Interface Shear Testing for Pipelines

by
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"I hereby declare that, except where specifically indicated, the work submitted herein is my own original work."

Technical abstract: Interface Shear Testing for Pipelines

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The world now demands oil and gas resources from deeper and further out into the sea, which increases the need for more offshore pipelines. It is preferable to install deepwater pipelines by laying the pipeline onto the seabed.

These pipelines experience cycles of thermal expansion and thus shear against the seabed when in operation. To study the induced forces, laboratory tests are conducted using the newly developed Cam-Tor device to better understand the pipe-soil interaction.

Other current existing shear devices such as the Cam-Shear, Tilt Tables and the Low Stress Shearbox are also capable of shearing an interface against soil at low axial stresses but there are still common limitations of these devices. The Cam-Tor device is designed to address a few of these limitations.

The aspects investigated in this project are: shearing of natural core sample, cyclic shearing behaviour, the influence of effective stress, the influence of interface roughness, and the effects of over-consolidation. All the tests undertaken was conducted on West African marine clay. In addition to this, the importance of pore-water measurement motivated the attempt to incorporate pore-water pressure transducer in the Cam-Tor device.

The tests were interpreted based on the critical state soil mechanics model, the theoretical framework proposed in Hill et al. (2012), as well as from a micromechanics perspective. The rotational results from the tests are converted to linear form by considering shearing at the characteristic radius, 0.72 R.

The cyclic behaviour of pipe-soil shearing and the effect of pauses between each cycle is an area of interest. Tests of cyclic shearing on soil samples with different over-consolidation ratios were also conducted. The findings in this project shows that shear reversal does not affect interface friction as much as the introduction of pauses between shear reversals, as the soil is allowed to re-consolidate under constant applied vertical stress. It was also found that the eventual μ values during cyclic shearing surpasses the drained value obtained during monotonic shearing. In this case, critical state soil mechanics model cannot adequately explain the behaviour whereas the understanding of micromechanics during shearing is needed to develop new perspectives in this shearing mechanism.

The findings from the effects of over-consolidation ratio (OCR) is also puzzling. Soil with OCR 6 shows much more prominent strength hardening behaviour compared to soil with OCR 3. However, soil with OCR 3 shows lower interface friction than normally consolidated soil. The packing and crushing action during the series of cycles and pauses has to be a major factor in determining interface friction.

Tests conducted on three interfaces of different roughness, over a range of two orders of magnitudes, shows counter-intuitive results. It is commonly understood that rougher

interface would give higher friction values. In this case however, the roughest interface has interface friction value between the 2^{nd} roughest and the smoothest interface. This may be due to the jamming of particles between the 'teeth' of the very rough interface surface. This packing could 'smoothen' the interface, giving an interface friction value that is much lower than expected.

Tests conducted on natural core samples, specifically in West African Clay which contains crush-able pellets show that pellets are crushed even under low speed shearing. This has not been the case in the remoulded clay samples as the measured interface friction was much higher and suggest drained shearing conditions.

The observed shearing behaviour of clay with high faecal pellet content questions many conventional way of interpreting shearing behaviour. Further study needs to be conducted in order to gain a deeper insight into the issue.

Some future work recommended includes: to conduct Particle Image Velocimetry analysis on cyclic shearing, and shearing using different interface roughness. Further tests can be conducted using different shearing speeds and to identify the speed at which viscous effect might occur. As for future development of the Cam-Tor, further work is required on the implementation of pore pressure measurements and verification of its characteristic radius.

Acknowledgements

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Contents

Nomenclature

c_v	Coefficient of 1-D Consolidation (mm^2 s$^-1$)
H_{dr}	Drainage path distance (mm)
s_u	Undrained Shear Strength (kPa)
u	Pore pressure (kPa)

Symbols

μ	Interface friction coefficient, $\frac{\tau}{\sigma_v}$
σ'_c	Preconsolidation stress (kPa)
σ_v	Vertical total stress (kPa)
σ'_v	Vertical effective stress (kPa)
τ	Interface shear stress (kPa)
t_m	Time to mobilise peak shear stress (s)
t_{end}	Time to end of test (s)
U_f	Pore pressure dissipation ratio

Subscripts

max	maximum value
min	minimum value
$mean$	arithmetic mean
res	residual value
$peak$	maximum value

Abbreviations

CPZ	Crushing pellet zone
CSL	Critical state line
LL	Liquid limit
NC	Normally consolidated
OC	Overconsolidated
OCR	Overconsolidation ratio, $\frac{\sigma_{v,max}}{\sigma_v}$
PIV	Particle image velocimetry
PL	Plastic limit
I_p	Plastic index
PPT	Pore-water pressure transducers
PWP	Pore water pressure
PSD	Particle size distribution

1 Introduction

Palmer (2008) aptly described pipelines as such:

> *Submarine pipelines are the arteries of the oil and gas industry, important in engineering practice and full of excitement and interest, though at first sight less spectacular than platforms and floating production systems.* (Palmer, 2008, Preface).

1.1 Background

The depletion of oil and gas resources in shallower waters have driven offshore developments further into deep water. Deepwater oil and gas reserves have stretched pipelines installations across seabeds to lengths greater than 1,000 km (e.g. Nord Stream Pipelines and Langeled Pipelines) and depths greater than 2,000 m (e.g. Independence Trail Pipeline in Gulf of Mexico) (Dean, 2010, chap. 7). Pipeline designs have hence become a more significant aspect in offshore developments, requiring research attention for economic, environmental and safety reasons.

In shallower waters, pipelines are laid and buried inside a seabed trench to safeguard against trawling activities or gouging of icebergs. This installation method is not necessary for deepwater pipelines. Instead, deepwater pipelines are commonly preferred to be installed by simply laying the pipeline onto the seabed. Without the contributing resistance from soil cover, the understanding of soil properties and the soil-pipeline interaction becomes crucial in pipeline design.

During pipe-laying operations, bending of the pipe may occur as one end of the pipe can be in contact with the seabed before the other end. This causes the soil behind the touchdown point to experience additional stress on top of its self weight (Zhang et al., 1999; White and Cathie, 2011). As a result, soil during pipeline shearing can be said to be slightly over-consolidated when shearing [1].

During the channeling of resources, extremely high temperature and pressure is required inside the pipe to ease the flow and prevent solidification. This causes cyclic expansion and contraction of the pipe, which results in walking of the pipe in the axial direction. The rate of pipe 'walking' between shutdown and start-up is dependent on the axial pipe-soil resistance. Figure 1 shows that when interface shear friction is lower, (i.e. $\mu_a = 0.10$ in the figure), the pipeline will experience greater end expansion due to greater pipe 'walking'. If axial resistance is high, the pipeline will not 'walk' as much, and hence will not develop axial compression stresses which may be high enough to induce buckling.

[1] The soil is said to be over-consolidated if its current vertical stress is lower than the vertical stress experienced before.

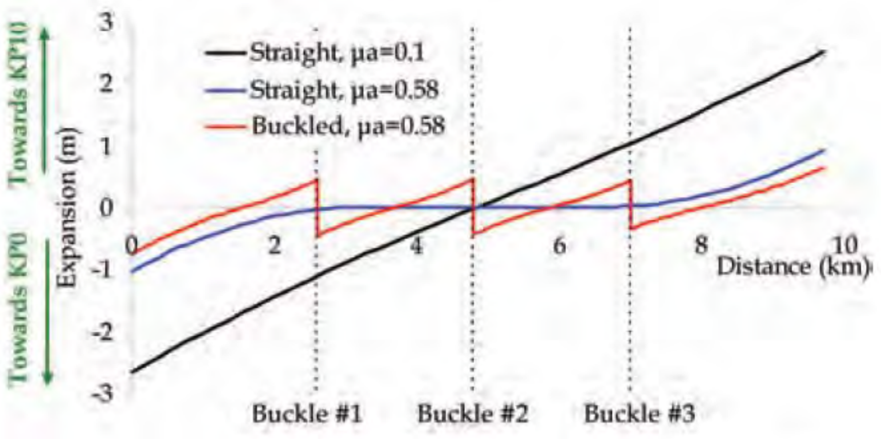


Figure 1: Displacement along a short pipeline with lateral buckle. Source: Bruton et al. (2007).

1.2 Pipe 'Walking'

Pipeline 'walking' is not a limit state concern, however, it should be understood well to prevent failure at mid-line or end connections (Bruton et al., 2007).

While increasing lateral restraint and introducing expansion spools help prevent buckling, a novel design approach is to work with, instead of against, pipeline buckling. The solution is to allow controlled buckling to relieve expansion. The typical lateral pipe movement within this pre-engineered buckle is significant and of a magnitude of five to ten pipe diameters as seen in Figure 2 (Bruton et al., 2007; Cheuk et al., 2007).

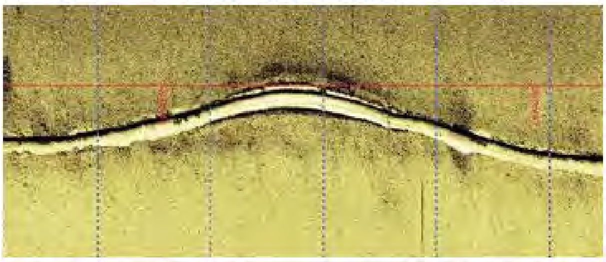


Figure 2: An image of a lateral buckle. Source: (Bruton et al., 2007; Kuo, 2011)

1.3 Control of pipe walking

Controlled buckling can only be achieved with a good prediction of the pipeline's lateral displacement. In fact, it is not sufficient to identify a safe resistance for pipeline design. Instead, a resistance envelope is needed where the upper and lower bounds are defined.

Traditionally, soil mechanics focuses on ensuring designs satisfy ultimate and serviceability limit states under working loads. This static design approach is no longer sufficient. A model to predict and quantify the change of soil resistance over large displacements is required.

1.4 Measuring axial resistance

The main design input parameter is the value of axial resistance. Using the concept of a simple friction law (Coulomb friction), the axial resistance (presented as the interface friction coefficient μ), is quantified by linking the pipe weight to the maximum available shear resistance seen in equation (1):

$$\mu = \frac{\tau}{\sigma_v} \tag{1}$$

This parameter is commonly used for both analytical and finite element based models. Unlike the static friction coefficient, the large amplitude displacement behaviour is taken account by considering the change of this coefficient with shearing strain.

Offshore soil and natural sediments are site specific. For example, the West African seabed is found to have biological 'enhanced' sediments in the top 0.5 to 1 meter soil layer which results in an increase in soil shear strength (Kuo, 2011).

The high precision required for the prediction of pipe-soil axial resistance necessitates the use of laboratory tests to cope with the site specific variability of soils. Another reason that highlights the need of shear devices is to improve the understanding of the relationship between rate of shearing, relative roughness of the interface, and soil stress state.

Albeit the importance of the role of shear testing devices, there are only a limited number of laboratory based interface shearing devices reported in current literature. Soil element laboratory testing is advantageous for its relatively straightforward interpretation of results and to allow repeatability in testing.

1.5 Existing Shear Devices

Previously (i.e. before the need to test soil for offshore pipeline design), soil tests were focused mainly on internal strengths of reconstituted soils and interface shear tests for geotechnical applications, where the applied normal stress is high.

In the application of pipelines, normal stress applied on the soil is low and constant, shearing distance is long, and shearing rate is varied over a broad range. The shear strength of soil is non-linear and it is highly stress-dependent in plastic soils at low effective stresses (Skempton, 1985).

Many of the commonly used soil-solid interface shearing test devices have high internal friction thus limiting the minimum achievable confining stress. Another common limitation shared by these devices is a short shearing distance, restricting the ability to reach residual shear strengths, as that will likely occur during pipeline shearing.

Hence, current understanding of the interface strength at low axial stresses is inhibited by the limited data from devices tailored for offshore pipeline design purposes. Table 2 summarises the interface shearing devices reported in current literature.

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 the nature of sample preparation. -Apparatus internal friction limits the minimum achievable stress.	Colliat et al. (2011)
	Remoulded	Rough and smooth	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 the nature of sample preparation. -The stress distribution on the interface varies depending on the angle of the table. -Shearing displacement is limited to the size of the table.	Pederson et al. (2003) Najjar et al. (2007)
Low stress shearbox (UWA)	Remoulded	Rough and smooth	>2.5	-Only remoulded samples have been tested. -Shearing displacement is limited to the size of interface material.	White et al. (2012) Hill et al. (2012)
Cam-Shear	Remoulded and natural	Rough and smooth	1 to 4.5	-Shearing displacement is limited to the size of the box (190 mm)	Kuo et al. (2010)

Table 2: Summary of existing interface shearing devices, modified after Bruton et al. (2007).

Three devices mentioned in Table 2: the tilt table device described in Najjar et al. (2007), the low stress shearbox developed at University of West Australia (UWA)(White and Cathie, 2011) and the Cam-Shear developed at University of Cambridge (Kuo et al., 2010) have all achieved tests at very low normal stresses. However, these devices are unable to shear large distances.

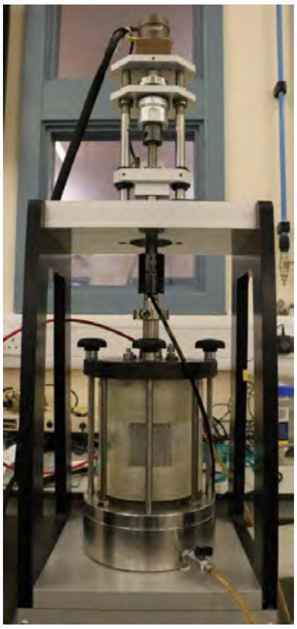
A stroke length of 190 mm can be achieved by the Cam-Shear apparatus. This length is sufficient for interface friction coefficient be sheared long enough to reach its residual value. The predicted length required for drained conditions to prevail is about 50 mm (Kuo et al., 2010). There is however a concern that at large displacements, the interface surface conditions may not be representative of pipeline-soil shearing (Ganesan et al., 2014).

As for the method of attaining residual shear strength by the tilt table device, although it has sheared soil samples for up to 100 mm, the data obtained is not representative of monotonic shearing. [2]

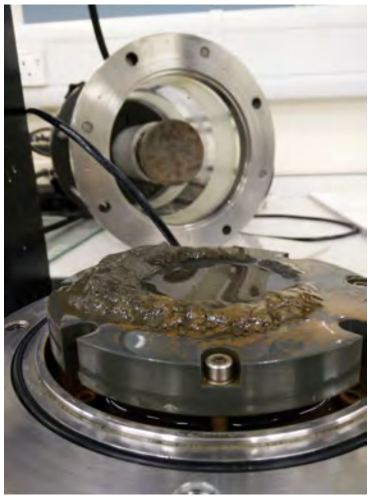
An interface device is needed to address the aforementioned limitations inherent to existing shear devices and to improve the accuracy of the measured interface friction value.

[2] The tilt table shears the specimen for 13 mm in one direction by raising tilt angle. Successive raising and lowering is then carried out to achieve cumulative shearing distance of up to 100 mm.

A new addition to the list of existing interface shearing devices is the newly developed Cam-Tor machine developed at University of Cambridge in collaboration with BP and Fugro (UK). [3] While most interface shearing devices mentioned in Table 2 are modified from existing shearing devices for the purpose of pipeline applications, the Cam-Tor is developed specifically for pipe-soil shear testing.



(a) Set-up of Cam-Tor during testings.



(b) Soil sample holder attached to the base of Cam-Tor after removing the pressure cell and loadcell.

Figure 3: The Cam-Tor Device.

