an interface shearing device for low effective stress pipeline testing, which is required to consider the following items:

- testing of both natural and remoulded samples,
- testing samples at very low stress levels consistent with applied design stresses,
- unrestricted shearing on soil–coating interfaces,
- measurement of both peak and "residual" strengths,
- shearing at a broad range of rates, and
- shearing on a range of interface roughnesses.

The lateral resistance to pipeline movement involves shearing internally on a sub-horizontal surface beneath a soil berm pushed upwards as the pipe translates (White and Cheuk 2008). Since soft sediments are expected to be anisotropic, an ideal test would shear soil internally on a horizontal plane. Axial resistance additionally requires soil-interface shearing.

More recently, direct shear testing devices have been modified to create a machine that is able to cover some of the above points (e.g. White et al. 2012). Limitations, however, still exist, such as the ability to achieve "residual" strengths (see **Table 1**). White et al. (2012) presents data using a modified shear box permitting testing at low stress levels, albeit through a shearing distance of only 10 mm. Reversal of shearing direction is also possible with the modified shear box; although, it is clear this testing procedure results in a significant variation in vertical displacement during each cycle of shearing.

The Cam-shear device described in this paper is another machine that attempts to deliver all the facilities listed above. The strength data that emerges can be described in terms of total stresses, with internal soil strength τ^* and interface strength τ. Parameters with respect to effective stresses are difficult to validate since the excess pore pressure at an interface can only be measured through the filter ceramic of an embedded transducer, and this may not be representative of the value induced by the interface material under test. In the work that follows, this philosophical difficulty is avoided by quoting the ratio of shear strength on the plane of interest to the pre-existing normal effective stress, achieved after a substantial period of pre-consolidation. In that way, apparent friction coefficients μ^* for internal soil shearing, and μ for soil-interface shearing can be quoted. These are analogous to consolidated-undrained soil strength parameters, although in this case the degree of drainage during shearing will depend on the test duration.

Cam-Shear Device

TESTING PROCEDURE

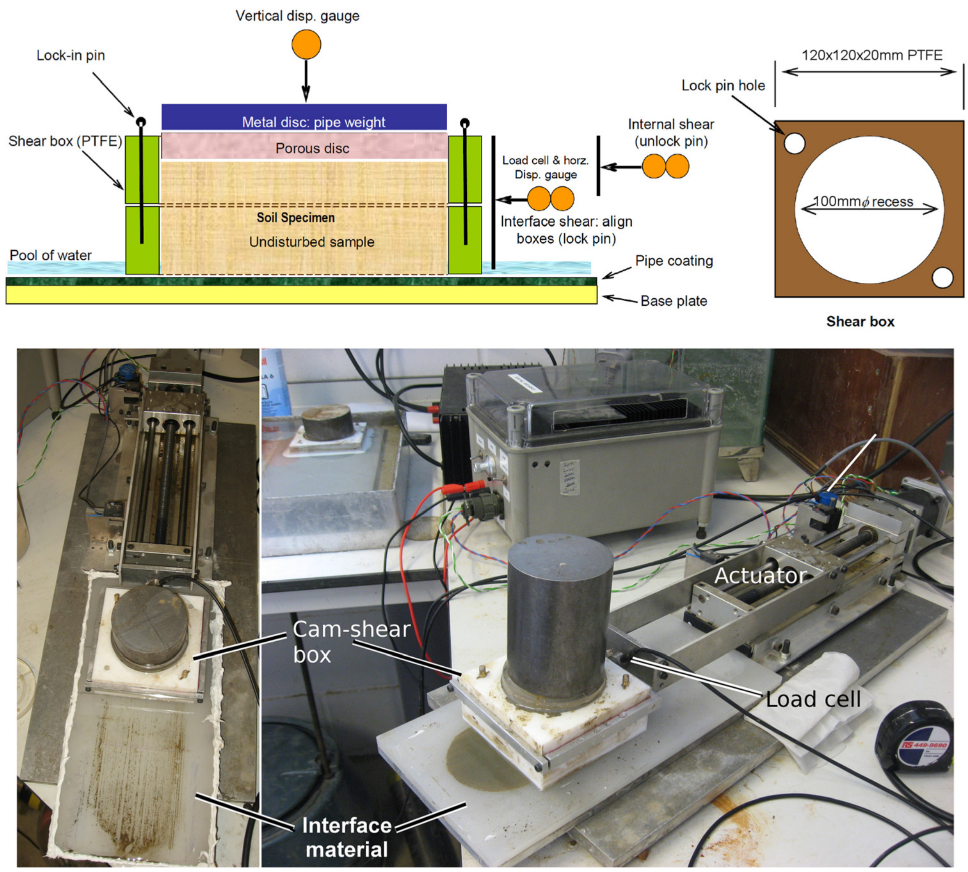
The Cam-shear device is a direct shear apparatus with negligible friction in the system, which allows test conditions to approximate the stress levels experienced by soils under pipelines. It was originally developed for the shearing of granulated bones at low stress levels by Dunlop et al. (2000). It was then modified to allow the simulation of axial pipe–soil interaction behaviour through the dragging of a clay sample exhibiting low undrained shear strength over a flat sheet of pipe coating material such as polypropylene or polyethylene (Haustermans 2002).

The key advantages of the Cam-shear device over existing interface testing devices are its simplicity and ease of application to axial pipeline behaviour, including shearing rates and pipeline stresses. The Cam-shear device comprises a split-plane shear box with an open base and an actuator, as shown in a schematic diagram and photographs of the whole device (**Fig. 1**). The shear box consists of two 100 by 100 mm blocks of polytetrafluoroethylene (PTFE), each 20 mm thick, with a central circular bore of 75 mm diameter to house the soil sample. Additional shear boxes can be specifically made for the testing of natural core samples that have a different diameter than those considered in this paper.

PTFE is used to minimise the inherent friction in the system. The weight of the sliding portion is only 0.583 kg (equivalent to a normal stress on the shear plane of 0.51 kPa) and the coefficient of friction between PTFE and smooth polypropylene is approximately 0.1. The sliding friction of an empty box is therefore equivalent to an inferred shear stress of approximately 0.1 kPa over the test area. The interface shear strength of the soil is measured by dragging the shear box, containing the clay sample, over a flat sheet of interface material. The Cam-shear arrangement also allows the consolidation of the sample against the interface material and therefore represents a simplified arrangement representing a pipeline coating interface moving above a sample of clay. If the coefficient of side friction between PTFE and clay is also taken as 0.1, the average vertical stress on the interface would be reduced by 10 %. It follows that the magnitude of the soil-interface friction coefficient would be

FIG. 1

Schematic diagram of the Cam-shear shear box showing the main features and photograph of Cam-shear test setup.



underestimated by 0.01. The vertical effective stress at the interface is created by the self-weight of the tested sample and the applied vertical load. At the soil–soil shear plane, the vertical effective stress is slightly smaller, corresponding to half the self-weight of the tested sample and the applied pipeline load. These small adjustments can easily be allowed for.

