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Chalk–steel interface testing for marine energy foundations

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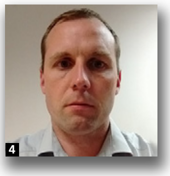

To aid deployment and recovery of tidal stream generators, gravity-based foundations rather than fixed-foundation alternatives are being considered in areas where the foundation may be placed directly onto an exposed rock seabed. Horizontal loading is usually critical in such applications, therefore specific knowledge of the interface friction between the foundation (made of steel or concrete) and seabed is important for design. This paper presents results of an interface testing programme of chalk–steel interfaces carried out utilising a computer-controlled interface shear tester under constant normal stress conditions against steel of different roughness. Results indicate that interface strength is significantly affected by the normal stress applied, as interface strength degrades for normal stress levels in excess of 30% of the chalk's tensile strength (~300 kPa). Large-displacement tests revealed a tendency of the ultimate interface frictional resistance to drop to values very similar to that of the basic chalk–chalk interface at normal stresses up to 300 kPa, whereas substantial additional degradation was noticed for normal stresses above 700 kPa. At low normal stresses and displacements the behaviour of the chalk–steel interface was captured by an alpha type approach related to the rock unconfined compressive strength, which has been developed for other higher strength rock types.

Notation

b	fitting constant
c	fitting constant
D_{50}	mean soil particle size
d	linear displacement
G_s	specific gravity
m_{sat}	saturated moisture content
n	porosity
R	radial position
R_a	average centre-line roughness
r	rock sample radius
T	torque
T_0	tensile strength
α	adhesion factor
δ_{peak}	peak interface friction angle
δ_{ult}	ultimate interface friction angle
θ	rotational displacement
μ	coefficient of friction
ρ_d	dry density
σ_v	normal stress
τ	shear stress
ϕ_b	basic friction angle

1. Introduction

To allow the design of gravity base foundation systems (GBS) for tidal stream generators it is necessary to have appropriate interface friction parameters for the foundation–seabed interface due to the potential for relatively high lateral loads experienced by these applications. As there are high-velocity currents at locations with high tidal stream energy potential, the seabed sediment is likely to be washed out (scoured) resulting in exposed bedrock at the seabed, which may form one half of the foundation surface (Small *et al.*, 2014). The determination of interface friction parameters requires laboratory interface testing using the foundation–seabed materials (typically steel or concrete in contact with different rock types) with testing undertaken at appropriate normal stress levels. Currently there is little guidance available to designers on how such parameters should be selected or the appropriate approaches to laboratory testing (Small *et al.*, 2014). What guidance is available is considered conservative based upon simple assumptions of interface behaviour rather than actual laboratory element testing. For example, Fraenkel (2002) estimated that at least 1500 t of GBS per MW are needed in order to maintain stability, but this value seems very high and conservative where there is significant drive to bring down the foundation costs for marine renewable devices (Carbon Trust, 2011). NAVFAC (1986) suggest a coefficient of friction, μ, of 0·7 (interface friction angle, $\delta = 35°$) for mass concrete on clean, sound rock but the origins of this value are unclear and it is not stated if this refers to a bonded or unbonded surface, or the types of rock that were used in its determination.

Existing guidance on rock–concrete and rock–steel interfaces typically relates to very different applications (and interface conditions) such as concrete bonded or cast against rock at high vertical stress levels. Examples of these are found at the interface between the base of a dam or cast in situ pile rock sockets (Horvath, 1978; Rosenberg and Journeaux, 1976; Williams and Pells, 1981) and rock–steel interfaces such as rock bolts (Li and Håkansson, 1999) or H-steel piles driven into rock (Yu *et al.*, 2013). These rock–steel interface examples result in constant normal stiffness (CNS) conditions, which lead to high normal stresses where the interface is subject to shear and constraint of dilation. This can result in normal stresses at the interface that are orders of magnitude higher than those that would be experienced for a tidal stream generator foundation (estimated at some hundreds of kPa, under constant normal stress conditions). This fact along with the potential for concrete bonding between interfaces may limit the relevance of previous interface studies.