The Cam-Tor is capable of the following:

- Shearing large distances of several meters, which is similar to pipeline shearing can be achieved by Cam-Tor. It can also carry out cyclic shear tests of any cumulative shear displacement.
- Testing samples that are still intact and contain natural sediments, in addition to testing reconstituted soil samples.
- Applying low axial stress of a broad range from 2 to 50 kPa.
- Shearing samples at rate of 0.0001 to 1 mm/s.
- Testing with a range of interface roughness.
- Having a constant shearing surface area.

[3]New data from the Cam-Tor interface shear testing device is presented in Kuo et al. (2015). The tests described in Kuo et al. (2015) was undertaken by Vincent as part of her Fourth Year Project (Vincent, 2014).

The arrangement of the interface applying a downward axial stress on the soil sample and the ability to maintain constant stress during shearing is representative of pipe-soil shearing.

1.6 Motivation and Objectives

This project is motivated by a lack of understanding of soil-solid interaction and long distance shearing. Better understanding of the pipe-soil shearing can be achieved with a purpose-designed shearing device. A testing device is also important for the purpose of determining the interface friction coefficient, a key design parameter that quantifies the axial resistance during thermal expansion of pipelines. The Cam-Tor device is developed specifically for these purposes.

This reinforces the importance of undertaking a series of tests on the Cam-Tor to better understand the device's capabilities and to investigate the effects of different pipe design parameters, shearing speed and soil conditions on the interface friction coefficient. The author has also explored the possibility of incorporating pore pressure transducers at the interface to gain additional insight into interface shearing behaviour.

The objectives of this project are to:

- Build upon prior work done by the previous fourth year project on the Cam-Tor.[4] There are many factors that influence the axial resistance of the pipeline. These factors also influence each other. Parametric testing is carried out to gain more information to better understand these interactions.

- Investigate the effects of cyclic shearing and re-consolidation between episodes.

- Investigate the shearing response of soil samples from offshore West Africa which have been reported by Kuo and Bolton (2014b) to have unique shearing properties.

- Explore the potentials of the Cam-Tor device. The possibility of incorporating pore pressure transducers is explored.

2 Literature Review

2.1 Theories

2.1.1 Influence of pore-water pressures in shearing

The shearing resistance response is generally governed by effective stress friction. Based on Terzaghi (1996)'s principle of effective stress, the effective stress, σ'_v is a function of

[4] Refer to §2.2 for a summary of the findings from previous project.

equation (2):

$$\sigma'_v = \sigma_v - u \tag{2}$$

where σ_v is total vertical stress and u is the pore water pressure.

When the soil grains shear against each other, there will be dilation or contraction of the soil depending on the initial packing of the soil grains. However, the low permeability of fine soils does not allow water within the voids to escape. This causes the soil to shear at constant volume, i.e. no dilation or contraction. To compensate, pore water pressure (PWP) will increase if the soil has the tendency to contract and vice versa.

In fast shearing, excess PWP generated during shearing is not allowed to dissipate quickly, especially in clays. Conversely, in slow shearing, any excess PWP generated has more time to dissipate. This is modelled by Schofield and Wroth (1968) using Critical State Soil Mechanics (CSSM) shown in Figure 4.

The Cam-Clay model yield surface is described in equation 3:

$$\frac{\tau_{max}}{\sigma'_0} = \mu_{crit} \ln(\frac{\sigma'_c}{\sigma'_0}) = \mu \tag{3}$$

where: μ is the variable friction coefficient, and μ_{crit} is the critical state friction coefficient. The four most basic stress paths are labeled in the diagram as 1-4. Normally consolidated (NC) soils contract when shearing and hence do not exhibit any peak shear stress before reaching the critical state line (CSL). Over-consolidated (OC) soils are those that have been pre-consolidated to a higher vertical stress than that applied during shearing. In the diagram, stress paths 3 and 4 are located at the 'dry' (dilative) side of the critical state line (i.e. $\sigma'_0 < \frac{\sigma'_c}{2.7}$). These soils dilate when sheared, reaching a peak stress before falling to critical state line.

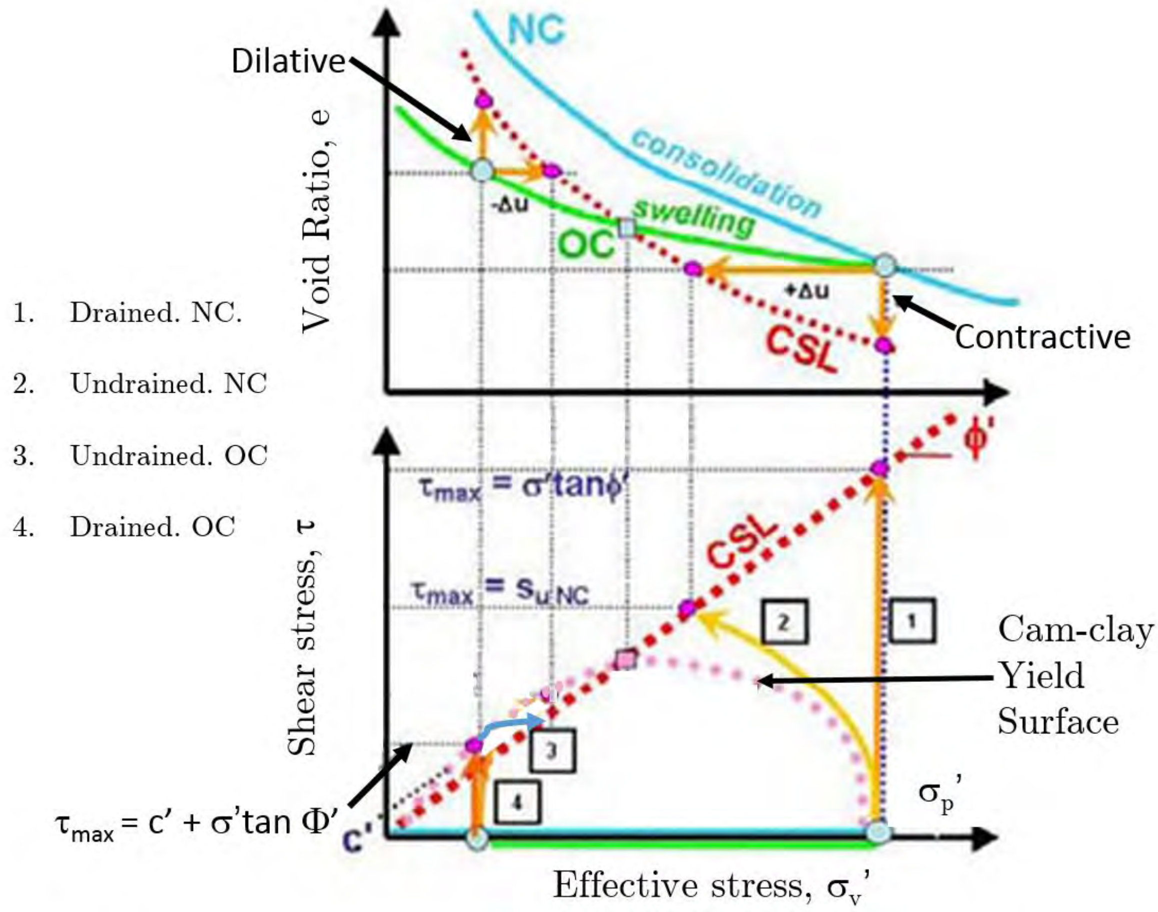


Figure 4: Critical state soil mechanics CSSM concept. Modified from Mayne.

From the CSSM model, we can see that undrained shearing of NC soils exhibit lower stress values as they reach the CSL. The stress path of undrained shearing of NC soils took a turn to the left as effective stress reduces due to the generation of positive excess PWP.

2.1.2 Interface shearing and drainage

The critical state soil mechanics model is a convenient framework for prediction of soil-soil shearing. CSSM can also assist in the interpretation of soil-solid shearing.

During shearing, the particles will be mobilised and this will generate excess pore-water pressure (PWP). The rate at which the soil particles are sheared will influence the magnitude of the excess PWP. If the soil particles are sheared at a low rate, excess PWP generated will be given time to dissipate. If sheared at a high rate, excess PWP will not be able to dissipate. The soil is described as shearing in undrained conditions in the case of fast shearing. The shear stress corresponding to undrained shearing, known as undrained shear stress, will be of a lower value than drained shear stress if excess PWP generated is positive.

After shearing for long enough to allow the excess PWP to dissipate, the shear stress will increase and eventually reach the drained value.

The logarithmic sigmoid function curve (also known as backbone curve) observed in Figure 5 shows that residual stress ratio is lower at higher speeds. The term 'residual stress ratio' is similar to the minimum interface friction coefficient, μ_{min} terminology that will be used in this report.

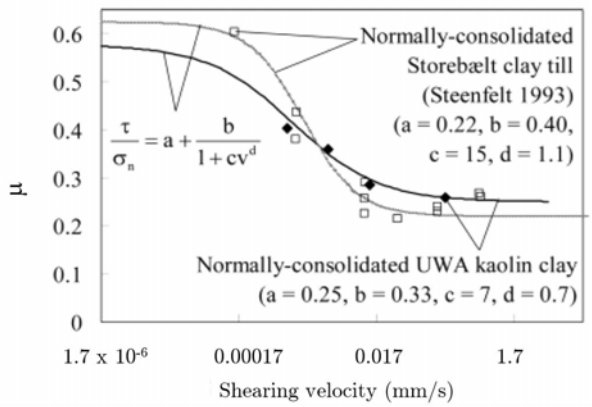


Figure 5: Interface shearing testings on kaolin clay and Storaebelt clay till. After White and Cathie (2011)

Since the rate of dissipation of excess PWP will influence the interface shear stress, parameters like soil permeability, k, and the coefficient of consolidation, c_v are useful to predict the shearing response.

Gibson and Henkel (1954) derived the expression given in equation (4) to relate time to mobilise peak shear stress, t_m to the degree of PWP dissipation, U_m of normally consolidated clays.

$$t_m = \frac{H_{dr}^2}{2(1-U_m)c_v}, \tag{4}$$

where c_v is the one dimensional consolidation coefficient and H_{dr} is the length of drainage path.

From equation (4), the time needed to mobilise peak shear stress can be found. The predicted time needed to achieve $U \geq 0.9$ can then be found by using equation (5) (Ganesan et al., 2014).

2.1.3 One-Dimensional consolidation

According to Taylor's Square Root Time Method (Taylor, 1948), the coefficient of one-dimensional consolidation, c_v can be estimated from the consolidation stage of the tests conducted.

To obtain the c_v, a graph of settlement vs square root of time is plotted. A settlement curve can be segmented into three general stages: immediate elastic response, primary consolidation stage and secondary creep stage. The initial elastic response of soil loading is ignored. The linear portion of the consolidation curve is extrapolated back to the origin to obtain the height of the sample corresponding to 0% consolidation. A line with a gradient 15% less than the original is then extended from the origin to the point of intersection with the non linear portion of the settlement curve. The point of intersection is defined to be 90% consolidation, corresponding to $U_{0.9}$ and t_{90}. Equation 5 is then used to calculate c_v of the sample:

$$c_v = 0.848\frac{H_{dr}^2}{t_{90}}. \tag{5}$$

From equations (4) and (5), we can estimate the minimum time required to dissipate the pore-water generated from

$$t_{end} - 0.5t_m \geq 0.848H^2/c_v. \tag{6}$$

2.1.4 Mechanisms affecting axial pipe-soil interaction

The framework in Figure 6 proposed by Hill et al. (2012) is very interesting as it gives a compact overview of the possible interactions between the factors that affect axial resistance of pipelines. Applying the concept of CSSM, this framework allows the use of elemental soil testings to acquire fundamental soil properties of different site-specific soils. The results can then be linked to other factors to determine pipeline design parameters.

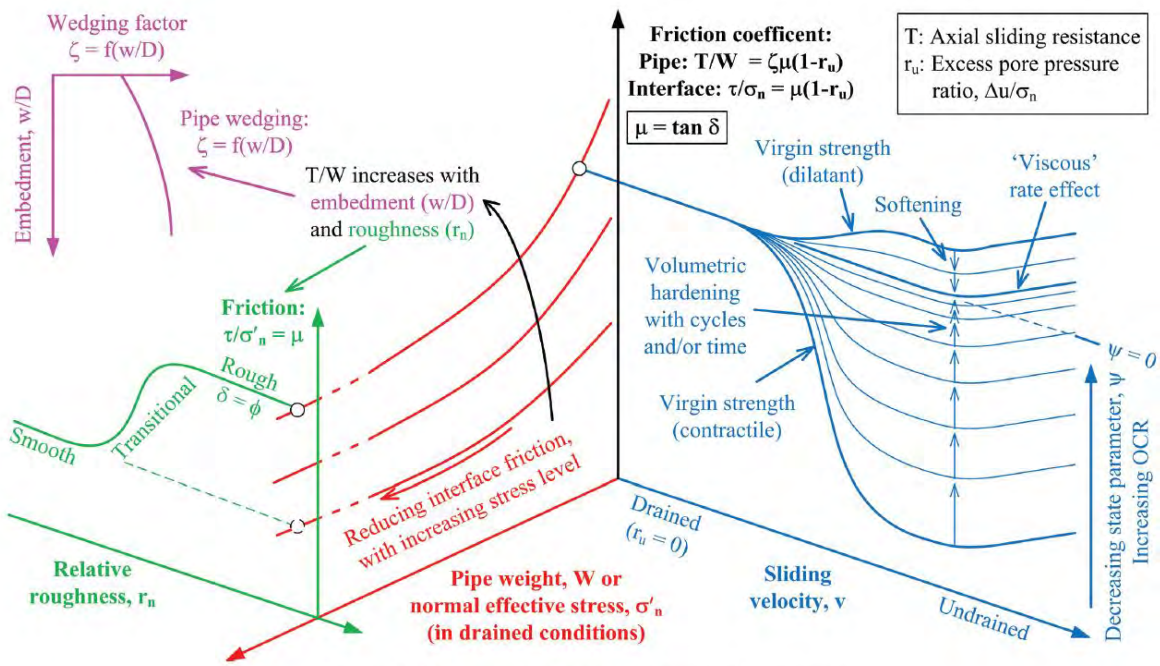


Figure 1: Mechanisms affecting axial pipe-soil interaction

Figure 6: Different factors affecting axial pipe-soil interaction. Source: (Hill et al., 2012).

There are four main elements in the framework and each of them is assigned a different colour. The main elements are as follows:

- red: effect of effective stress;
- blue: drained - undrained transition, as explained in §2.1.2;
- green: relative roughness between soil particles and solid surface;
- purple: pipeline applications and in situ conditions.

Shown in Figure 7 is one of the soil element test data set that contributed to the described framework. These test data are obtained from the tilt table device developed at University of Texas and shear box device developed at University of West Australia (Table 2). A few observations can be made from Figure 7:

- The effect of smoother interface which shows a lower magnitude logarithmic sigmoid function curve.
- The strengthening effect of over-consolidation.
- Velocities greater than 0.01 mm/s shows undrained shear behaviour while velocities lower than 0.001 mm/s shows drained shear behaviour.