The internal shear strength of the clay sample τ^* can also be measured by removing the locking pins and splitting the shear box at its mid-depth (see **Fig. 1**). The top of the box is moved relative to the bottom to impose shear failure in the soil. Normal stress corresponding to the pipe weight is either applied by steel discs bearing on top of the soil sample or by a hanging loading frame. Drainage layers are inserted between the loading platen and the clay (at the top of the sample) to allow water to escape during consolidation and shearing.

A linear push–pull device (actuator) is used to apply relative displacement between the stationary interface material and the moving clay sample in the shear box, and also to shear the clay sample after splitting the box. A 30 V DC motor coupled with a tachometer drives the actuator through a lead screw, as seen in **Fig. 1**. Both load-controlled and speed-controlled testing can be undertaken; although in the series of tests undertaken herein, only speed-controlled testing is used. The device is used in this work to run tests at speeds from 5 to 0.005 mm/s. A total stroke length of 190 mm can be achieved.

The interface shear strength and the internal soil shear strength are measured by a 50 N load cell (Novatech Z250, push–pull type) coupling the shear box and the actuator. The horizontal displacement and the associated shearing rate are measured using a draw wire potentiometer mounted on the actuator. A linear variable displacement transducer (LVDT) monitors the vertical displacement in the soil specimen during sample consolidation. This LVDT was removed prior to conducting shear tests due to the very large magnitude of lateral displacement.

SAMPLE PREPARATION

The Cam-shear apparatus is designed to accept natural samples extruded from core tubes. However, the tests reported below utilised reconstituted, reconsolidated natural marine sediments from deep water locations off the west coast of Africa. Each bulk sample of test material was reconstituted at a moisture content of 1.3 to 1.5 times the liquid limit. The samples were homogenized by thorough mixing for about 30 min, and were kept under a gauge partial vacuum pressure of 25 kPa for about 20 min to remove entrained air bubbles.

According to site investigation data obtained at the same site as the grab sample used in this test programme, the used sediment has a clay fraction (particles $< 2\,\mu$m) of 60 %. The plastic and liquid limits are, respectively, 60 and 160 % (from the fall cone test). The in situ bulk unit weight of the sediment is 13.1 kN/m^3 at the natural moisture content of about 180 %. The soil sample was reconstituted with saline water (salinity 35 mg/l). The coefficient of consolidation of the reconstituted clay sample was approximately 0.04 mm^2/s from one-dimensional swelling tests corresponding to unloading from 6 kPa to 0.5 kPa.

ONE-DIMENSIONAL CONSOLIDATION

The samples were over-consolidated to match undrained shear strength profiles obtained during in situ T-bar tests at the offshore sites. A maximum consolidation pressure of either 8 or 14 kPa was chosen for these clays.

To enable one-dimensional consolidation, the shear box was placed on the interface material (polypropylene), and the gap around the outside base of the box was sealed with a silicon sealant (marketed under the trade name Geocel Aquaria) so as to be water-tight at the interface. The sealant could be peeled off without affecting the interface material. A layer of filter paper, a porous disc, and geotextile was placed between the loading platen and the clay sample to permit single upward drainage. The consolidation pressure was applied by weights suspended on a hanger. Consolidation periods were typically between 1.5 and 2 days. Settlement was measured using an LVDT resting on the upper surface of the PTFE drainage disc and logged using a mobile data-logger using the commercial software, Dasylab.

SWELLING AND RECOMPRESSION

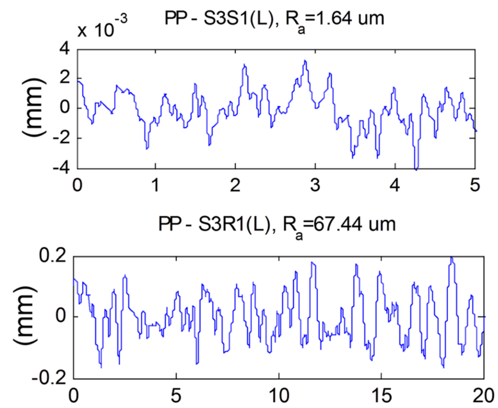
After normal consolidation was complete (as indicated by measurements of the vertical settlement), the sample was permitted to swell back under a normal vertical stress of 0.5 kPa for at least one third of the normal consolidation time. This swelling was followed by recompression under a vertical stress equivalent to the simulated pipe contact pressure for approximately the same period again—one third of normal consolidation time.

After consolidation, the silicon seal between the shear box and the interface material was carefully peeled off. The pipeline interface surface was kept submerged in a water tray during shearing to prevent the sample from drying out and to produce a more representative testing environment.

PIPE COATING INTERFACE MATERIAL

Pipe coating samples of polypropylene were supplied by industry partners. The surface roughness of the coatings was characterised by a profilometer (Taylor Hobson Precision, Form Talysurf) that plots surface undulations at a suitably-magnified scale by traversing a sensitive probe on the coating surface along the longitudinal pipe axis (see **Fig. 2**). The average surface roughness of the coating R_a was found to be $R_a = 1.64\,\mu$m for the smoother interface (PP-S3S1(L))and $R_a = 67.44\,\mu$m for the rougher interface (PP-S3R1(L)). Based on particle size distributions (PSD) of reconstituted samples taken from the area of interest, D_{50} is found to range from 3 to 10 μm (Thomas et al. 2005) using different methods to reduce the sediment to its constituent parts. Following Oliphant and Maconochie (2007), relative roughness R can be estimated by

FIG. 2 Roughness profiles for interfaces. PP-S3S1(L): smoother interface with $R_a = 1.64\,\mu$m. PP-S3R1(L): Rougher interface with $R_a = 67.44\,\mu$m.



$$R = \frac{R_a}{D_{50}} \tag{1}$$

Based on Eq 1, and assuming $D_{50} = 10\,\mu$m, the rougher interface used in this series of tests has a relative roughness of 6.74 whereas the smoother interface has $R = 0.16$.