Many systematic efforts to investigate the interface shear behaviour between soil and geotechnical structures (rather than rock) have been made. Potyondy (1961) conducted tests between various soil types (e.g. sand, clay and silt) and concrete, steel and wood, while Peterson *et al.* (1976) conducted tests with sand–steel interfaces. Both studies were carried out using the direct shearbox. In the 1980s significant research was undertaken on sand–steel interfaces by Uesugi and Kishida (1986a, 1986b) and Kishida and Uesugi (1987) utilising the simple shear apparatus. Since these earlier studies research has been continuously undertaken using various devices and laboratory equipment such as the ring shear and curved shearbox (Iscimen and Frost, 2010). Typically it has been found that interface frictional resistance increases as the solid (or the surface representing the structural element) surface roughness increases up to a specific limit defined as critical roughness. At this limit, which approaches the internal friction angle of the sand, a sand–sand slip occurs as a soil–soil shear rather than interface shear predominates. This is because as the surface roughness tends to the diameter of individual sand particles, the sand effectively interlocks with the foundation surface forcing the shearing away from the interface and into the soil mass (Kishida and Uesugi, 1987; Peterson *et al.*, 1976; Uesugi and Kishida, 1986a). It has also been shown that as the roughness increases, more displacement is required to mobilise the peak friction angle (Iscimen and Frost, 2010). Where the soil particle diameter exceeds the surface roughness, the constant volume interface friction angle decreases significantly with increasing D_{50}, with a cut-off point of $R_a/D_{50} = 0{\cdot}015$ for sand (Jardine *et al.*, 1993). For sand–steel interface used in offshore driven piles tan δ_{cv} is often limited to 0·55 (28·8°) (Lehane *et al.*, 2005). Although there have been significant previous studies for soil sheared against common civil engineering materials, there is a dearth of information for rock–steel interface testing under the conditions that may be encountered in tidal stream or other renewable energy foundation applications, namely, non-bonded connections to the seabed under relatively low normal stresses and variable stiffness conditions. It is acknowledged, however, that tidal stream generator foundations will also be subjected to cyclic loading which will need to be considered when interpreting the results of the monotonic tests presented in this paper.

This paper presents results of chalk–steel interface testing utilising a specially commissioned torsional interface shear tester (IST). This device was used to investigate the interface material properties that influence interface shear strength under stress and displacement levels appropriate to tidal stream GBS foundations. The potential for the use of the equipment in large-displacement events has also been explored, which may be relevant to driven pile installations that can potentially be used to anchor or support tidal stream generator alternatives. This paper focuses on applications on chalk because it is a rock with extraordinary characteristics (Lord *et al.*, 2002) and areas in the south of the UK that have been identified as of significant tidal stream potential may consist of chalk seabeds (e.g. Race of Aldernay and Casquets, which represent around 6% of the total UK tidal resource (Carbon Trust, 2005)), or where driven pile solutions may encounter such material types.

The aim of this paper is to provide information regarding the controlling parameters on the foundation–seabed interface sliding resistance and to provide interface properties that have the potential to be used during the design process.

2. Laboratory testing

2.1 Description of chalk samples used for laboratory testing

The samples were collected from the active Imerys Mineral Limited's Quarry, Westwood, Beverley, HU17 8RQ, UK (501740, 438256). Blocks of chalk typically 350 by 300 by 280 mm were obtained directly after quarrying and prior to crushing for use in the chemical industry. Unfortunately, owing to the working status of the quarry and the required health and safety regulations, the research team was not directly involved with the sampling of the chalk, thus making it difficult to comment on the structural setting of the chalk in situ. The chalk is White Chalk from the Flamborough Chalk Formation (Upper Chalk unit, northern province English Chalk) referred to informally as the Flamborough Sponge Bed (Lord *et al.*, 2002; Whitham, 1991, 1993). This source of material was selected because of the fresh nature of the chalk (i.e. recently quarried),

Property	Chalk	Sandstone
Dry density, ρ_d: Mg/m^3	2·06	2·23
Porosity, n: %	23	13
Voids ratio, e	0·3	0·15
Saturated moisture content, m_{sat}: %	11·4	6
Specific gravity, G_s	2·7	2·57
UCS, dry samples: MPa	30·00	31·50
UCS, saturated samples: MPa	9·30	—
Tensile strength, T_0, dry samples: MPa	1·10	2·60
Tensile strength, T_0, saturated samples: MPa	0·96	—
Young's modulus, dry samples: GPa	8·60	10·20
Young's modulus, saturated samples: GPa	2·85	—

Table 1. Summary of key index properties for the chalk and sandstone samples

immediately placed under cover and the fact that the chalk was free from flints that may interfere with characterisation and interface testing.