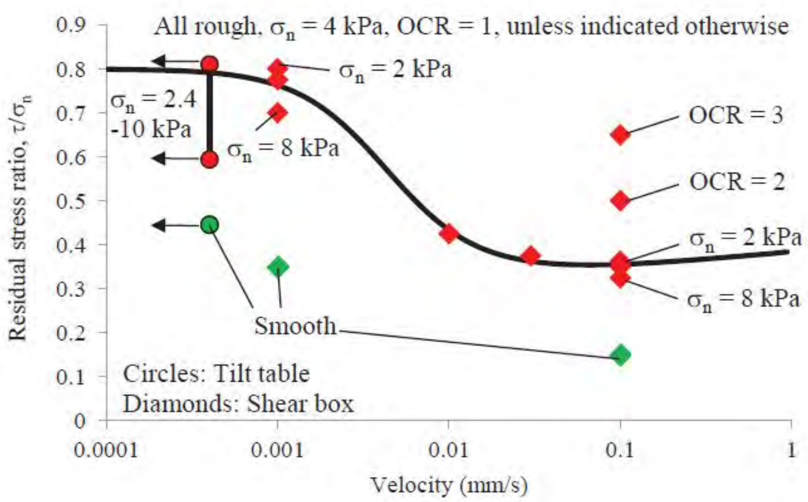


Figure 7: Soil element test data of residual interface strength. Source: Hill et al. (2012)

This project will undertake tests using the Cam-Tor with varying factors affecting axial resistance shown in the framework in Figure 6, specifically: over-consolidation ratio (OCR), shearing speed, interface relative roughness, effective stress, and cyclic shearing. This project also attempts to test natural soil samples and compares it against interface shear resistance of remoulded samples. The results obtained in this project will be compared against data presented in Figure 7 to verify the Cam-Tor as well as to contrast the effect of shearing clay samples from different marine sites.

2.1.5 Cyclic shearing and re-consolidation

The Critical State Soil Mechanics model described in §2.1.1 is used to give an insight into the response of cyclic shearing. In Figure 8, a soil sample was sheared under vertical effective stress of 40 kPa in undrained conditions. Before its subsequent cycles, excess pore water pressure is allowed to dissipate, bringing the vertical effective stress back to 40 kPa. This cyclic shearing causes the yield surface to expand and the volume of the sample to contract. This shearing behaviour of alternating cyclic and re-consolidation episodes is expected during pipelines' thermal cycling (during shutdowns and restarting of oil and gas transfer).

As shown in Figure 8, it is expected that cyclic shearing of normally consolidated or lightly over-consolidated[5] sample will result in hardening response of the soil. After sufficient cyclic shearing and re-consolidation episodes, the soil will eventually reach the point on the critical state line that corresponds to the total vertical applied stress. This value will be the same as the drained shearing value.

The CSSM model however does not predict the behaviour of cyclic shearing at the 'dry' side. This is because soil is assumed to behave elastically withing the yield surface of the CSSM model. This does not address any stress history experienced by the soil and it ignores any anisotropic behaviour. Based on previous studies on soil-soil shearing behaviour, 'work hardening' is predicted to occur if soil on the 'dry' side if sheared excessively (Davis and Selvadurai, chap. 7).

2.2 Previous Interface Shear Testings Conducted on Cam-Tor

Two types of remoulded and reconstituted soil were tested on the Cam-Tor, they are taken from Southern North Sea and offshore Angola. The tests undertaken were monotonic long distance shearing. Soil samples were sheared at two alternating speeds: 'slow' and 'fast', corresponding to 0.001 mm/s and 0.1 mm/s. The general pattern of shearing was slow - fast - slow or fast - slow - fast.

From the tests undertaken on these soil samples from two different origins, the results from Figure 9 and Figure 10 can be summarised as follow:

- During fast shearing, three distinct characteristics was observed: a peak, followed by a drop in value, and after shearing for some distance (The distance corresponded to t_{end} calculated using equation (6).)
- Interface of greater roughness generates higher residual values.
- Rate effect is distinct in rough interface shearing, but not when sheared against smooth interface.

[5]'lightly over-consolidated' is defined as $\sigma'_{v0} > \sigma'_c/2.7$

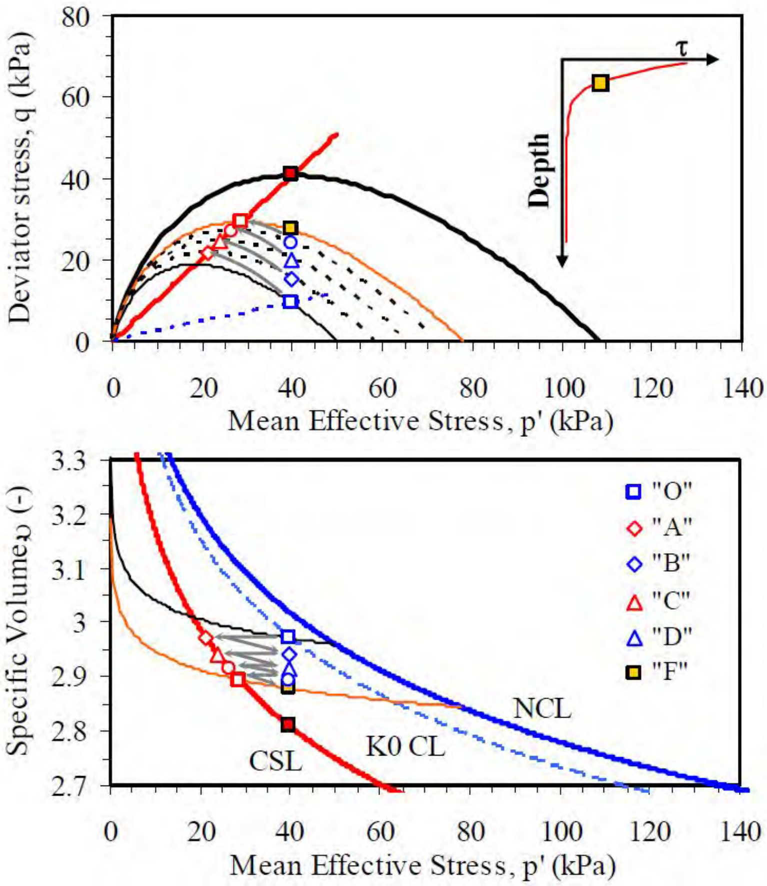


Figure 8: CSSM: soil hardening and volume reduction behaviour under cyclic loading. Source: Deeks et al. (2014).

- The peak stress is observed to be higher in Angolan soil.

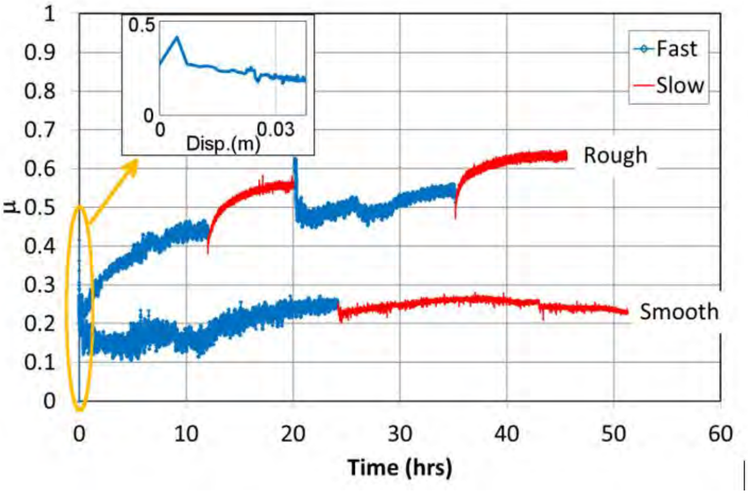


Figure 9: Example rough and smooth interface tests for SNS sample. Inset: initial fast peak plotted on semi-log axes. Source: Kuo et al. (2015).

Based on the Particle Image Velocimetry (PIV) analysis on shearing behaviour, a Crushing Pellet Zone (CPZ) of 2 mm thick is observed during the initial shearing stage.

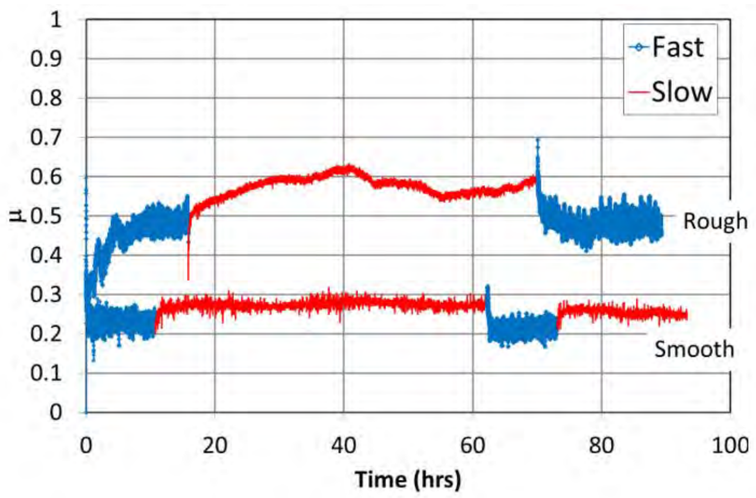


Figure 10: Example rough and smooth interface tests for Angolan sample. Source: Kuo et al. (2015).

The CPZ is approximately 1 mm thick at the end of a hundred minute shearing (several meters shearing displacement).

From the PIV analysis carried out on the Cam-Shear reported in Kuo and Bolton (2014a) and Cam-Tor carried out by Vincent (2014), the peak observed was contributed by the interlocking of soil grains at the interface. Diagonal cracks are seen to form near the interface as the soil is further sheared. Then, a greater shear zone is mobilised, where there is relative movement between soil particles .

After the peak, the shearing is described as having overcame the 'breakout' stress. The diagonal cracks 'heals' and relative movement between interface and soil is more prominent than soil-soil relative movement.

It was observed that the particle size in the CPZ at the end of the test has particle size an order of magnitude smaller than initial size. It was hypothesised that the shear mechanism and hence thickness of the shear band is related to grain size.

2.3 Natural offshore sediments in West African Clay

The site specific nature of offshore seabed sediments adds complexity to the design of pipelines.

Deep ocean West African clays at shallow sediment depths have been found to exhibit unexpectedly high undrained shear strengths, high plasticities and high water contents. These values are significantly higher than other offshore locations such as the Gulf of Mexico where water content and plasticity indices may range from 50% to 125% and 30 to 70. This is in contrast to the range of 150% to 250% and 70 to 120 found in West African Clay (Kuo, 2011).

Kuo and Bolton (2013) found evidence of the cause of this interesting 'crust' behaviour of West African Clay. The crustal material contains sand size capsules that has been identified as faecal pellets. These faecal pellets have been over-consolidated in the guts of invertebrates such as polychaetes. Being 'over-consolidated' in the guts gives these pellets enhanced meterial properties (Kuo and Bolton, 2014b).

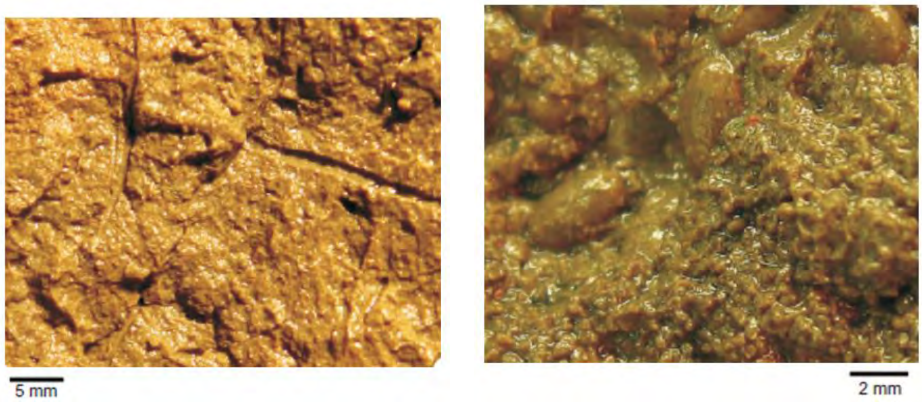
Figure 11: Pellets of greater than 300 μm

2.3.1 Effect of interface shearing with natural cores (intact material)

Variability of the soil content at different offshore sites and its consequent contribution to the geomechanical behaviour of the soil have come under scrutiny in the recent years. Shear tests on deep ocean clay crust presented in Kuo and Bolton (2014b) shows evidence of the effect of sediments in the soil sample and the need of laboratory testing of natural core material.

It is found that the voids caused by burrowing invertebrates and the faecal pellets affects the soil behaviour so much that it challenges the common assumption that soil-soil shearing is equivalent to shearing between a rough interface and soil. Counter-intuitively, the interface shear friction of a rough interface may be lower than that of a smooth interface. Furthermore, it was observed that during 'fast' shearing ($\sim$ 0.5 mm/s) of typical pipelines under normal stresses of 2 - 6 kPa, there was a complete loss of interface friction. It was hypothesised that this may be due to crushing of faecal pellets and randomness in burrow pores.

Natural core samples that were used for testing in Kuo and Bolton (2014b) demonstrates heterogeneous structure in soil samples, caused by disturbance of the dynamic sea environment as seen in Figure 12. However this heterogeneous structure is well preserved when taken from seabed, showing its potential to be tested on as an intact, natural material.



(a) Invertebrate burrows found in core sample.

(b) Faecal pellets distributed unevenly in core sample.

Figure 12: Example of core sample that contains heterogeneous structure yet intact and undisturbed soil sample. Source: Kuo and Bolton (2014b)

3 Apparatus and Experimental Tests Procedure

3.1 Cam-Tor

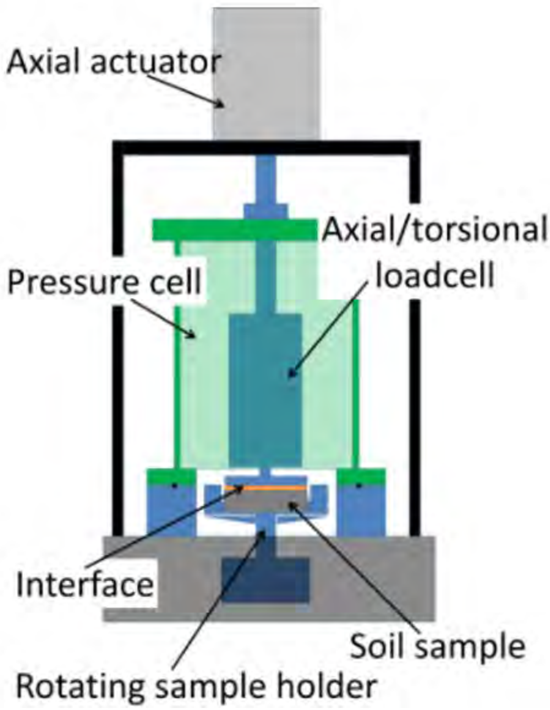


Figure 13: Schematic diagram of the Cam-Tor machine. Source: Kuo et al. (2015)

The Cam-Tor has six main parts labelled in Figure 13. The functions and characteristics of each part are explained briefly below:

- Axial actuator - Controls the axial movement of the loadcell and the interface attached to it to apply vertical stress to the soil sample.

- Loadcell - The cylindrical body holds the assembly of both axial and torsional loadcell within it. The axial loadcell is capable of maintaining constant stress application on the soil. The torsional loadcell measures and logs the torque exerted on the interface as the rotating soil sample below shears against the interface.

- Pressure cell - Prevents evaporation of water besides maintaining pressure within it if necessary. Practically, for this project, it was used to give rigidity to the loadcell and interface set-up.

- Sample holder - The sample holder has a few parts in its assembly. It has a cutting ring that is capable of preserving the sample's natural structure when it is removed from the core and fitted into the sample holder. The sample holder is screwed onto a rotating mechanism in the Cam-Tor.