Testing Procedure

Each Cam-shear test involved three stages:

(a) Sample preparation carried out over a flat sheet of pipe coating material—one dimensional consolidation from a clay slurry to the required normal stress corresponding to a particular pipe contact stress, and with a prior overconsolidation episode if required to ensure that the undrained strength of the clay is appropriate,
(b) An interface shear test, carried out at an appropriate shear speed,
(c) A soil shear test (on a split plane 20 mm above the interface).

Some tests involved cycles of shearing during stage (b), sometimes at different speeds, and with different pauses between successive shear cycles. The measured interface and soil strengths were corrected for the very small superfluous friction in the shearing device. Soil strength measured from the split tests was corrected for the reduction in the area of soil–soil contact while shearing.

During this test programme, clay samples were overconsolidated either to about 8 kPa or to about 14 kPa in order to provide higher undrained shear strengths corresponding to those typically found immediately below mudline in field tests offshore west Africa.

The factors varied during the Cam-shear testing were:

(1) Preconsolidation pressure σ_c: either 14 or 8 kPa,
(2) normal stress σ during shearing: between 1and 4.5 kPa,
(3) surface roughness R_a of the coating material: = 1.64 μm or 67.44 μm,
(4) surface condition: dry, smeared with clay, smeared with clay and submerged,
(5) speed V: 0.5, 0.05, or 0.001 mm/s,
(6) pause period prior to shear reversal (various lengths).

Influence of Shearing Speed

It is recognised that the rate of shearing can strongly influence test data, which may elicit a fully drained, partially-drained, or undrained soil response. Shearing of soils is associated with the simultaneous build-up of excess pore pressure (positive or negative), and dissipation as the pore water tries to reach pore pressure equilibrium. This is a transient phenomenon, and is primarily governed by the drainage and compressibility characteristics of the soil, the length of drainage path to the boundaries, and the duration of loading. In a direct shear test running at constant speed, the duration of the test is simply the maximum displacement divided by the shear speed. This has led authors to associate the degree of drainage in a shear test with the velocity of sliding, although it is obviously the case that a given soil sample tested in a Cam-shear box at 0.01 mm/s over a displacement of 100 mm (and therefore with a duration of 2.78 h) should be much closer to displaying the drained residual strength of the soil than would a conventional shear box tested over 10 mm at the same speed (for a duration of 17 min).

Furthermore, it is not even clear whether excess pore pressures develop solely as a function of the applied stresses, in which case they should reach their maximum values relatively early in the shear process, or whether pore pressures continue to develop with shear displacement, due to disturbance or damage to the soil microstructure, even though the applied stresses remain almost constant. It is recognised, for example, that the cyclic application of a constant amplitude of shear stress causes granular materials to generate excess pore pressures cycle by cycle, compensating for the tendency for cyclic compaction (Martin et al. 1975). It might be argued that the disturbance caused by a succession of projections from a rough sliding interface might have a similar effect on clays.

Moreover, soil mechanics literature also accepts that strain rate influences stiffness and strength. Kulhawy and Mayne (1990) show data from 26 clays that register an average increase in undrained strength of 10 % per factor 10 on time. This phenomenon has been modelled (Lin and Wang 1998; Kuhn and Mitchell 1993) as a thermally activated rate process, the consequence of suppressed creep irrespective of excess pore pressures.

Without conducting additional investigations, it is impossible to know which of these mechanisms may be responsible for some particular macroscopic behaviour observed in direct shear. All that can be required of this test equipment is that it simulates field conditions and leaves open the possibility of deeper inquiries into the origins of any observed behaviour. Accordingly, test results should at least include information regarding test speed and duration in relation to an estimate of the time required for transient flow. Evidence of an increase in strength with rate of testing even for samples that should each be undrained would suggest that creep-related rate effects were also in evidence.

TEST DURATION IN RELATION TO TRANSIENT FLOW

Gibson and Henkel (1954) have derived an expression for normally consolidated clays using Biot (1941), relating the time t_m to mobilise peak strength in a direct shear box test and the corresponding degree of dissipation of excess pore pressure U_m during that mobilisation

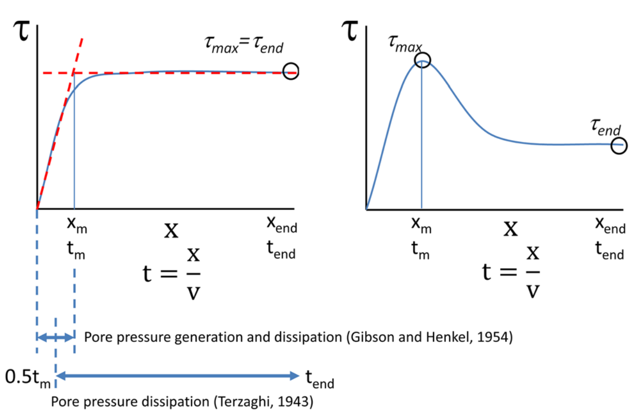
$$t_m = \frac{H^2}{2(1 - U_m)c_v} \tag{2}$$

where:

c_v = the coefficient of one-dimensional consolidation, and
H = the drainage distance.

Their theory considers that pore pressures are simultaneously created and dissipated during time, t_m, and then dissipate away for $t > t_m$. **Figure 3** illustrates this in relation to typical data of shear stress versus displacement, marking peak strength $\tau_{\max}$ mobilised at time t_m with the final strength τ_{end} recorded at the end of the test at time t_{end}. The selection of t_m in cases where no peak is observed requires some judgment concerning the likely point after which further excess pore generation may be negligible. This judgment goes beyond the analysis of Gibson and Henkel (1954) because soil being sheared in a general state

FIG. 3 Nomenclature for characteristic values of strength, displacement and time.



of overconsolidation may develop either positive or negative excess pore pressures, so that continuing drainage may lead either to an increased or decreased shear strength.

Applying Eq 2 to the state of drainage at peak strength mobilisation, we may say that a "drained" peak strength would require $U_m \geq 0.9$ so that $t_m \geq 5H^2/c_v$, whereas an "undrained" peak strength would be recorded if $U_m = 0$ so that $t_m \leq 0.5H^2/c_v$.

In the following period, $t > t_m$, it will be considered sufficient to apply Terzaghi's standard consolidation theory with an assumed dissipation time $(t - 0.5t_m)$. We may then say that a "drained" strength at the end of the test would require $U_{end} \geq 0.9$ so that $(t_{end} - 0.5t_m) \geq 0.8H^2/c_v$, whereas an "undrained" strength would ultimately be recorded if $U_{end} \approx 0$ so that $(t_{end} - 0.5t_m) \leq 0.08H^2/c_v$.