The chalk was characterised using both field and laboratory techniques prior to interface shear testing and the results are summarised in Table 1. These results classify the chalk as of very high density according to CIRIA 574 (Lord *et al.*, 2002). The chalk on return to the laboratory had a very low moisture content of 0·3%. Dry density and saturation moisture content determination (BS 1377-2: 1990 (BSI, 1990)) highlighted that the moisture content of the chalk was below the 90% minimum level of saturation recommended for field identification procedures so the chalk was saturated prior to these tests by applying 95 kPa vacuum to submerged samples (in de-aired and de-ionised water) for 24 h. In order to ascertain the level of saturation, the moisture content after following the aforementioned process was compared to the saturation moisture content $w_s = 11·49\%$ of the chalk (calculated according to BS 1377-2:1990 (BSI, 1990)) and very high levels of saturation were achieved (99·6–99·7%). Samples were also oven dried in order to investigate the effect of moisture content on the unconfined compressive strength (UCS), tensile strength and interface shear resistance behaviour.

2.2 Scope of testing

Interface testing between chalk–steel interfaces at normal stresses relevant to those anticipated at real tidal stream projects (Ziogos *et al.*, 2015b) was carried out in order to obtain the friction properties necessary for the determination of the sliding resistance of a GBS. The UCS of chalk has previously been found to vary significantly with saturation levels, generally showing lower strengths for saturated samples compared to dry ones (Matthews and Clayton, 1993). Therefore tests using both dry and saturated samples were carried out in order to examine the variation of UCS on the shear resistance. In

Material	Average roughness, R_a: μm
Chalk (saw cut)	3·1
Sandstone (saw cut)	19·0
Polished steel	0·4
Machined steel 1	7·2
Machined steel 2	34·0

Table 2. Summary of the interface properties of the materials tested

addition, the effect of steel roughness was investigated ($R_a = 0·4–34$ μm, Table 2) along with the effect of normal stress ($\sigma_v = 16–1000$ kPa) over relatively short displacements of 10 mm during shear.

Sandstone–steel interface testing was also undertaken ($\sigma_v = 16–316$ kPa) to allow the comparison between the interface behaviour of chalk and a typical sedimentary rock (sandstone) that exhibits more 'conventional' behaviour. The samples used for the testing were sourced from the Caithness area and specifically from a disused quarry located south of John O'Groats, Scotland, UK (ND37150 70138). This area was selected because it is adjacent to the Pentland Firth, which exhibits significant tidal resource (Carbon Trust, 2011). The Old Red Sandstone that was recovered for testing is yellow-orange in colour and medium grained (Johnstone and Mykura, 1989). In addition the laboratory determination of UCS revealed a very similar value (Table 1) compared to that of the dry chalk samples, which allows comparison of the shear resistance of the two rock types focusing on parameters other than the compressive strength (e.g. the grain size that differs significantly between sandstone and chalk).

In addition to the tests designed to investigate the behaviour of chalk relevant to tidal stream generator GBS foundations (low normal stress and low displacement), an extra set of tests was undertaken to very large cumulative displacements (7·0 m). These tests were to check the potential of the IST device but also may be considered relevant to the displacements that may be encountered in driven piling or underneath a sliding foundation or tow head used in the offshore oil and gas industries.

2.3 Determination of unconfined compressive strength

The tensile strength T_0 of the rock samples was determined using the Brazilian tensile test (ASTM D3967-08 (ASTM, 2008)). Initially the tensile strength was used as an indirect method to determine the UCS of the chalk using conversion factors. The results indicated that the empirical conversion factors used to convert the tensile strength to UCS, as found for other sedimentary rocks (e.g. Altindag and Guney, 2010), were not appropriate for chalk; therefore direct unconfined

Interface				
	σ_v: kPa	Chalk–steel $R_a = 0{\cdot}4$ μm	Chalk–steel $R_a = 7{\cdot}2$ μm	Chalk–steel $R_a = 34$ μm
		Measured peak interface friction angle, δ_{peak}: degrees		
Tilt table	~0·6	30·0	36·5	37·5
IST	16	36·0	39·5	40·5
IST	79	39·5	41·5	42·5
IST	159	37·0	40·0	39·5
IST	316	33·5	37·0	40·0
IST	700	32·5	31·5	38·5
IST	1000	31·0	31·5	35·0

Table 3. Comparison of results of chalk–steel interface testing utilising the tilt table and the IST device

compression tests were carried out in accordance with ISRM (2007) utilising a 250 kN Instron universal testing machine. Direct UCS tests were carried out on cylindrical oven-dried and saturated chalk samples with a height to diameter ratio equal to 2 (see Table 1) at a strain rate of 0·1 mm/min. Dry sandstone samples were also tested for comparison purposes. Both the oven-dried sandstone and chalk samples exhibited very similar values of UCS (31·5 and 30·0 MPa, respectively), whereas a 69% decrease in UCS was observed for the chalk when tested in a saturated condition. Deterioration of chalk UCS after saturation has been reported before by Matthews and Clayton (1993); however, they noticed a smaller average reduction of 50%. This difference may be due to the high density of the chalk tested in this study, whereas Matthews and Clayton (1993) report UCS for a variety of chalk densities.