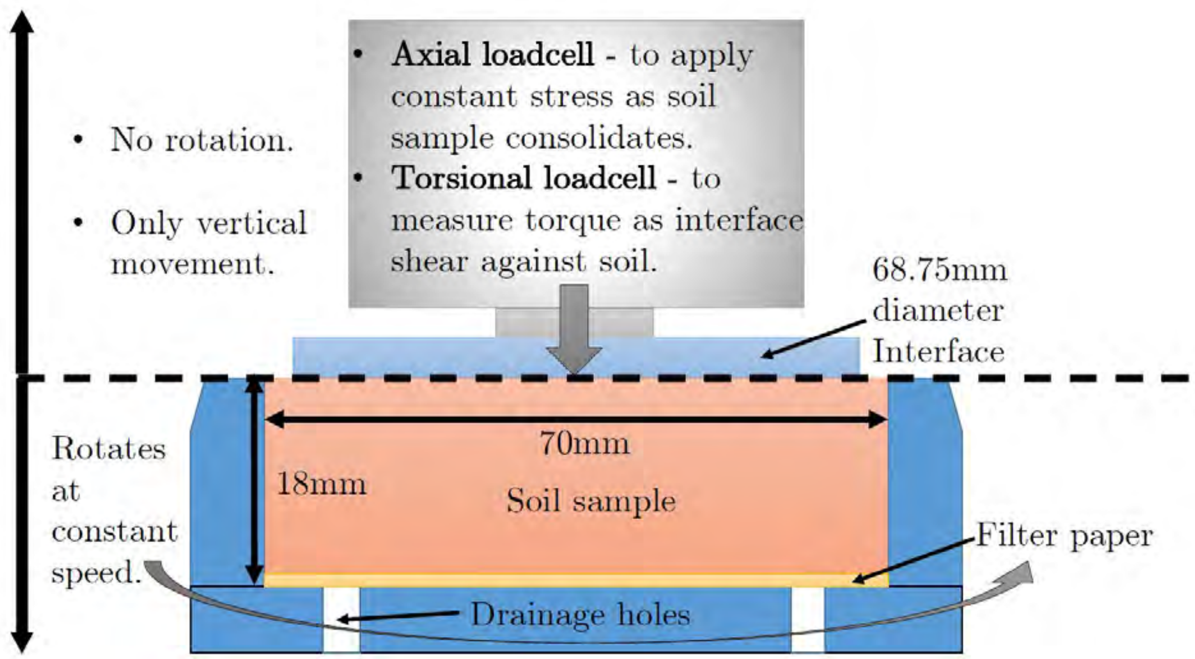


Figure 14: Schematic diagram of Cam-Tor test.

The soil sample rotates at a constant shear rate below the interface as illustrated in Figure 14. As it rotates at constant rate, the soil sheared by the interface will experience a distribution of strain displacement along the radius of the disc shaped interface. The control parameters of the experiments and data obtained from the tests are analysed at a characteristic radius of 0.72R, where R is the radius of the sample. The factor 0.72 is determined by shear stress analysis described by (Kuo et al., 2015).

3.2 Sample Material and Preparation

3.2.1 Soil Sample

Two types of West African soil sample are used for testings. The samples are taken from two different offshore sites: offshore Angola and Congo. These samples are found to contain faecal pellets.

A summary of approximate values of soil properties are tabulated in Table 3.

	Liquid limit, LL (%)	Plastic limit, PL (%)	Plastic Index, I_p (%)	Natural Core Sample Water content (%)	Pellet percentage (%)
Angolan	130	Unknown	Unknown	100	42
Congolese	190	65	129	160	24

Table 3: Soil sample properties. The values in the table are just approximations from natural core samples, soil investigation data from Fugro and results from wet sieving. Due to the heterogeneous nature of natural soil, these are only indicative values.

The liquid limit of the Angolan soil and Congolese soil are approximately 130 % and 190 % respectively.

3.2.1.1 Results from wet sieving

The particle size distribution (PSD) of the soil sample shown in 15 is determined by wet sieving. As mentioned in Kuo and Bolton (2013), wet sieving was carried out instead of conventional PSD analysis to preserve and prevent pellets from breaking. In contrast to the drying, crushing and remoulding process of a conventional PSD analysis, the bulk sample is instead carefully washed through a set of eleven sieves of different sizes during wet sieving. The natural core sample was wet sieved once before remoulding. Wet sieving is then conducted on the remoulded soil sample to illustrate the difference between natural and remoulded soil sample.

PSD results from Vincent (2014) on Southern North Sea (SNS) soil sample were also included to compare and contrast the marine soils from two different places. These SNS soil samples were reported to not contain any faecal pellets.

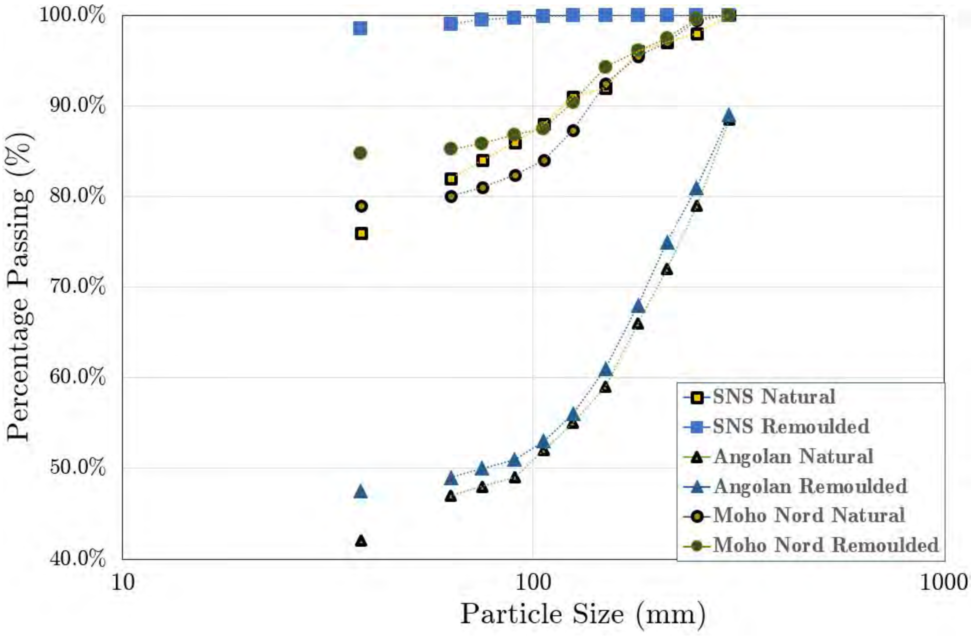


Figure 15: PSD of three different soil samples

From the PSD, it is identified that:

- The soil sample from offshore Congo has 24% faecal pellets by original total dry mass.
- The soil sample from offshore Angola has 42% faecal pellets by original total dry mass.

Both soil samples are found to also contain glauconised faecal pellets. Glauconised pellets are biologically enhanced and are found to be harder. They appear to be greenish in colour, distinguishing themselves from other pellets. Also found in the clay matrix is a variety of marine debris, namely, sponge spicules, shell fragments and diatoms as illustrated in Figure 16.

3.2.2 Sample preparation

Tests were undertaken on both naturally occurring soil samples (intact) and remoulded and reconstituted soil samples.

3.2.2.1 Preparation of intact soil sample

The extraction of the soil sample from its the cylindrical core tube is done by sectioning a disc of soil sample with height greater than the sample holder's height. A cutting ring that comes with the sample holder assembly is pushed into the soil slowly as shown in Figure 17. A cheese wire is used to trim the soil around the cutting ring. Figure 18a shows the soil sample after the trimming process. Figure 18b shows how the ring is placed into the sample holder.

Figure 16: A photo taken after material trapped in sieve larger than 300μm are dried. Pellets photographed here shows two distinct colour. The pale greenish-brown pellets are glauconised and hardened faecal pellets. (Kuo, 2011) The cuboid-like material can be identified as a diatom.

Figure 17: Cutting ring seen to be pushed into a disc of core sample.

3.2.2.2 Preparation of remoulded and reconstituted soil sample

The water content of the remoulded soil sample is targeted to be slightly lower than the liquid limit. The approximate water content used in Angolan soil is between 110 - 120 %. For Congolese soil, the water content of the sample used were between 170 - 185 %. A soil sample from the core is softened by soaking it in water. After soaking it overnight, the sample will be tested on its water content to determine the additional amount of water needed to achieve the desired water content. It is then remoulded until homogenised as shown in Figure 19 and poured into the sample holder in Figure 18b.

(a) Sample prepared for intact material shear testing.

(b) Cutting ring is fitted onto the sample holder before attaching it onto the Cam-Tor machine.

Figure 18: Preparation of soil sample.

Figure 19: PSD of three different soil samples

3.2.2.3 Consolidation

Before the start of shearing stages of each experiment, the soil sample will be consolidated up to its pre-consolidation pressure, σ'_c.

As illustrated in 14, there is a gap at the edge of the interface where material can extrude out during consolidation and shearing. The consolidation process is carried out in stages of increasing constant vertical stress. For example, to load up to 8 kPa, consolidation was carried out in three stages of increasing stress levels (2 kPa - 4 kPa - 8 kPa), where each level took a consolidation time of approximately one day. The soil sample was allowed to reach approximately t_{90} of the consolidation curve at each stress level as shown in Figure 20.

There are draining pores at the bottom of the soil sample as seen in Figure 14. The drainage boundaries are assumed to be located at the bottom of the soil sample, giving a drainage path, H_{dr} of 18 mm. Although there is gap next to edge of the interface, the drainage path length to the bottom is relatively shorter. Hence the drainage path is taken as 18 mm.

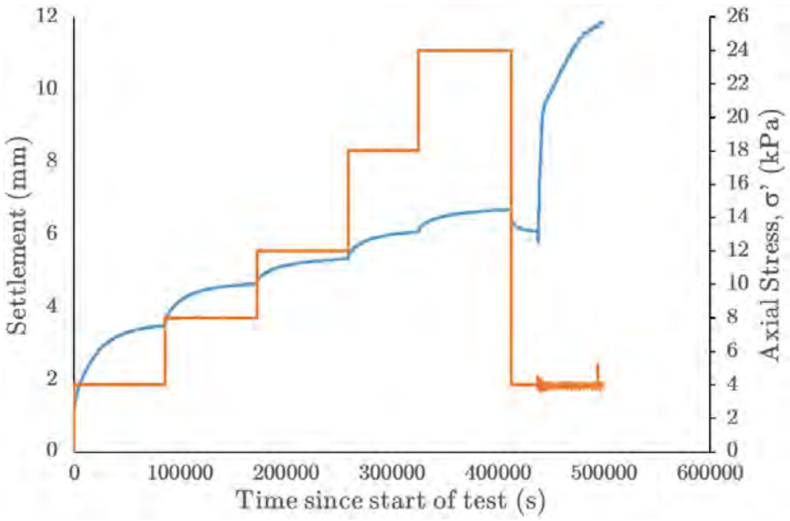


Figure 20: An example of consolidating soil sample to pre-consolidation stress, σ'_c in stages.

3.3 Interface Material

Table 4 shows the interface material's surface roughness[6]. The interface has a diameter of 68.75 mm. The porous plastic interface was used in the set up for the measurement of pore-water pressure, as explained in Chapter 5.

Material	Average roughness, R_a (μm)	
	Before testing	After testing
Stainless Steel Rough 1	6.08	5.58
Stainless Steel Rough 2	7.54	7.22
Stainless Steel Smooth	0.30	0.30
Porous Plastic (Very rough)	32.42	31.87

Table 4: Interface surface roughness.

3.4 Testing Procedure

After sample preparation and consolidation, the Cam-Tor shearing stages can begin. In this report, tests conducted are sheared under the following conditions unless stated otherwise.

- constant vertical stress of 4 kPa,
- over-consolidation ratio (OCR) of soil sample = 1 (normally consolidated),
- shear against rough interface (6.08 μm for Angolan clays and 7.54 μm for Congolese clay)

[6]The average roughness is the most universally used parameter for describing the surface asperity height(Eid et al., 2014). It is the arithmetic mean of the distance between the profile and the mean line of the surface profile.

- cyclic tests conducted have 30 cycles with 30 minutes pauses before shear reversal.

4 Results and Discussion

Outline of section

This section has three main parts: monotonic shearing, shearing on intact material and cyclic shearing.

First, the consolidation coefficient of the soil sample used is identified from the consolidation curves to justify if the drainage path assumption is valid. Results of the drained values found in monotonic shearing is then established for the use of comparison against other results.

Results of fast shearing is then presented and its key features is discussed.

Before discussing the effects of different parameters on cyclic shearing with pauses, the effect of pauses and the difference between cyclic and monotonic shearing is discussed.

Measured parameter

The measured interface friction coefficient, μ between the interface and the soil is described as

$$\mu = \frac{\tau}{\sigma_v}. \tag{7}$$

The torque measured during shearing is corrected for an initial reading that is caused during the consolidation phase. The torque values are then converted to shear stress at the characteristic radius (i.e. 0.72 R) as mentioned in § 3.1.

Tests conducted on Angolan clay were coded with "AI" or "AR" followed by the test number, whereas tests conducted on Congolese clay were coded with a "C" followed by the test number.

Test programme

For Angolan clay, both remoulded and natural (intact) soil sample were tested. Tests on intact soil sample were labelled "AI" while remoulded ones were labelled "AR".

Test results from AI 2, AI 3, AR 4, AR 6 and AR 8 are not presented in this report. These tests did not generate any useful results due to a misalignment between the interface and the soil sample [7]. The data collected from this tests showed very prominent, oscillating

[7] The interface is connected to the loadcell, while the loadcell is connected to the actuator that drives the interface downwards to apply constant pressure on the soil sample. When the soil sample settles during consolidation and shearing, the interface will be pushed into the sample holder. The gap between the edge of the interface and the sample holder boundary is very small. Any slight displacement of the actuator-loadcell-interface assembly away from its precise vertical axis will cause the interface to brush against the sides of the sample holder, causing unreliable torque readings.

at 360°.

The parameters used for each tests is tabulated in Appendix B.

4.1 One-dimensional consolidation coefficient

The one-dimensional consolidation coefficient, c_v of the soil sample can be determined from the consolidation curves. The values are found to be within acceptable ranges (an order of magnitude) when compared to data presented in Kuo (2011). Therefore, the drainage path simplification is justified and the effect of soil extrusion is only significant in the primary elastic compression stage.

The approximate consolidation coefficient for Angolan soil is 0.02 mm^2/s. The c_v value for Congolese clay is 0.006 mm^2/s as shown in Figure 21. Figure 21 illustrates the method described in § 2.1.3 to determine c_v.

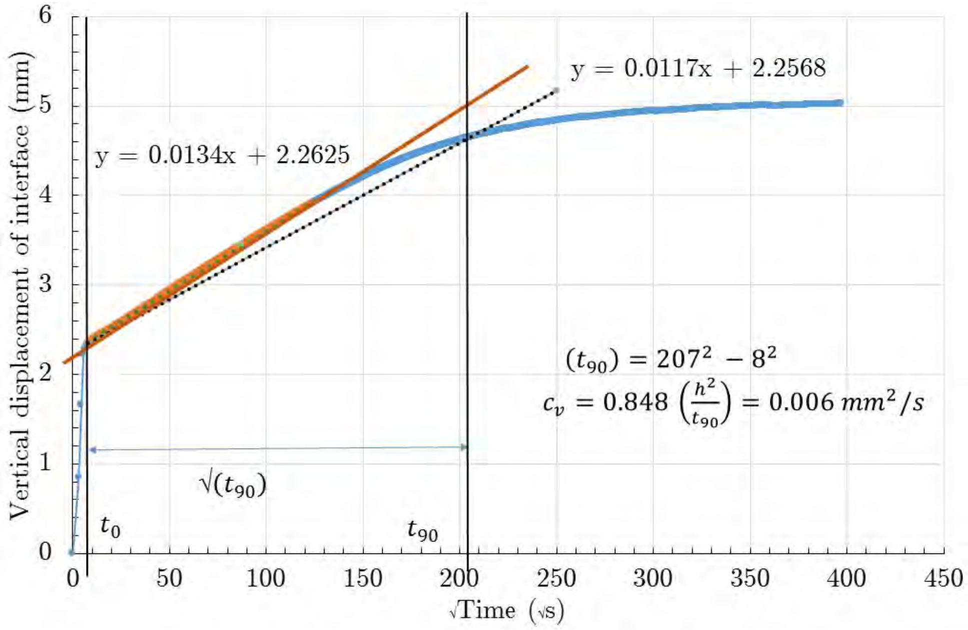


Figure 21: Obtaining the 1-D consolidation coefficient, c_v from test data.