There are some additional considerations required in the use of the Gibson and Henkel solution to interpret the Cam-shear data to be reported here. Firstly, it is assumed that excess pore pressures are created throughout the sample thickness, and not just in the vicinity of any shear rupture band that may form. More localised pre pressures would lead to faster consolidation. Secondly, the Gibson and Henkel solution is based on the assumption that the split plane is symmetric to the drainage boundaries. In the Cam-shear setup, the split plane was asymmetric to drainage boundaries with drainage only permitted at the top. This was taken into account by using the shortest drainage length of 10 mm for the parameter H in Eq 2 applicable to soil–soil tests. The direct shear box testing against an interface is similarly assumed to generate uniform pore pressures that dissipate upwards, so that drainage distance H in Eq 2 is taken to be equal to 30 mm in soil-interface tests, which is the approximate thickness of clay in the box. Finally, since the samples were over-consolidated, the coefficient of consolidation c_v for soil swelling was used rather than for virgin compression.

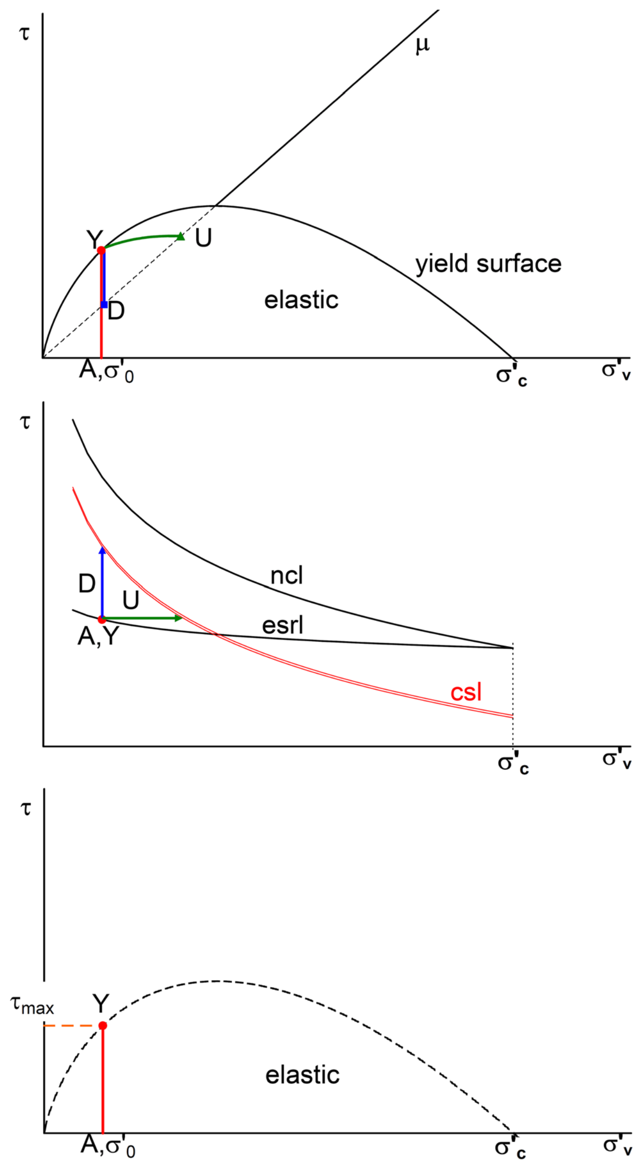
Depending on the behaviour observed during shearing, judgments can be made as to whether tests were completed in a drained, partially-drained, or undrained manner, as indicated in the following section. The interface shear strength τ and soil strength τ^*, both at the peak (maximum) value and at the end of a test (after approximately 60 to 120 mm shear displacement of interface sliding, and about 12 mm of split-plane shearing), have been identified as shown schematically in **Fig. 3**, where the shear strength is given as τ followed by a subscript defining the phase of the test. Some tests showed a monotonic increase in resistance to the end of the test whereas others showed a softening response after the peak.

Interpretation of Direct Shear Test Results

The lack of secure knowledge regarding the degree of drainage of excess pore pressures in direct shear tests has always been regarded as a serious drawback. However, some attempt must be made to overcome this problem in the case of Direct Simple Shear (DSS) tests, and interface shear data where direct shear is the most economical way of conducting tests. A method of interpretation will be recommended below based on the precepts of Critical State Soil Mechanics (CSSM), Schofield and Wroth (1968).

It is well-known that overconsolidated clays have a curved strength envelope on a Mohr–Coulomb diagram. A CSSM view of these peak strengths is that they arise from interlocking of the basic soil grains, which have to dilate before they are free to shear (Schofield 2006). Expressed in terms of direct shear stresses, the normalised Cam Clay yield surface is given as Eq 3.

FIG 4 Schematic diagram showing the definition of peak shear strength values obtained from the Cam-shear apparatus using the Cam clay model.



$$\frac{\tau^*_{max}}{\sigma'_0} = \mu^*_{crit} ln\left(\frac{\sigma'_c}{\sigma'_0}\right) = \mu^* \quad (3)$$

where:

μ^* = the variable friction coefficient, and

μ^*_{crit} = the constant critical state friction coefficient.

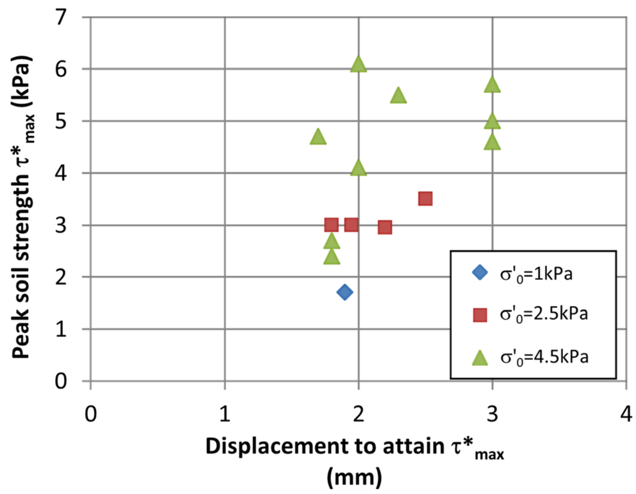
The stress paths taken by the soil to reach its ultimate state may not be known, if pore pressures are not measured. Nevertheless, if the initial vertical effective stress σ'_0 following vertical consolidation is known, and if it is accepted that within the yield surface the behaviour is elastic, then the yield strength, τ_y, can be determined (see **Fig. 4**) path A to Y, regardless of whether the test is undertaken in a drained or undrained manner. Once yielded, the soil may follow the idealised paths shown by Y–D for drained shearing or Y–U for undrained shearing (at constant volume). In any case, the predicted maximum shear strength, τ_{max}, developed in a DSS test starting on the "dry" side of the critical state line (i.e. $\sigma'_0 < \sigma'_c/2.7$) will be the yield stress, τ_y, or perhaps slightly higher in an undrained test. There is, therefore, some theoretical underpinning for the presentation of soil strength data in the form τ^*_{max} versus $\sigma'_0 \ll \sigma'_c$.