2.4 Tilt table testing

Prior to the main interface testing the basic friction angle of the chalk ($\phi_b = 30{\cdot}5°$) and sandstone was determined using simple tilt table testing in line with the methodology outlined in USBR 6258 (USBR, 2009). This involves tilt table testing of two 54 mm dia. samples of 27 mm thickness placed on top of each other. The samples were prepared by coring of a block of chalk (54 mm nominal diameter) and then dry cross-cutting of the core using a diamond saw. The interface frictional resistance was determined on this saw-cut surface (as per USBR 6258 (USBR, 2009)). The ϕ_b values were 30·5° and 38·5° for chalk and sandstone, respectively. Previous experience of the results from the low normal stress tilt table tests show good correlation with the more advanced testing techniques at elevated stress levels (Table 3). Therefore, apart from using the tilt table test to determine the basic friction angle, the simple test was also used to test the chalk samples against the steel interfaces used in the main study.

2.5 Description of the IST device

A computer-controlled torsional IST (supplied by GDS) was utilised for the execution of the main part of the interface shear testing programme (Figure 1). This device consists of an axial actuator at the top of the rig, which can apply up to 5 kN of vertical load, and a rotational actuation system at the base, capable of applying torque up to 200 Nm. Below the axial actuator is a combined load/torque cell arrangement with capacities of 5 kN and 200 Nm, respectively. The axial actuator applies the normal load to the samples under test and is fixed against rotation, whereas the rotational actuator applies the torque/rotation from below (tests can either be torque or rotation controlled).

A special clamping system was developed to allow rectangular interchangeable foundation interface elements of 65 × 90 mm with a thickness of 8 mm to be clamped at the base of the rig above the rotational actuator (Figure 1(b)). Similarly, below the load/torque load cell a clamping device was developed to clamp short, round rock samples (54 mm dia. and 25 mm high). During the test, the upper rock sample was fixed while the lower steel sample rotated at a predetermined rate (rotation controlled test). The IST used here is an evolution of that previously used by Kuo *et al.* (2015) for the low-stress interface testing of pipelines (referred to as the 'Camtor' device). This previous device incorporated an outer pressure cell and allowed the testing of soil samples up to 70 mm dia. and 20 mm thick against pipeline surface elements. During the tests torque and normal load were measured using a calibrated torque/load cell and vertical and rotational deformation measurements were provided by a calibrated encoder attached to the stepper motor.

The tests were conducted under constant normal stress conditions on both dry and saturated samples under six different normal stress levels of 16, 79, 159, 316, 700 and 1000 kPa. In order to conduct testing of submerged interfaces for saturated samples, a custom-made poly(methyl methacrylate) (PMMA) box (bath) was attached to the lower rotation platen, Figure 1(c). The saturated rock sample was clamped using the top holder and the bath was filled with de-ionised water, in order to prevent drying of the saturated sample. For saturated tests, the interface was kept under constant normal stress for 15 min before the initiation of shearing in order to allow any excess water pressure dissipation at the interface; for the same reason the shearing rate was kept at a low level at 0·05 mm/s of equivalent horizontal displacement. Every test lasted approximately 36 min and was terminated when an equivalent horizontal displacement of 10 mm was reached (corresponding to 42·5° rotational displacement). In addition to the aforementioned tests, three tests up to a horizontal displacement of 7·0 m (29 750° rotational displacement) were carried out under three different stress levels (100, 300 and 700 kPa) using saturated samples. The shearing rate for these tests was 1·25 mm/s and each test lasted 16·5 h. A higher rate was used for these tests in order to allow the execution of each test within a