4.2 Slow shearing to determine 'drained' value

Recalling the logarithmic sigmoid function curve in Figure 7, shearing at a rate slower than 0.001 mm/s will generate interface friction in drained conditions. From the graph in Figure 22, these are the reference 'drained' values that will be compared to other tests to gauge the degree of pore-water dissipation within other tests. Two sigmoid curves are presented in Figure 5 to show the effect of different type of soil used for shearing.

Caution should be taken when using the 'drained' values in Figure 22 as a shearing rate of 0.001 mm/s may not be slow enough, thus do not give enough time to allow dissipation of excess PWP. Note that 'C 2' in the graph is sheared at a slower 0.0001 mm/s which should give a better and safer indication of a fully drained shearing value.

More test data is needed to verify if the differences seen in the graph is due to the difference of soil type (different plasticity and water content) or that shearing Angolan clay at 0.001 mm/s is partially drained shearing.

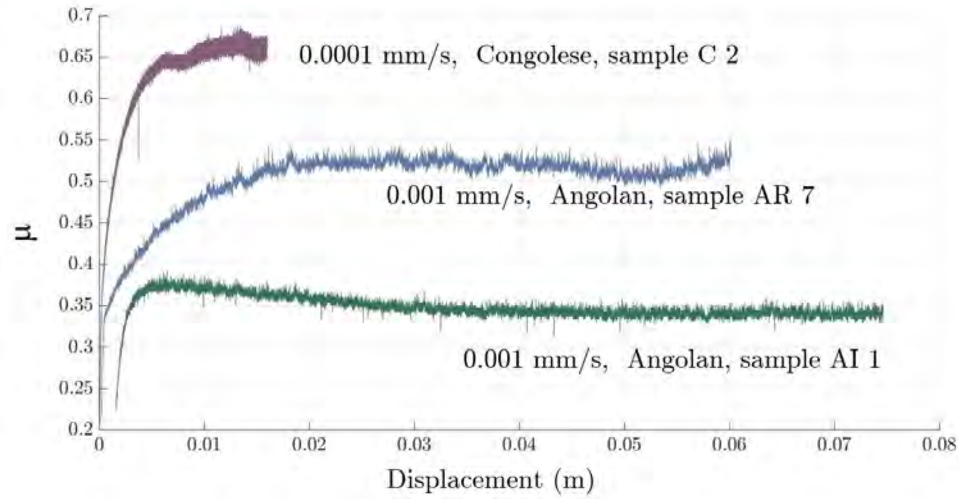


Figure 22: Shearing at low speed. Congolese soil was sheared at 0.0001 mm/s, an order of magnitude slower than the Angolan soil.

4.2.1 Effective Stress

The slow shearing behaviour of the Congolese clay under a total vertical stress of 2 kPa and 4 kPa is similar as those seen in Figure 23. Both samples were sheared at 0.0001 mm/s. Soil sample in C 8 was sheared for more than a month at 0.0001 mm/s, completing almost two full 360°(not shown in graph). The drained friction value remained constant at 0.65, verifying that the results are reliable and that it is fully drained. This behaviour is similar to data results of other soil sample shearing presented in the logarithmic sigmoid function curve in Figure 7.

During slow shearing while applying low vertical stress, the interface friction is a function of friction angle, similar to shearing between sand-sand grains.

Applying such low vertical stress at 2 kPa is at the limit of the Cam-Tor device capabilities. The results in this section verify that 2 kPa shearing is indeed possible on the Cam-Tor.

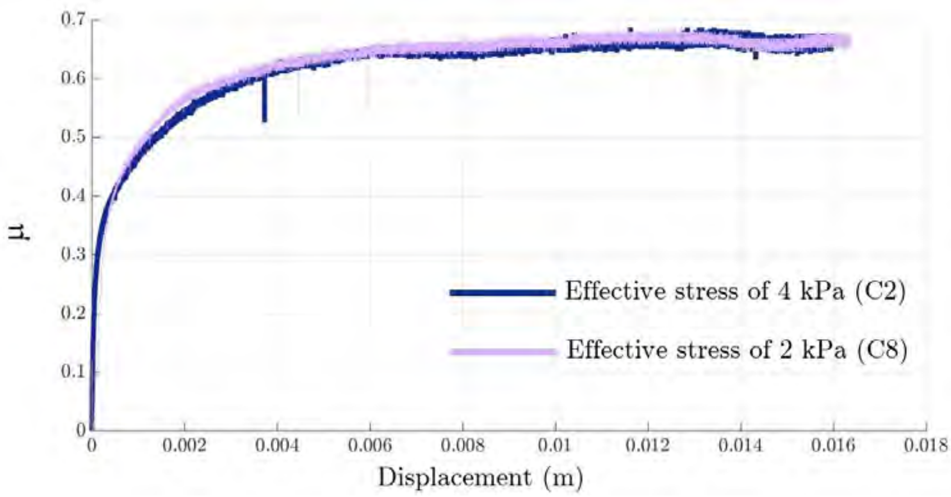


Figure 23: Graph of soil shearing at different effective stresses. Test results from C2 and C8. Test data of C2 is thickened intentionally to illustrate that the drained value is the same as C8.

4.3 Monotonic Shearing Behaviour

4.3.1 Fast shearing

Recalling from the observations Vincent (2014) mentioned in § 2.2, fast shearing tests (assumed to shear in an undrained manner) show these common shear stress features:

1. a distinct peak (identified as μ_{peak}) followed by,
2. a drop to the undrained shear phase (identified as μ_{min}), and
3. a regain of interface friction value as excess pore-water pressure dissipates (identified as μ_{res}).

The interface friction eventually approaches the drained value. In Figure 24, results of Test C 1 shows similar features as the results from Vincent (2014) (plotted in yellow).

Test AR 9 however, did not show this trend. It is suspected that in Test AR 9, the soil element was not sheared for a sufficiently long distance to dissipate excess PWP before it was switched to shear at a slower rate of 0.1 mm/s. Test AR 9 is a three-stage (fast - slow - fast) test designed to investigate the effect of changes in shearing speed.

4.3.2 Discussion

4.3.2.1 Residual values in fast shearing

None of the residual values, μ_{res} recorded during fast shearing in Figure 24 are as high as the 'drained' or partially drained values from Figure 22.

There may be two possibilities to this discrepancy between slow (small displacements) drained shearing and pre-sheared (large displacements) drained shearing.

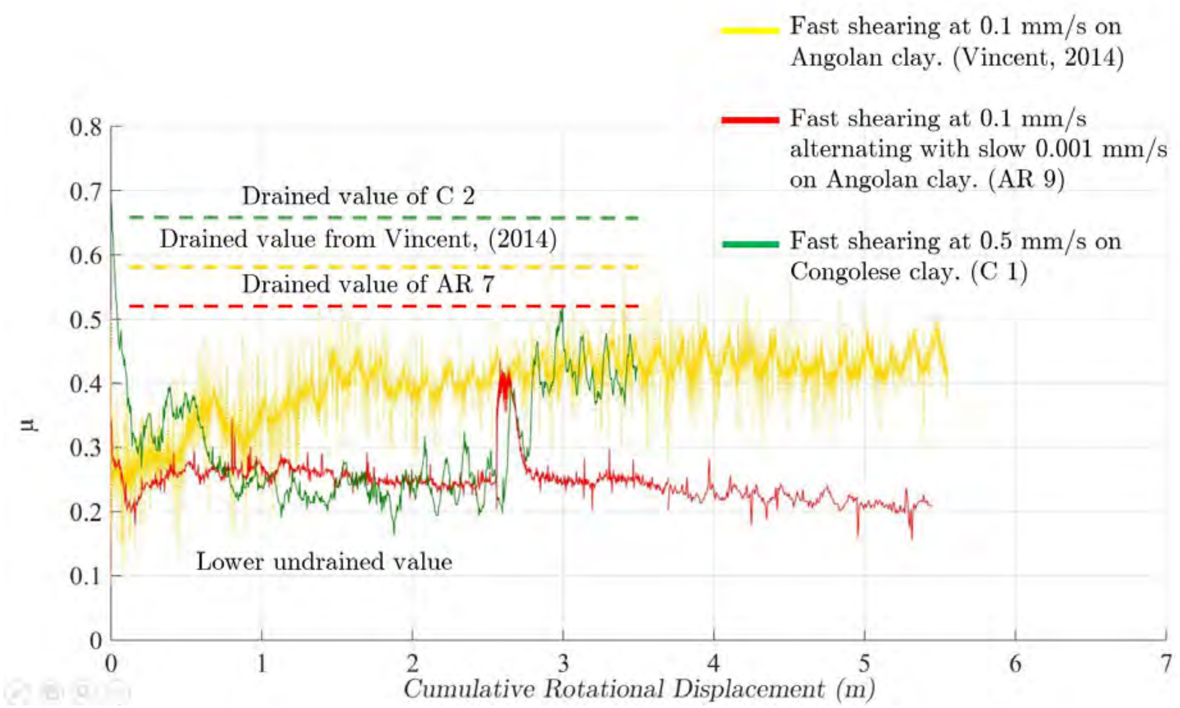


Figure 24: Fast undrained shearing over large displacements. Test results are from AR 9 (red), C1 (green) and Vincent (2014) (yellow).

This may suggest that even though there is an increase in interface friction, the value has yet to reach drained values. From the equations in § 2.1.3 and the c_v values shown in § 4.1, drained conditions should have been attained in Angolan clay samples (AR 9 and Vincent, (2014)'s results) after 1.7 meters, which is equivalent to 4.6 hours of shearing. [8]

If drained conditions were achieved in Vincent (2014)'s test, this discrepancy might be due to the crushing of grains and the repacking of soil structure. However, the question as to whether drained conditions will eventually prevail or have already prevailed can only be verified if pore-water pressure values at the interface are measured.

Soil that is sheared quickly over long distances would have experienced crushing of particles, causing a change in particle size and packing structure. Thus, the interface friction can be lower when the soil reaches drained conditions as excess PWP dissipates. The 1 mm crushing pellet zone (CPZ) observed by Kuo et al. (2015) will have a more homogeneous particle size as larger pellets are ground into much smaller pieces. Compared to the drained shearing in slow shearing where pellets are not broken down, the more uniform particles found in the CPZ after fast shearing do not share the same structure as when the soil-soil particles are more interlocked. A better interlocked and robust soil-soil structure gives higher resistance when sheared against the interface.

This hypothesis is further supported by the higher 'drained' friction value of 0.4 in

[8]The time predicted for drained conditions to prevail for Congolese sample C 1 is approximately 12.7 hours, which would occur after at least 4.7 meters of shearing. This suggests that for the sample in C 1 has not been sheared sufficiently to reach drained conditions.

AR 9 (in red) observed. The value coincides with 'drained' values of Vincent (2014)'s test (Figure 24) and is significantly lower than the slow shearing drained value in AR 7.

In AR 9, the soil particles are sheared and broken into smaller pieces by previous fast shearing within the CPZ. Although drained conditions have not prevailed at the end of fast shearing in Test AR 9, a hint of its drained value can be seen when shearing speed is changed to slow. Soil particles in the CPZ do not provide the same high resistance as compared when sheared at slow rate although both shears at drained conditions.

Other factors that might contribute to the discrepancy is the variability of the soil sample. For example, the initial water content of these soil samples are very different. Further studies are needed to gain more insight into this phenomenon.

4.3.2.2 Peak value

Peak values recorded during Cam-Tor tests are generally of less significance. Due to the nature of strain distribution along the interface radius, the bulk of the soil will not experience peak stress at the same time. Conversely for residual stresses, all the soil at the interface would have been sheared sufficiently to have reached the residual stress. The friction values recorded during shear tests in the Cam-Tor should thus be interpreted as a 'moving average' that smears out peaks that arise radially, i.e. the recorded peak friction value should *not* be interpreted as a peak friction *at* the characteristic radius.

Peak values are only observed for its presence and its relative magnitude between different tests.

4.3.2.3 Rate effect on minimum value

The minimum interface friction coefficient μ_{min} value occurs during the turbulent shearing and crushing phase. This should be a more prominent feature in the shearing of marine clay with crush-able faecal pellets. The μ value in this phase tends to fluctuate more due to the random distribution of pellets in the soil sample.

When shearing at fast rate, pellets that are not glauconised (hardened) are crushed into finer pieces. The collapse of soil structure and the movement of crushed debris will induce positive excess PWP. The effective stress will then decrease, hence depressing the interface friction coefficient.

It is observed that μ_{min} for the faster shearing in Test C 1 is sustained longer before a gain in μ is seen. This is as predicted seen excess PWP takes longer to dissipate at higher shear speed.

4.4 Remoulded vs Intact soil sample

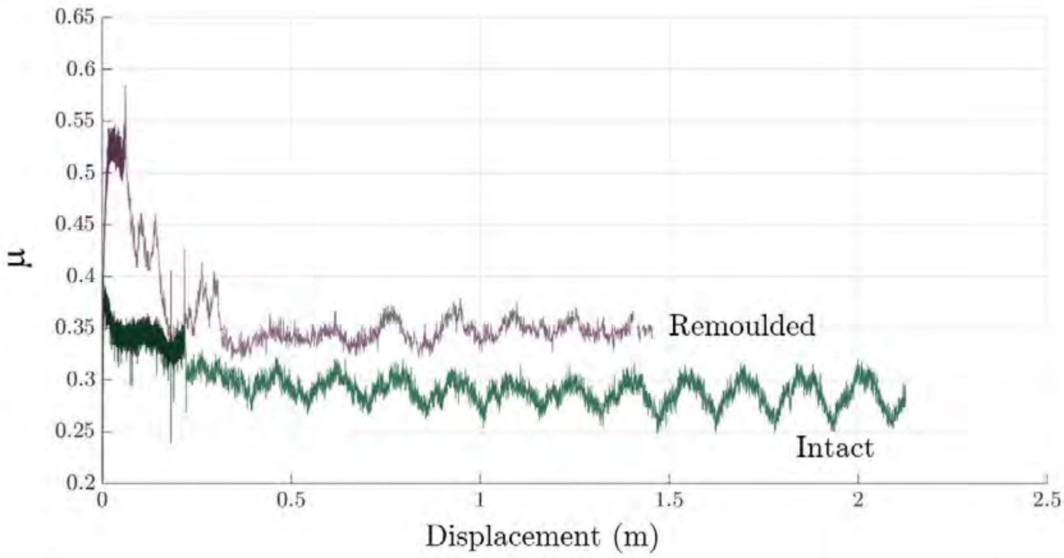


Figure 25: Graph of slow (0.001 mm/s) - fast (0.1 mm/s) - slow shearing transition of remoulded and intact soil sample. Test results from AI 1 and AR 7.