Soil–Soil Test Results

DETERMINATION OF DRAINAGE CONDITION FOR SOIL–SOIL TESTS

A typical shear strength-displacement plot for a test undertaken at 0.05 mm/s is shown in **Fig. 5**; **Fig. 6** shows the shearing distance required to attain peak strength for tests completed at 0.05 mm/s. **Table 2** summarises the nature of soil response predicted at peak for the shearing rates utilised in this series of tests. The maximum strength of the soil is generally mobilised within 2.5 mm of shearing displacement. Therefore, at the peak shear strength, the test completed at 0.001 mm/s exhibits partially-drained conditions. Both faster tests (0.05 and 0.5 mm/s) will experience undrained conditions. At the end of the test corresponding to a shear displacement of 12 mm, the two faster shear speeds exhibit partially-drained conditions whereas the slow test at 0.001 mm/s now corresponds to a drained strength.

FIG. 6 Summary of shear displacement to attain soil peak strength.



INFLUENCE OF OCR AND SHEAR RATE

Figure 7 presents the variation in soil shear strength for different shear speeds at two preconsolidation pressures σ'_c (8 and 14 kPa). It is observed that for each σ'_c, an increase in shear speed results in an increase in measured shear strength. Although tests undertaken at 0.5 and 0.05 mm/s were determined in **Table 2** to represent undrained shearing, there is a distinct difference in shear strength. This could be attributed to a rate effect (suppressed creep) increasing the strength of the "fast" test. The curves shown in this figure correspond to Cam–clay yield surfaces fitted to the data corresponding to the arithmetic average of critical state coefficient of friction μ^*_{crit} found by applying Eq 3 to each data point.

There is clear evidence of preconsolidation pressure influencing the drained strength. In other words, it might be

FIG. 5
Example plot showing soil strength plotted against shearing displacement.

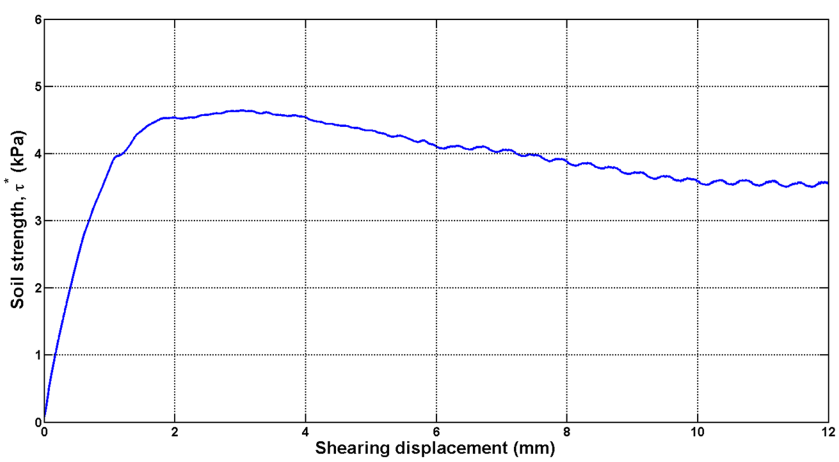


TABLE 2 Drainage conditions in typical soil–soil shear tests.

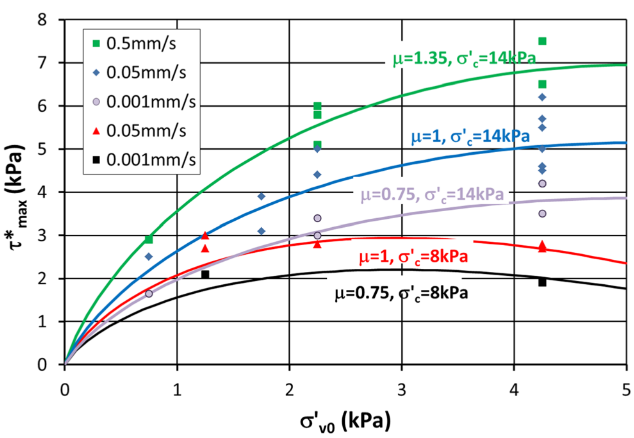
Consol. coeff. c_v mm^2/s	Drainage length H mm	Displace. to τ^*_{max} X_m mm	Displace. at τ^*_{end} X_{end} mm	Rate of shearing V mm/s	U_m $1-\frac{0.5H^2}{c_v t_m}$	Predicted peak soil response	U_{end}	Predicted end soil response
0.04	10	2.5	12	"slow" 0.001	0.5	Partially drained	1	Drained
				"medium" 0.050	0	Undrained	0.36	Partially drained
				"fast" 0.500	0	Undrained	0.12	Nearly undrained

reasonable to suppose that a greater undrained strength at any given depth in situ may indicate a higher preconsolidation pressure and a correspondingly higher drained strength (which could equally be expressed as a higher friction factor). Recalling that friction coefficient is the gradient of a line drawn to any given data point from the origin, it is also clear that for clay of a given preconsolidation pressure, the peak friction factor reduces with increasing initial effective stress normal to the shear plane.

The data of **Fig. 7** is replotted in **Fig. 8** on axes of τ^*_{max} (maximum shear strength) and σ'_0, both normalised by σ'_c. The significance of shear speed is now easier to assess. It can be seen in **Fig. 8** that the chosen yield surface, with $\mu^* = 1.0$, approximately satisfies the data of reconstituted clay tested at a shear speed of 0.05 mm/s. Faster tests provided higher strengths, and slower tests gave lower strengths, as might be expected for clays showing rate effects.