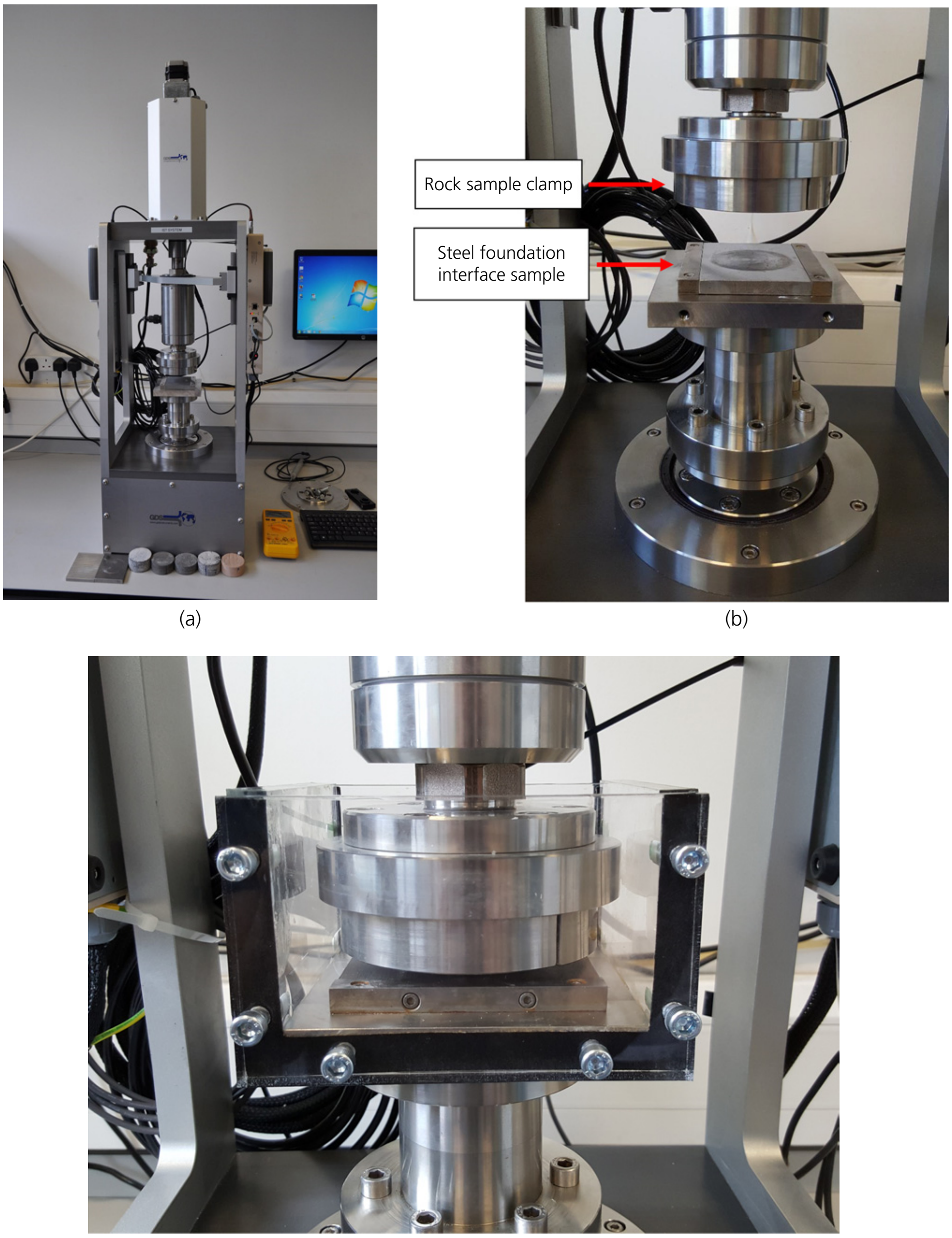

Figure 1. (a) Interface shear tester apparatus. (b) Detailed view of the IST sample mounting arrangement. (c) Detailed view of PMMA bath used for saturated testing

reasonable time frame. The scope of these tests was to investigate any possible degradation on the chalk surface (and consequently to the shear strength of the interface) at very high shear deformations that can potentially occur for applications such as pile driving. Therefore, the steel plate with $R_a = 7{\cdot}2$ μm was selected, as the roughness lies in the middle of the roughness range of steel piles used in practice ($R_a = 5$–10 μm for steel piles (Barmpopoulos *et al*., 2010)).

The torque measured during IST testing was converted to average shear stress as per Equation 1 after considering Saada and Townsend (1981) for ring shear testing.

1. $$\tau = \frac{T}{\int_0^r 2\pi R^2 \mathrm{d}R} = \frac{3}{2\pi r^3} T$$

The radial deformation was converted to a linear displacement at a reference point considered at a distance equal to half of the radial length of the circular rock sample, as per Equation 2.