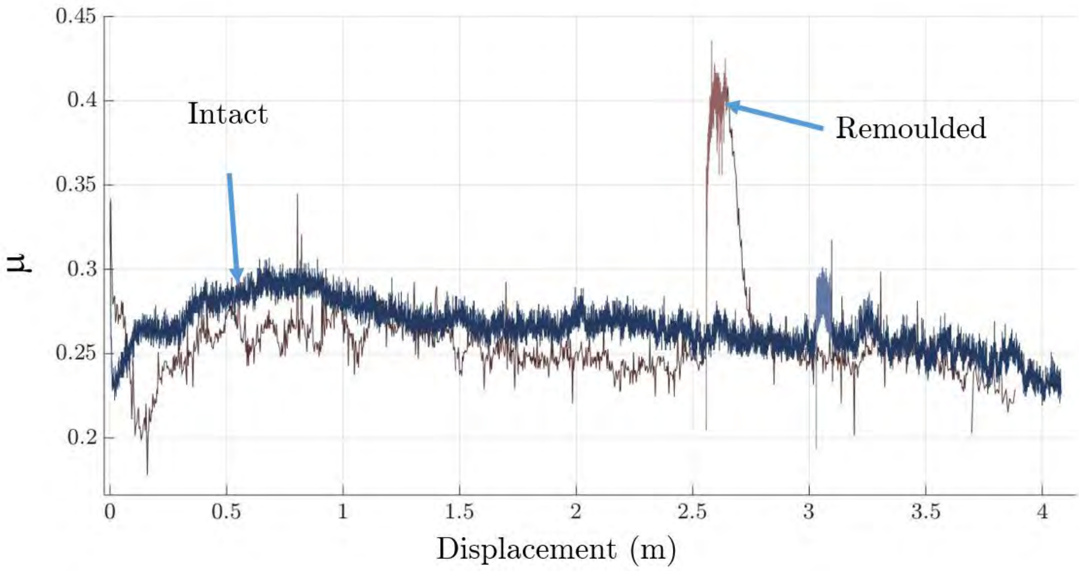


Figure 26: Graph of fast (0.1 mm/s) to slow (0.001 mm/s) shearing transition of remoulded and intact soil sample. Test results from AI 5 and AR 9.

In this section, the shearing behaviour between remoulded and natural (intact) soil sample are compared and contrasted. The first comparison is done between tests where soil has been sheared initially at 0.001 mm/s before shearing at 0.1 mm/s. The second is to compare between tests where soil have been sheared at 0.1 mm/s, followed by 0.001 mm/s and finally back at 0.1 mm/s again.

It is observed that the rate effect is more prominent in remoulded samples for both cases. There is a bigger difference between the shearing resistance between fast and slow shearing seen in remoulded sample.

This suggests that in intact soil sample, crushing occurred even at slow shearing, generating large positive excess PWP, hence decreasing the effective vertical stress and ultimately, the interface friction coefficient. There are less crush-able pellets in remoulded sample, as pellets that survived the remoulding process tend to be the harder glauconised pellets. The remoulded sample shears at the drained value during slow shearing.

4.5 Cyclic Shearing Behaviour

Data collected from cyclic shearing tests are commonly plotted on a μ vs cumulative distance (m) graph or on a μ vs stage number graph. The interface friction coefficient plotted on the graph of μ vs stage number is taken as the mean value of the particular stage. This value is quoted as μ_{mean}.

Unless stated otherwise, all cyclic shearing tests have 30 shearing stages with a 30-minute re-consolidation period where shearing is paused. Also unless stated otherwise, cyclic shearing is at a rate of 0.5 mm/s.

4.5.1 Effect of pauses

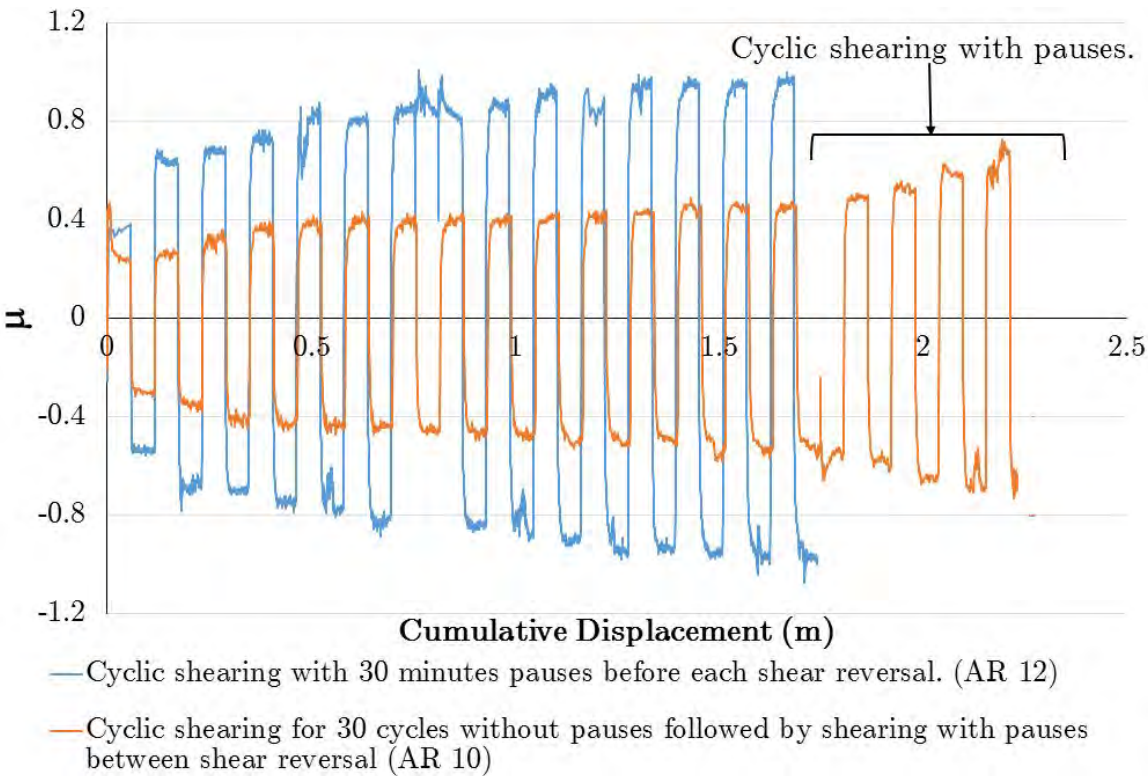


Figure 27: Comparison between cyclic shearing with pauses and without pauses between each shearing stage. Test results from AR 10 and AR 12.

Cyclic shearing in pipelines usually have pauses between each shearing stage. A test without pauses was conducted for comparison. In this Test AR 10, after 30 cycles of shearing without pauses between each stage, cyclic shearing is continued for another 8 cycles, this time with pauses between them.

The μ_{mean} of the cyclic shearing tests *without* pauses were approximately 0.5 the value of the μ_{mean} of the cyclic shearing with pauses. Also, the increment of μ_{mean} between each stage is insignificant for cyclic tests conducted without pauses. Note in Figure 27 that when pauses are introduced after 30 cycles, there is a significant strength gain after each pause.

4.5.2 Cyclic vs Monotonic

The same graph in Figure 27 is plotted again in Figure 28, this time to make comparison between cyclic shearing, 'drained', and undrained friction values of monotonic shearing. In this graph, we can observe that interface friction during cyclic shearing without pauses is similar to the undrained value of monotonic shearing. As pauses are introduced, the interface friction increases greatly, surpassing the drained value of monotonic shearing.

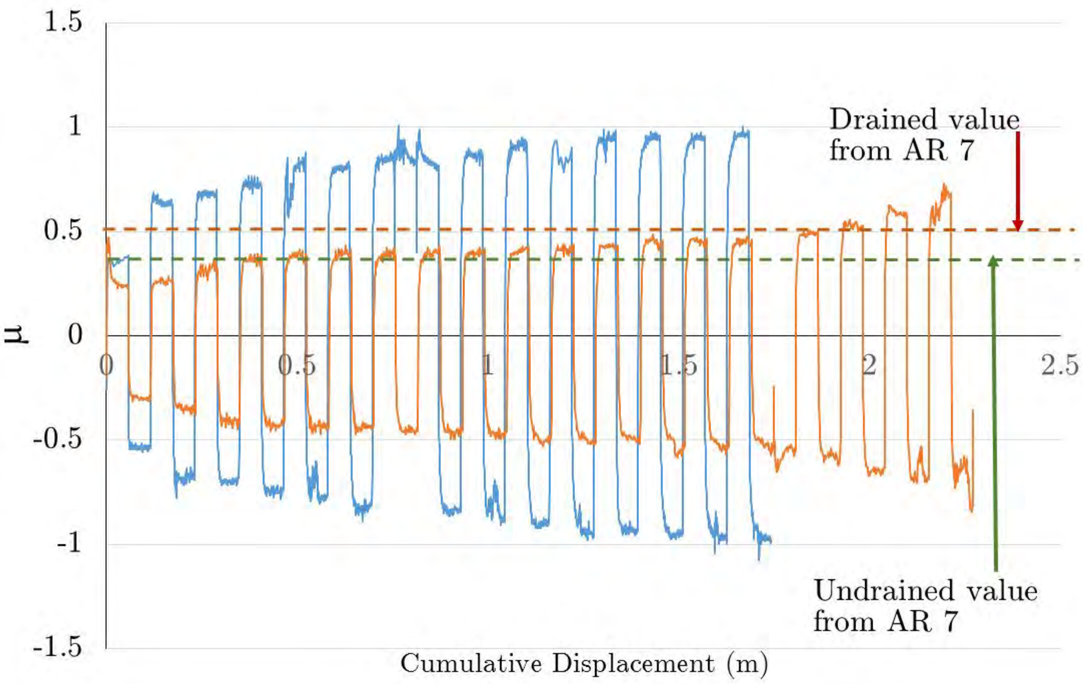


Figure 28: Graph is similar to Figure 27. Drained and undrained values are added to compare results from monotonic shearing.

4.5.2.1 Discussion

Recalling the critical state soil mechanics model mentioned in § 2.1.5, the $\mu_{residual}$ at the end of the cycles is predicted to be similar to the drained μ attained in slow monotonic ‘drained’ shearing, as drained condition is said to have prevailed. This is shown in Figure 29.

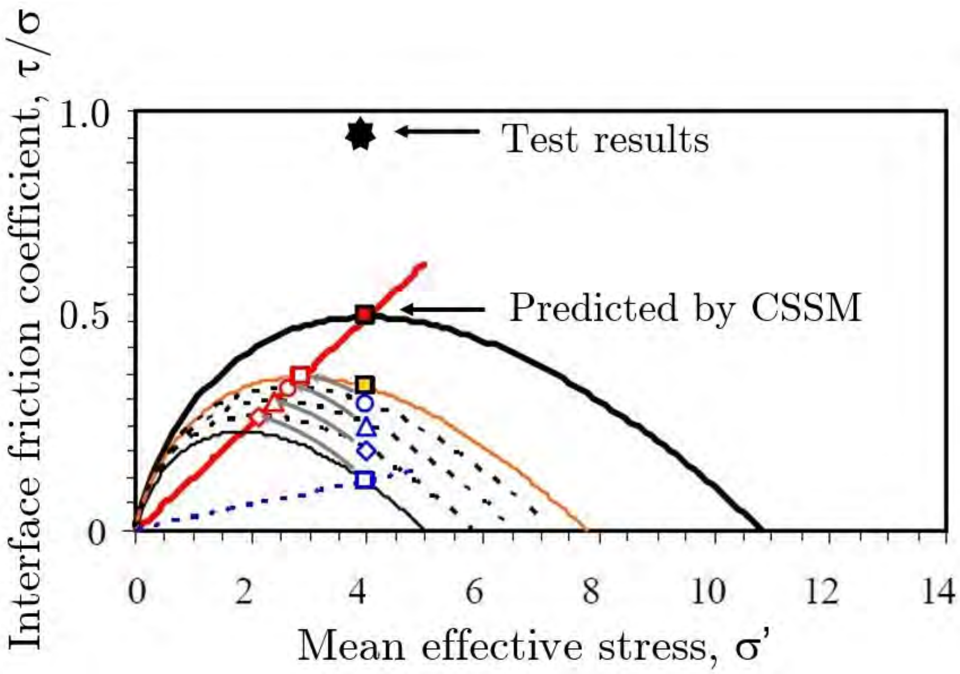


Figure 29: Comparing test results to the predicted drained value based on CSSM. Modified after Deeks et al. (2014).

This highlights the importance of understanding the micromechanics of large displacement interface shearing in explaining the higher residual interface friction coefficient seen at the end of cyclic shearing and periodic pauses. There are a few aspects to consider when trying to explain the strengthening behaviour induced by episodes of shear reversals and pauses.

1. From Figure 28, it can be observed that shear reversal does not play a significant role in influencing the interface friction coefficient. The pauses on the other hand show a great influence on the interface friction coefficient.

2. Large strains of the soil at the interface relative to the layer of soil below it will cause weakening of the soil layer as some interlocked soil grains roll along with the movement of the interface. This is observed in the PIV analysis shown in Figure 30. If the soil is allowed to re-consolidate after each cycle, there will be a strengthening effect of this weakened zone.

3. The pauses also allow repacking of the soil structure, allowing any unstable soil particles to fall into any voids generated during shearing. The vertical stress applied is not great enough to crush any particles, but at the same time, it allows the soil particles to find voids to ‘bury itself’. This protects the particles from being in a

vulnerable position susceptible to shear crushing. This also increases interlocking between soil-soil particles.

4. Each time when the interface starts to shear again after pausing, the interface will need to bulldoze through a better 'reinforced' soil structure. In the case of monotonic slow shearing, the drained interface friction coefficient is a function of the angle of friction between intact soil material and interface. For cyclic shearing (with pauses), the interface friction coefficient is a function of an increasing angle of friction as the soil packing structure is periodically modified after long cumulative displacement shearing.

5. The hypothesised mechanism should only be valid for cases where the soil contains crush-able particles, like the faecal pellets found in West African Clay.

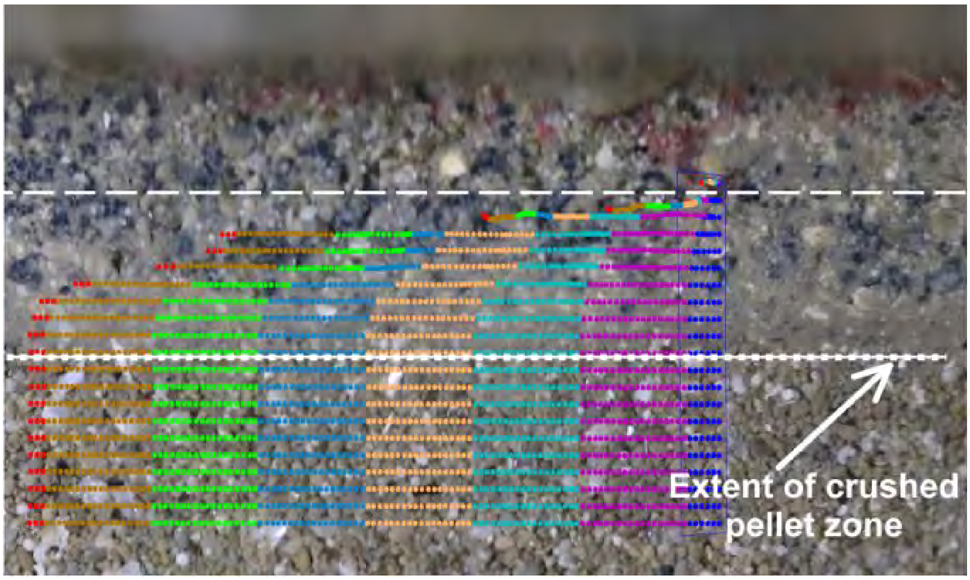


Figure 30: PIV analysis showing the relative strain between soil-soil at the top of the crushing pellet zone (CPZ). Source: Kuo et al. (2015).