Plots of friction coefficient versus the logarithm of overconsolidation ratio (OCR $= \sigma'_{v,max}/\sigma'_{v,o}$) provide a succinct normalisation. **Figure 9** shows that the peak soil–soil shear strength expressed as a friction coefficient μ^*_{max} varies linearly with log-OCR, and confirms that the data of tests preconsolidated to different stresses do merge in accordance with Eq 3. It is also observed, however, that the gradient of these lines is a function of shear speed; in contrast with the expectation of Eq 3 that the gradient would simply be the critical state friction coefficient,

FIG. 7 Variation in shear strength when sheared at different rates for $\sigma'_c = 8$ and 14 kPa.



regarded by Schofield and Wroth (1968) as a constant. It should be noted that no data was acquired for the range $1 < \text{OCR} < 2$.

There are strong grounds for accepting that the normalised undrained strength $s_{u,max}/\sigma'_{v,o} \equiv \tau_{max}/\sigma'_{v,o} \equiv \mu^*_{max}$ following the notation used in this paper, can be taken as 0.5 for these West African clays (plasticity index PI ≈ 100 %) when normally consolidated, i.e., for OCR = 1, following the empirical expression given as Eq 4 Skempton (1954).

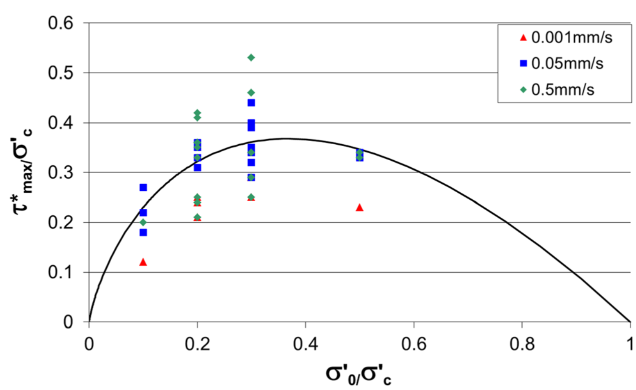
$$\frac{s_{u\,max}}{\sigma'_{v,nc}} = 0.11 + 0.37PI \tag{4}$$

A cut-off has accordingly been placed at this ordinate in **Fig. 9**. If the initial effective stress in the clay at the pipe invert was less than 1 kPa, and if a preconsolidation of at least 14 kPa is required to explain the in situ undrained strength, it should follow that the OCR of this clay would exceed 14. The origin of this apparent overconsolidation in situ was discussed by Kuo and Bolton (2013).

Interface Tests

Figure 10 presents results of reversing shear tests at a speed of 0.5 mm/s conducted both on rough and smooth interfaces. These plots show that for both interface types, the strength at the end of pass 1 (see **Fig. 10**) has dropped significantly below the peak strength. In the case of the rough interface, after a shearing distance of 100 mm, the strength had fallen to close to

FIG. 8 Cam-shear data of remoulded, reconstituted West African clay plotted on normalised axes of shear strength and vertical effective stress.



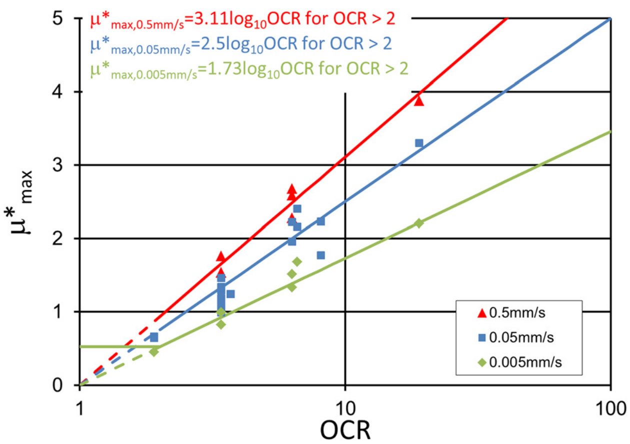


FIG. 9 Peak apparent internal friction coefficient μ^*_{max} versus initial plotted on semi-log axes.

zero. Shearing in the reverse direction (pass 2) and then again in the forward direction (pass 3) results in the strength recovering and levelling at about 2 kPa. The smooth interface generates an end of test strength of between 1 and 2 kPa. Although end-of-test interface strengths appear to be similar, the rate of strength decay is observed to be significantly different. The smooth interface rapidly falls to almost 50 % of peak strength within 5 mm of displacement whereas the rough interface requires approximately 20 mm for the peak strength to fall by the same amount. This indicates that the micromechanical processes of shearing are different for the two interfaces. **Figure 11** presents a summary of the shearing distance required to attain 50 % of the peak strength, plotted for various test conditions. This shows that an interface shearing device must be able to shear at least 30 mm in one direction if it is to capture the fall from peak strength.

Figure 12 shows a marked difference in interface resistance behaviour between tests completed at 0.001 and 0.05 mm/s. Both testes were undertaken on samples with a normal stress of 4.5 kPa and an initial OCR of 3. The fast test exhibits a sharp peak in resistance, which reduces dramatically to a constant residual resistance. The slower test exhibits a smaller peak with a small drop in resistance, and remains relatively constant with shear displacement. This difference in behaviour is likely to be caused by differing drainage conditions during shearing, as discussed below.

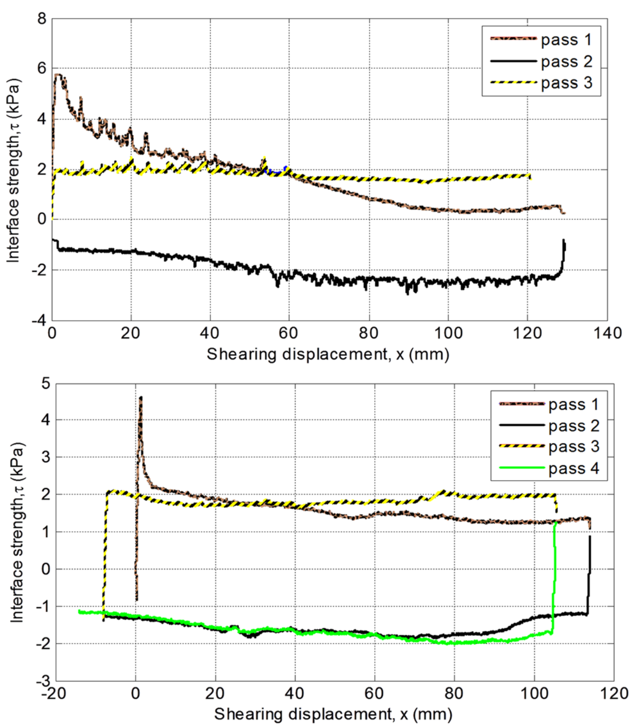


FIG. 10 Plots of "fast" shear test (0.5 mm/s) showing interface strength plotted against shearing displacement. Upper test data: rough interface; lower test data: smooth interface.