4.5.3 Rate effect

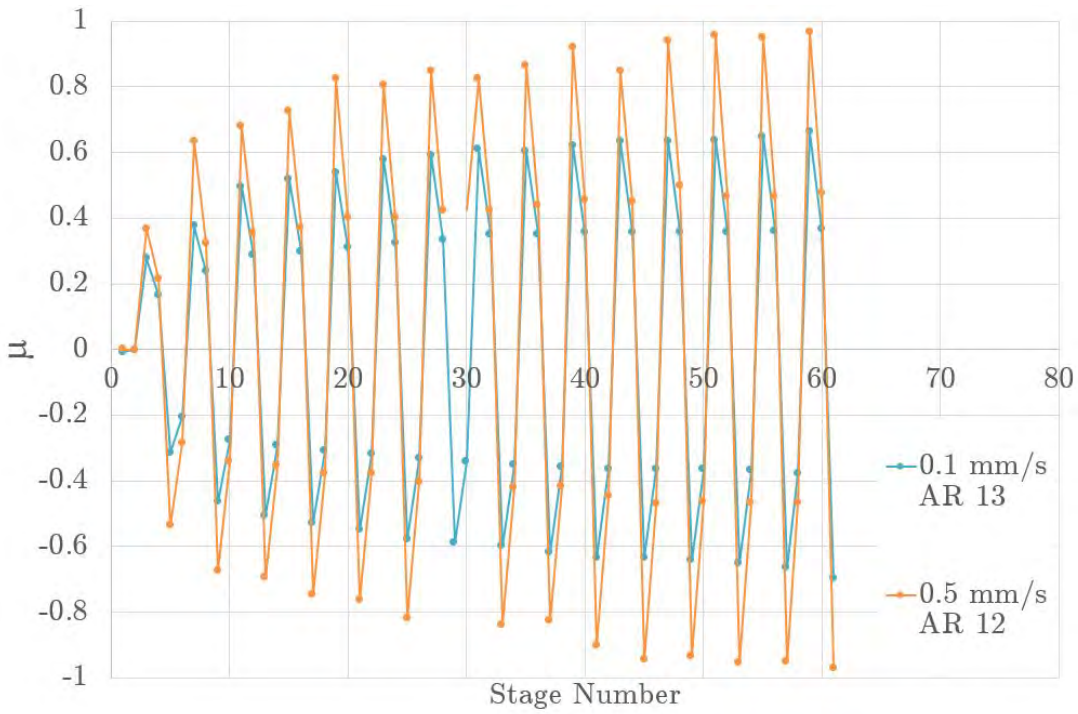


Figure 31: Graph showing effect of difference in shearing rate: 0.5 mm/s and 0.1 mm/s. Test results from AR 12 and AR 13.

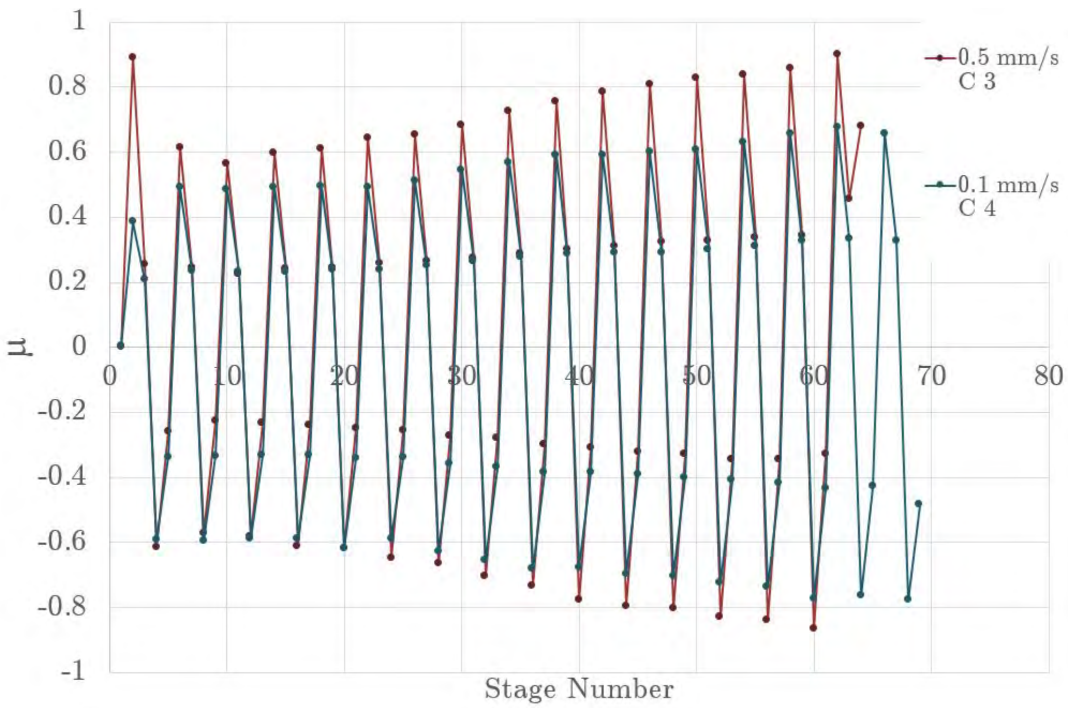


Figure 32: Graph showing effect of difference in shearing rate: 0.5 mm/s and 0.1 mm/s. Test results from C3 and C4.

From the results in Figures 31 and 32, μ for shearing at 0.1 mm/s is consistently lower than 0.5 mm/s. If the crushing and repacking action is the reason for the higher drained value seen in cyclic shearing when compared against the drained value in monotonic slow shearing, the results in Figures 31 and 31 could be explained with the same hypothesis.

It is observed that the interface friction during the faster shearing at 0.5 mm/s is higher than the interface friction during the slightly slower shearing at 0.1 mm/s. Based on the backbone curve presented in Figure 5, it is seen that if 0.1 mm/s is fast enough for fully undrained shearing to occur, shearing at 0.5 mm/s should be fully undrained as well. The interface friction coefficient for both cases should have then been similar. This is not what we see in this test comparison.

4.5.4 Effect of OCR

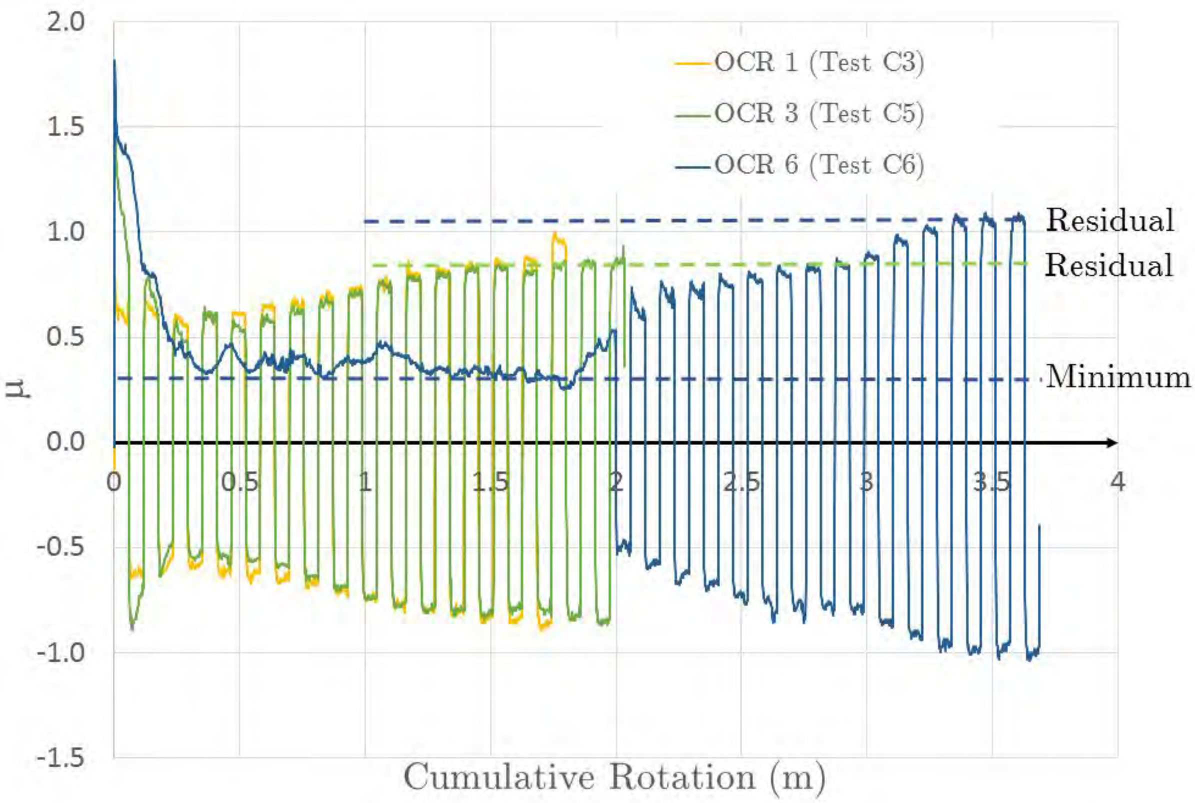


Figure 33: Graph of shearing Congolese soil at different OCR. Test results from C3, C5 and C6.

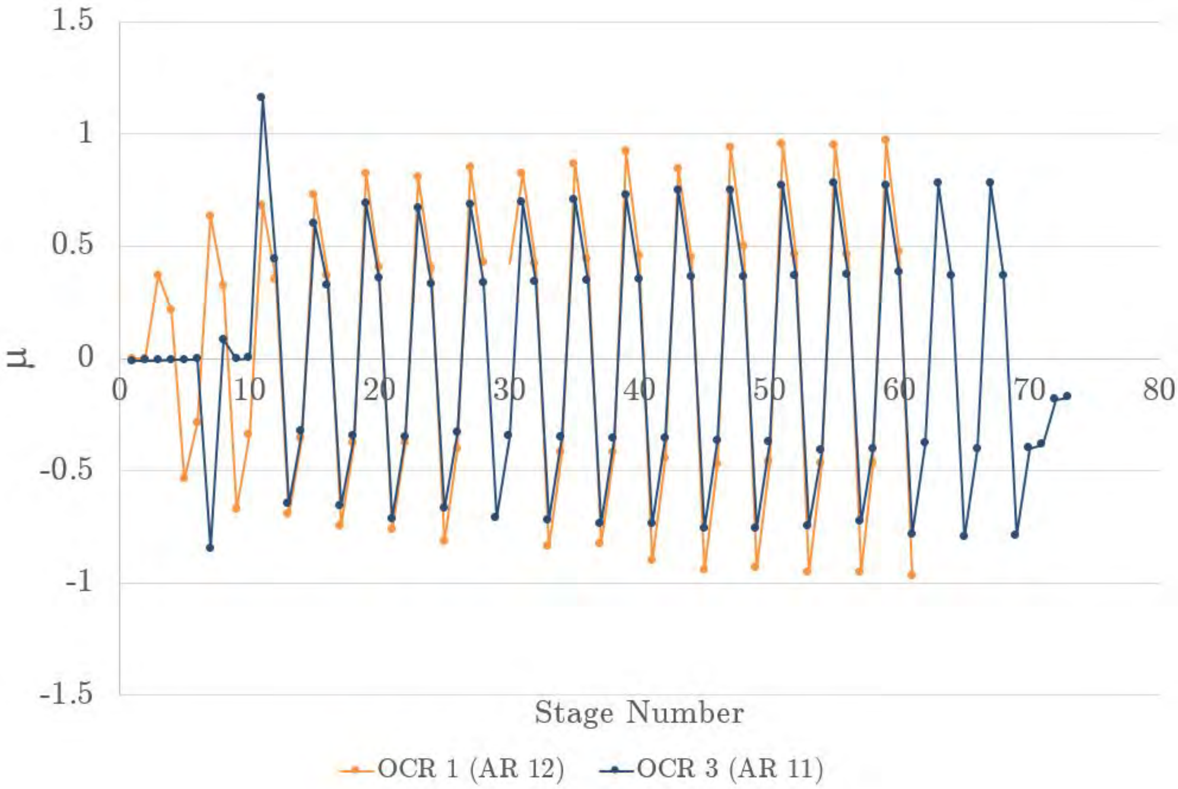


Figure 34: Graph of shearing Angolan soil at different OCR. Test results from AR 11 and AR 12.

The effect of over-consolidation is studied by carrying out 0.5 mm/s fast shearing on soil with OCR of 3 and 6.

Based on the critical state soil mechanics model, normally consolidated (OCR = 1) soil is located on the 'wet' side of the critical state line, whilst soil with OCR = 6 is located on the 'dry' side. OCR = 3 is located near the intersecting point between the yield surface and the CSL where $\sigma'_v \approx \frac{\sigma'_c}{2.7}$. Although soil at OCR = 3 is located at 'dry' side of the CSSM model, it can behave either in a dilative or contractive manner. From the displacement data collected during tests, it is noticed that the shearing was slightly dilative. After shearing for two cycles, the soil sample contracts. The flaw with the settlement data is that soil at the outer radius are extruded during shearing, hence, it is difficult to judge the real settlement.

From Figures 33 and 34, it is observed that apart from the peak value, the interface friction against soil of OCR 1 is slightly higher than OCR 3. The difference between soil of OCR 1 and OCR 3 is seen to be more obvious in Figure 34 where Angolan clay is used instead of Congolese soil.

Based on the data, and assuming that a higher percentage of pellets do give lower interface friction when shearing soil of OCR 3, it can be understood that if the soil is sufficiently dense and well packed, there are no voids for the crushed materials to fill into. The existing vertical stress will also not be high enough to assist crushed debris to interlock well with the soil particles below. This will cause rolling of the particles in the crushed pellet zone, thus interface friction coefficient will be lower if there is a thicker zone of crushed debris between the lower intact soil and the interface. This is just a possible scenario that could explain why soil of OCR 3 has interface friction lesser than OCR 1.

More data is needed to show if there is a reliable trend between the proportion of pellets in the soil sample and its effect on the shearing behaviour of over-consolidated soil. Imaging of the shearing behaviour should give the best illustration of the shearing mechanism.

As mentioned in § 2.1.5, work hardening is expected for shearing of soil situated on the 'dry' side of the CSL. In Figure 33, shearing of soil of OCR 6 shows evident cyclic hardening behaviour. Further tests should be carried out for varying OCR values (including OCR 3) to understand how over-consolidation on the 'dry' side affects interface friction. To investigate the influence of pauses on cyclic hardening, tests with and without pauses on 'dry' side soil should also be carried out.

In Test C6, the soil of OCR 6 is initially allowed to shear in a monotonic manner before shearing cyclically. This is done in the interest of finding out if soil sheared at OCR 6 will also show the three key features, i.e. μ_{peak}, μ_{min} and μ_{res}.

4.5.5 Interface Roughness

4.5.5.1 Effect of interface roughness

It is mentioned previously that the interface friction coefficient do not always have positive correlation with the roughness of the interface. Rough interfaces can sometime give a counter-intuitively lower μ_{res} relative to those from smoother interfaces.

A series of fast tests at 0.5 mm/s with three interfaces of different roughness is conducted. The roughest interface has an average roughness of $R_a = 32\mu m$, followed by the second at $R_a = 6\mu m$ and the smooth at $R_a = 0.3\mu m$.

This experiment showed that $\mu_{smooth} < \mu_{roughest} < \mu_{moderate}$ (Figure 35). This result shows a clear behaviour that rougher interfaces do provide better interlocking with soil particles, resulting in higher interface friction values to a certain extent. If the interface roughness is much greater than the soil particles, the soil particles may comfortably be jammed in the big troughs of the surface profile. The soil particles will then not 'see' the rough surface profile of the interface and instead slide between soil particles stuck in between the 'teeth' of the interface and the soil particles slightly beyond the interface.

In general terms, for higher interface efficiency between interface and soil particles, compatibility between interface surface roughness and soil particle size needs to be achieved.