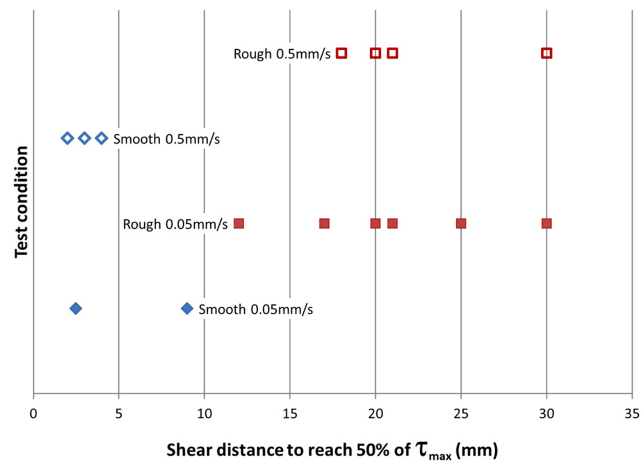


FIG. 11 Influence of interface roughness on shearing distance required to reach 50 % of τ_{max}.

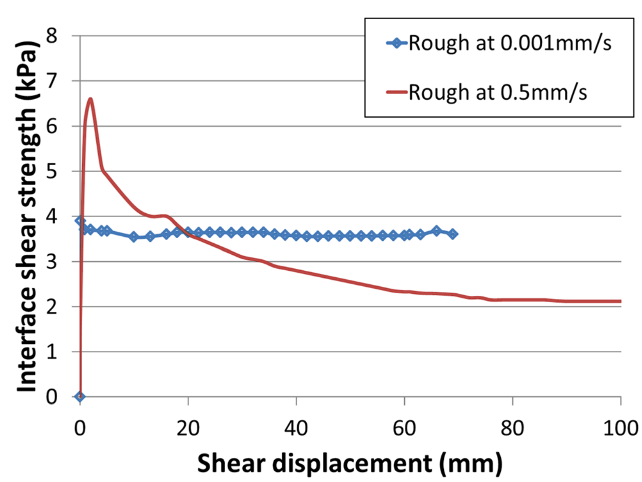


FIG. 12 Comparison of rough interface strength profiles when shearing at slow (0.001 mm/s) and fast (0.5 mm/s) speeds.

TABLE 3 Drainage conditions in typical soil-interface shear tests.

Consol. coeff. c_v mm²/s	Drainage length H mm	Displace. at τ_{end} X_{end} mm	Rate of shearing V mm/s	U_{end}	Predicted end soil response
0.04	30	100	"slow" 0.001	1	Drained
			"medium" 0.050	0.36	Partially drained
			"fast" 0.500	0.12	Nearly undrained

DRAINAGE CONDITION OF INTERFACE TESTS

Table 3 summarises the drainage conditions expected at the end of interface tests, corresponding to a shear displacement of approximately 100 mm. It can be seen that drained conditions are predicted at the slow shear speed of 0.001 mm/s, whereas partially drained and nearly undrained conditions should remain for the medium and fast tests, respectively. Thus, the test curves shown in **Fig. 12** correspond to drained (0.001 mm/s) and nearly undrained (0.5 mm/s) tests.

PEAK INTERFACE SHEAR STRENGTH

Plots of peak interface friction μ_{max} are given in **Figs. 13** and **14** for rough and smooth pipe coatings, respectively; trend lines are included. Shearing against a rough interface clearly shows μ_{max} progressively reducing with reduction in shearing rate; comparable to that observed for soil–soil shear tests. The tests completed on the smooth interface again follow this trend, but less conclusively. Although the degree of drainage will increase as the shear speed reduces from 0.5 through to 0.005 mm/s, it has been proposed in **Fig. 4** that this may not strongly influence the peak strength. An interpretation emphasising shear rate effects may be preferred. Just as with the soil–soil tests, the data for interfaces suggests that if Eq 3 is invoked, then μ'_{crit} should increase with shear rate.

END OF PASS INTERFACE SHEAR STRENGTH

Figure 15 presents the interface shear strength at the end of the first pass plotted against the peak interface shear strength for both rough and smooth interfaces prepared in different ways. When a pipeline has consolidated its bed, its subsequent shear displacements are such that the interface always slides over consolidated clay. The inevitable consequence of using direct shear tests is that, following consolidation, the shear displacement begins to overlap the clay onto clean interface material. This discrepancy may partly be accounted for by smearing the interface ahead of the shear box with clay, and if drying is to be prevented, by submerging the smeared surface. **Figure 15** presents a variety of such data, but no clear trends emerge regarding surface preparation.

FIG. 13 Peak apparent friction coefficient μ_{max} on a rough interface, versus initial OCR plotted on semi-log axes.

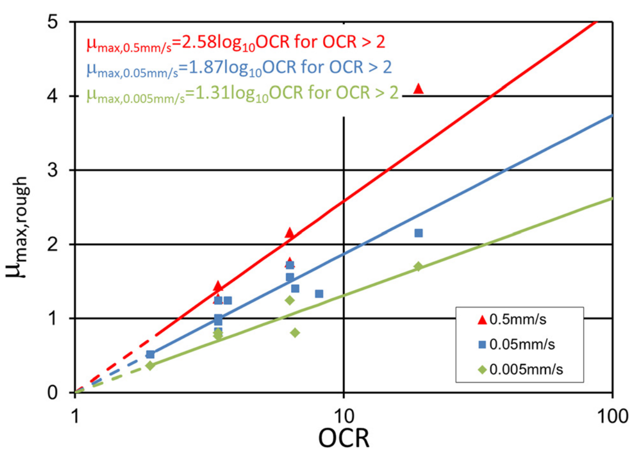


FIG. 14 Peak apparent friction coefficient μ_{max} sliding on a smooth interface, versus initial OCR plotted on semi-log axes.