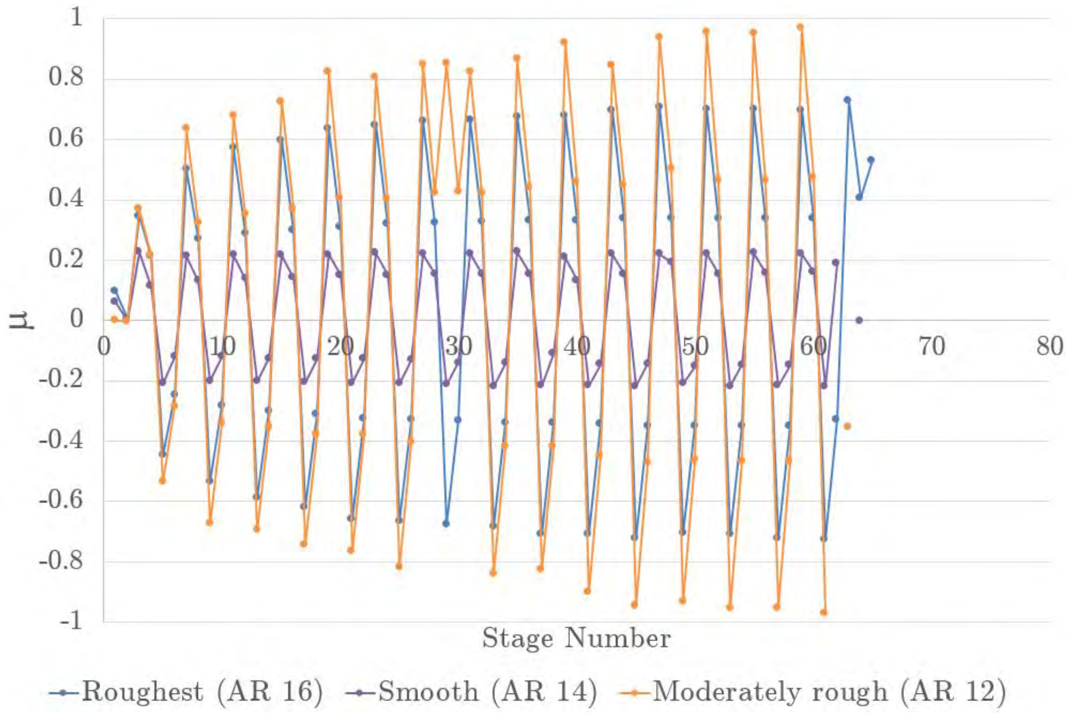


Figure 35: Graph of shearing soil against interfaces of different roughness. Shearing rate of the soils is 0.5 mm/s. Test results from AR 12, AR 14 and AR 16.

4.5.5.2 Smooth interface

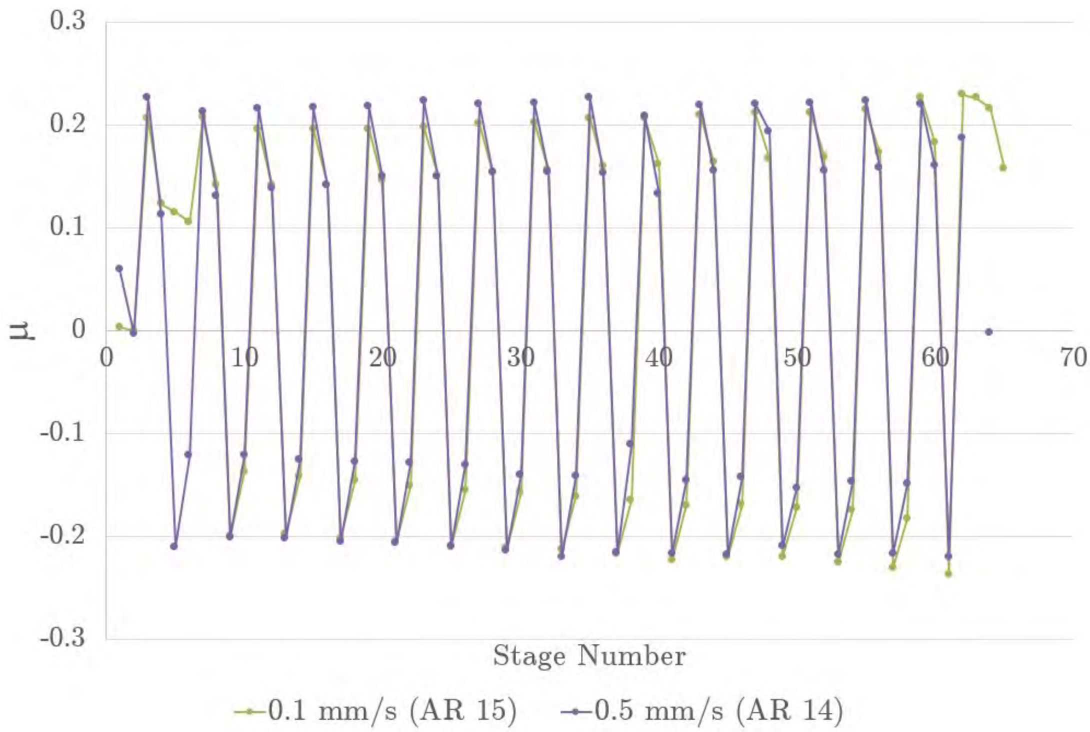


Figure 36: Graph of shearing soil against interface of smooth interface at different shearing rate. Test results from AR 14 and AR 15.

From the graph in Figure 36, it is observed that there is no rate effect when soil is sheared against a smooth interface at different speeds. No crushing is expected during shearing against a smooth interface hence the speed of the interface shearing does not influence smooth interface soil shearing. Sliding between interface and the soil grains is predicted to have happened and the interface friction value may be the characteristic sliding friction.

5 Installation of the Pore-water Pressure Transducer

The micromechanics of shearing can be more clearly visualised if the change of pore-water pressure PWP is known.

For example, if the PWP is known during tests of cyclic shearing with pauses, we can find out if the soil structure hardens throughout the shearing process or if hardening occurs only after dissipation of excess pore-water pressure (shearing becomes drained). The question as to whether fast shearing will always continuously generate PWP and never achieve drained state can be answered.

The author has undertaken necessary modification works on the set-up of the Cam-Tor apparatus especially in fabricating an interface that holds a pore-water pressure transducers (PPT) in place. The PPT is embedded into the interface and is placed right behind a porous plastic interface as shown in Figure 37. The PPT is located at the characteristic radius of the interface. The PPT used is of very low range, rated at 350 mbar. During tests, it was found that the PWP readings were severely dampened from what was expected by almost an order of magnitude. This may be due to the porous plastic used as the interface that diminishes the transmission of PWP.

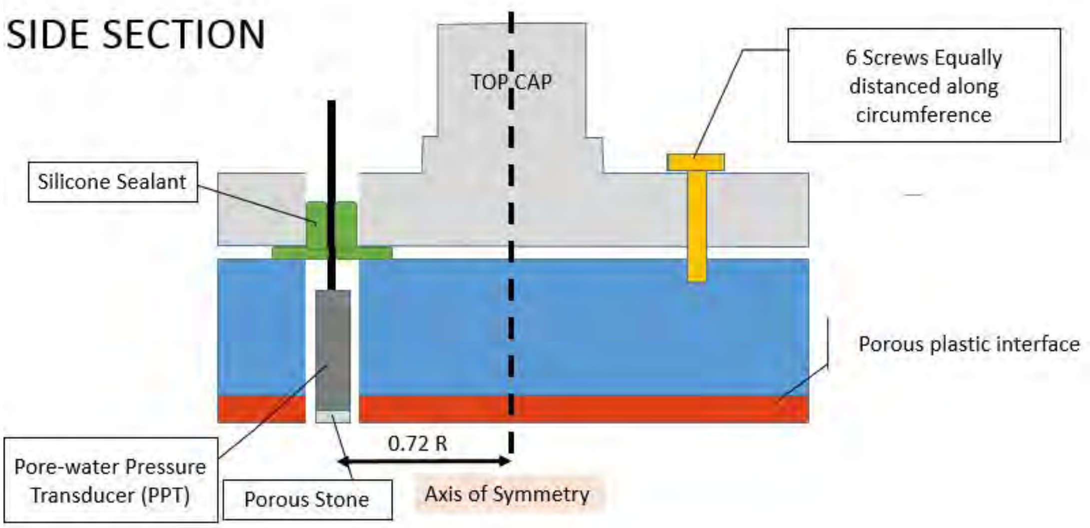


Figure 37: Section view of the embedment of the pore pressure transducer in the interface.

Installation of PPT was also carried out in the macroscale interface direct shear test device as reported in Eid et al. (2014). It was commented that the porous stone that surrounds the PPT can affect the measured excess pore-water pressure. Due to such complications, the PPT was used only to verify whether excess PWP has been fully dissipated at the end of the test, especially tests with large shear displacements.

Future work is recommended where other porous material are used as the interface.

6 Conclusion

The purpose of this project is to better understand the factors that influence the axial resistance of the pipeline. Pipelines shear:

1. over large displacements, and
2. in a cyclic manner with pauses before shear reversals.

Interpreting the results from Cam-Tor tests conducted on West African Clays, the following conclusions are drawn from this study:

- The coefficient of consolidation determined from the consolidation curves are found to be consistent across tests and agrees well with literature. This justifies the drainage path assumption in Cam-Tor and shows that the effect of soil extrusion is only significant during the primary elastic compression stage.
- Shearing at 0.001 mm/s may not be slow enough for excess PWP to dissipate. This is inferred from the residual value of shearing Angolan clay at 0.001 mm/s being much lower then when shearing Congolese clay. The limitation of this comparison is that there is a factor of soil variability. This statement was nevertheless put forward as a caution for any future comparison with what might be a partially drained value.
- There is no difference in interface friction observed for soil samples at two different applied vertical stress (2 kPa and 4 kPa) when the samples are sheared in 'drained' mode (slow shearing). This suggests that friction angle is the main determinant of μ when soil is sheared slowly.
- The drained μ of slow drained shear tests and the eventual 'drained' μ of large displacements fast shearing do not agree well. It is hypothesised that this is due to the change of inherent friction angle between the grains due to crushing in fast shearing. Consideration to whether drained conditions will eventually prevail should be taken as this cannot be confirmed until the pore-water pressure of the 'drained' state is measured.
- It is believed that shearing natural soil sample even at low shearing speed crushes the weak pellets and generates excess pore water pressure. There is not much difference

between slow and fast shearing of intact soil sample and this suggest excess PWP is not allowed to dissipate in both cases.

- Shear reversal does not play a significant role in influencing the interface friction coefficient. The introduction pauses on the other hand show great influence. It is hypothesised that the pauses allow repacking of the soil structure and strengthening of high soil-soil strain zones.

- Normally consolidated soil is seen to have higher μ_{res} than soil with over-consolidation ratio (OCR) of 3, while soil with over-consolidation ratio (OCR) of 6 displays the greatest strength hardening effect.

$$\mu_{OCR\ 3} < \mu_{OCR\ 1} < \mu_{OCR\ 6} \tag{8}$$

  PIV analysis will be very helpful in explaining the possible mechanism that occur during shearing at different initial packing of grains.

- The comparison of tests using different interface roughness show that $\mu_{smooth} < \mu_{roughest} < \mu_{moderate}$. It is counter-intuitive for $\mu_{roughest}$ to be lower that $\mu_{moderate}$. Due to the much bigger troughs in roughest interface surface relative to soil grains, the particles tend to fill in and clog up the troughs. The interface is now seen as a 'smoother' interface and there is lesser interlocking between the 'teeth' of the interface surface profile with the soil particles. This is not proven yet and PIV imaging or further testing needs to be done to verify.

- When sheared against a smooth interface ($R_a = 0.3\mu m$), the rate of shearing does not influence interface friction. The interface friction in this case is predicted to be the characteristic sliding friction.

- The pore-water pressure induced during shearing is the missing puzzle to solving the questions to which shearing mechanism occurred during shearing. The installation of a working pore-water pressure transducer is crucial to understanding this. Although this attempt was not successful, future development on this issue is highly recommended.

7 Recommendations for Future Work

This section highlights the areas in which future work can focus on.

- There is a need for PIV analysis of cyclic shearing to better understand the underlying mechanism of strength hardening of soils with different over-consolidation ratios. PIV analysis of extremely rough interface shearing and smooth interface shearing can be carried out to verify if the proposed sliding mechanism occurs.

- Further tests using the Cam-Tor at different speeds, especially more than 0.1 mm/s, to plot a backbone curve for West African clay will be useful. This is to allow a better identification of the boundaries between drained, transitional, and undrained shear behaviour. Moreover, it will be helpful to identify the rate at which viscous effects will become prominent.

- The characteristic radius, 0.72 R used for analyses in this project is determined based on theories. Verification if 0.72 R is indeed the characteristic radius is important to allow more accurate comparisons against data from other shearing device.

- The limitation of the Cam-Tor is its inability to record peaks during shearing as there is strain distribution in the radial direction. Further development and modifications done to record only the torque contributed by an isolated ring of soil at the characteristic radius will be very useful.

- Continue developing methods for the measurement of pore-water pressure.

8 Appendix

8.1 Appendix A: Test programme

Test	CASE					
	SPEED (mm/s)	NO. OF CYCLES	ROUGHNESS - RA	OCR	σ'	SOIL SAMPLE
AI1	0.001 - 0.1 - 0.001	-	5.6	1	4	Angolan - Natural
AI2	0.001 - 0.1 - 0.001	-	0.3	1	4	Angolan - Natural
AI5	0.1 - 0.001 - 0.1	-	5.6	1	4	Angolan - Natural
AR7	0.001 - 0.1 - 0.001	-	5.6	1	4	Angolan - Remoulded
AR9	0.1 - 0.001 - 0.01	-	5.6	1	4	Angolan - Remoulded
AR10	0.5	30 Cyclic w/o pause	5.6	1	4	Angolan - Remoulded
AR11	0.5	30	5.6	3	4	Angolan - Remoulded
AR12	0.5	30	5.6	1	4	Angolan - Remoulded
AR13	0.1	30	5.6	1	4	Angolan - Remoulded
AR14	0.5	30	0.3	1	4	Angolan - Remoulded
AR15	0.1	30	0.3	1	4	Angolan - Remoulded
AR16	0.5	30	64	1	4	Angolan - Remoulded
C1	0.5	-	7.4	1	4	Moho Nord - Remoulded
C2	0.0001	-	7.4	1	4	Moho Nord - Remoulded
C3	0.5	30	7.4	1	4	Moho Nord - Remoulded
C4	0.1	30	7.4	1	4	Moho Nord - Remoulded
C5	0.5	30	7.4	3	4	Moho Nord - Remoulded
C6	0.5	30	7.4	6	4	Moho Nord - Remoulded
C8	0.0001	-	7.4	1	2	Moho Nord - Remoulded

Figure 38: Parameters used for each test.

8.2 Appendix B: Risk assessment review

A risk assessment is conducted at the start of the project. The aim of the assessment is to identify hazards and take precautionary measures to reduce the chances of the hazards materialising.

When setting up apparatus, proper manual handling habits was exercised when moving the heavy pressure cell. When doing soldering work, a good distance was maintained between eyes and soldering action. A magnifying tool was used if necessary.

The laboratory desk was cleared from time to time and any wiring trip hazards from computers are tucked away and taped.

It is important that hygiene is maintained before and after conducting tests. This includes thorough washing of hands after coming in contact with soil. Any open cuts was protected and remained sterile from any contact with soil sample. Disposable gloves were used in this occasion.

During the use of computer to carry out data organising and analysing, correct posture was adopted and regularly breaks were taken to avoid injuries from bad posture and eye strain.

By the end of the project, none of the hazards were encountered.

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