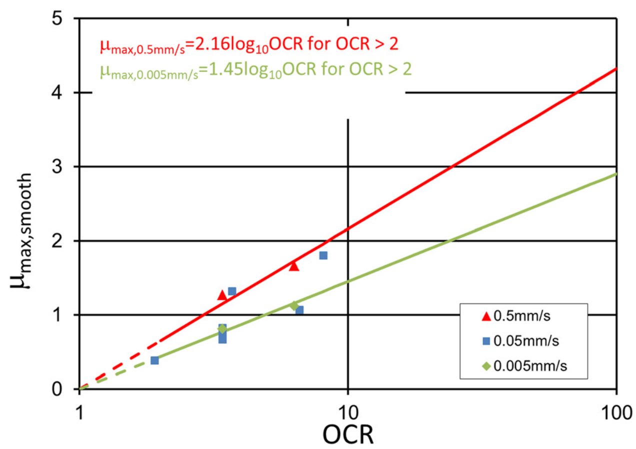
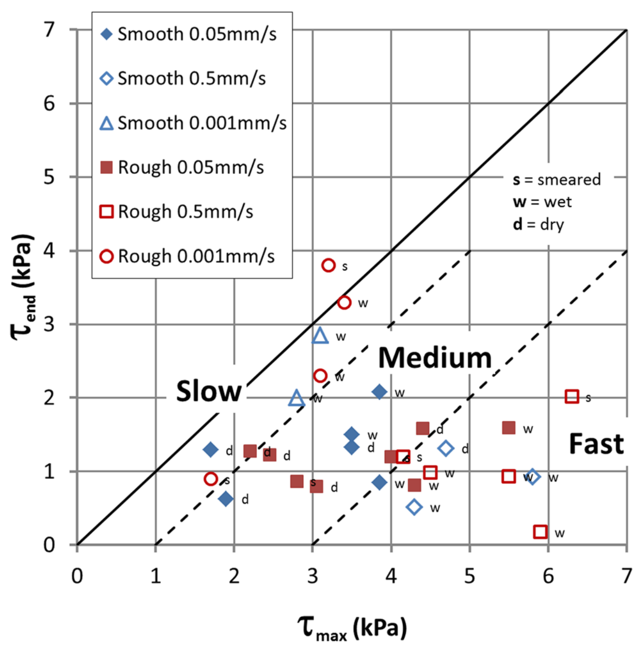


FIG. 15 End of pass 1 interface shear strength plotted against peak interface shear strength.



Notwithstanding the scatter, it is seen that slow "drained" tests feature the smallest drop from peak to residual strength, if it is accepted that enough displacement has occurred for the strength at the end of the pass to be treated as residual. The greater the speed of shearing is, the larger the drop from peak to residual strength. Approximate bands of data recovered at various speeds are shown in the figure. It is striking that the residual strength data in **Fig. 15** suggest that positive pore pressures induced in an undrained zone of disturbance will dissipate if sufficient drainage time is permitted. On the other hand, the peak strength data in **Figs. 13** and **14** for the interfaces, and **Fig. 9** for the internal soil shear strength, show exactly the opposite effect, suggesting that negative pore pressures developed in undrained shearing can dissipate with time and permit a reduction in peak strength. The mechanism underlying the apparent development of positive pore pressures within overconsolidated clays subject to large and rapid shear displacements, especially on rough interfaces, is worthy of further investigation.

Conclusions

The Cam-shear apparatus permits offshore soil samples recovered from site to be fitted inside a shear box in the form of PTFE blocks, and sheared under very low normal stresses. The whole box can be slid over a sheet of interface material, or it can be split to find the adjacent soil shear strength. The Cam-shear device is capable of measuring a soil shear strength of 1 kPa within 0.05 kPa. Tests have been reported here on reconstituted samples, in an effort to discover trends linking the peak and residual interface strengths of a West African clay with speed, distance, and duration of shearing, the overconsolidation ratio of the clay, and the interface roughness.

The drainage condition experienced by the sheared soil samples is highly dependent on the rate of shearing. A discussion on a method for estimating the drainage condition at both peak and end-of-test stages is presented. This method utilises the Gibson and Henkel approach to determine the drainage condition at peak strength and then assumes the applicability of Terzaghi's standard consolidation theory to determine the drainage condition at the end of the test. Based on the soil–soil shear tests completed in this study, it was found that at a shear rate of 0.001 mm/s, peak strengths were obtained in partially drained conditions, occurring at a shear displacement of less than 3 mm. Both 0.05 and 0.5 mm/s shear speeds should have generated undrained peak soil–soil strengths. A shearing distance of 12 mm corresponding to the end of the soil–soil test was sufficient to generate drained strengths at 0.001 mm/s and partially drained conditions for tests undertaken at 0.05 and 0.5 mm/s. Estimates of the degree of drainage were also made for interface shear tests. Because the displacements to reach peak strength tended to be smaller, and the drainage distances were longer, the degree of consolidation at peak strength was smaller than in the soil–soil tests. Because the interface tests ultimately displaced further, the proportional consolidation at the end of the tests was, however, greater.

The shear rate and OCR are each observed to influence the peak strength, with faster rates and higher OCR conditions corresponding to higher peak strengths both for soil–soil tests and interface tests. This is consistent with the anticipated induction at peak strength of negative excess pore pressures in undrained tests on overconsolidated clays. Plotting the peak interface friction coefficient against log OCR resulted in linear correlations consistent with the predictions of Critical State Soil Mechanics, but with a gradient increasing with shear speed. It was demonstrated theoretically that degree of drainage should have a negligible influence on peak strength if overconsolidated clay remains elastic up yield, so the shear speed effects may be more related to the suppression of creep, as described by rate process theory.

Fast tests continued to show a fall from peak strength throughout the shearing process whereas slow tests tended to plateau after an initial small peak in strength. Analysis of the shearing displacement required to reach 50 % of peak strength in fast tests showed that rough interfaces require 30 mm in monotonic shear, whereas smooth interfaces required less than 10 mm. The Cam-shear apparatus appeared to be capable of providing sufficient monotonic displacement for the estimation of residual strength values; however, there may be a concern regarding the unrepresentative surface conditions encountered at large displacements. Furthermore, the prior conditioning of the interface (wet, smeared, or dry) was shown not to significantly influence the measured shear strengths.

The residual (end of test) interface friction coefficient showed a strong reduction with increasing shear speed, the opposite trend to that of the peak strength. This suggests that positive excess pore pressures can be generated continuously inside the localising shear zone that forms following the development of peak strength. Further research could usefully be conducted to explain this behaviour, notwithstanding the semantic and experimental difficulties of defining and measuring representative pore pressures on shearing surfaces.

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