2. $$d = \theta \frac{r\pi}{360}$$

where θ is rotational displacement, τ is shear stress, d is linear displacement, r is the rock sample radius, R is radial position and T is torque.

2.6 Description of steel samples (foundation analogues)

Mild steel was used to prepare rectangular ($65 \times 95 \times 8$ mm) plates that represented foundation analogues for the interface testing. As discussed in the introduction and found in the literature (Ziogos *et al.*, 2015a, 2015b), roughness has a major effect on the interface behaviour, therefore different preparation techniques (polishing and machining) were applied and resulted in three different foundation analogues with a wide range of surface roughness (R_a between 0·4 and 34 μm).

Polishing with a surface grinder using a BAA60 – K7V wheel resulted in surface roughness average $R_a = 0{\cdot}4$ μm. Machining, using a shaping machine and an appropriately adjusted shaping tool, resulted in R_a values of 7·2 and 34 μm. The range of roughness obtained covers the roughness of some of the steel elements commonly found in geotechnical applications (for example, $R_a = 5$–10 μm for steel piles, Barmpopoulos *et al.*, 2010), allowing the utilisation of the results in other applications.

2.7 Surface roughness characterisation

Interface roughness can be quantified by many parameters, but one of the most frequently used is R_a (centre-line average roughness), which is the computed average of all deviations of the roughness profile from the median (centre) line over a defined profile length and is defined in Equation 3 (Degarmo *et al.*, 2003). A hand-held Taylor Hobson Surtronic Duo stylus contact profilometer was used to determine the average centre-line roughness (R_a) of all of the samples used for interface testing (rock and steel). The profilometer has a range of measurement up to 40 μm and a traverse length of 5 mm. For each sample, five R_a measurements were taken and the mean value was selected. Calibration of the device was carried out periodically against a standard roughness profile with $R_a = 5{\cdot}81$ μm and $R_z = 21{\cdot}5$ μm (supplied with the device). The interface properties of all the materials used for testing (rock and steel samples) are summarised in Table 2.

3. $$R_a = 1/L \int_0^L |z(x)|\mathrm{d}x$$

where L is profile length and $z(x)$ is deviation from the centreline at point x.

3. Results

Figures 2(a)–2(c) show the normalised shear stress–displacement curves from saturated chalk–steel interface tests on steel of increasing roughness. A typical result shows a slightly elevated initial shear stress followed by a slight reduction in shear stress post peak (or yield) and then remaining relatively constant until the end of the test. It is apparent that yielding or peak shear stress is observed at increasing displacement levels as the normal stress on the chalk increases, as seen in Figure 2(a) for normal stresses above 159 kPa. It is also noticeable that, as the normal stress increases, there is an increase in shear resistance up to a normal stress of 79–159 kPa and then a reduction in the shear resistance, with the lowest shear resistances associated with the highest normal stress of 1000 kPa. Table 4 shows the summarised results of testing where δ_{peak} (Table 3) is defined as the maximum value at a shear displacement up to 4 mm and δ_{ult} is defined as the minimum value in the region of 8–10 mm. The results in Figure 2(d) show the tests for sandstone sheared against steel of $R_a = 7{\cdot}2$ μm where the curves are more 'noisy' because sandstone is rougher and harder, whereas it is apparent that higher interface shear stresses are mobilised in the softer chalk.

Figures 3–5 show the results of IST testing of the saturated and dry chalk against the various steel interfaces in terms of the different normal stresses. For the dry tests the peak interface friction angles δ_{peak} range from 35° to 45° and δ_{ult} from 22·5° to 40°. For saturated tests, δ_{peak} ranges from 27° to 42° and δ_{ult} from 22° to 40°. All of the results from IST testing are summarised in Table 4. These results suggest that the shear stress–displacement response at the interface is not proportional to the normal stress. It is possible that a simple constant friction angle based on Mohr–Coulomb failure criterion model for interface shearing may not be appropriate for chalk. The peak interface friction angles noted do not seem to be significantly affected by the degree of saturation, although the values are higher for the dry tests. They do not seem to reflect the variation of UCS change noted between dry and saturated chalk samples (69% reduction in UCS from dry to saturated).

All of the figures have been annotated with the value of the interface friction angle derived from the low-stress tilt table testing (Table 3) and the basic chalk–chalk friction angle (again obtained from tilt table testing as described earlier). The derived interface friction angles from IST testing tend to exceed these values for the lowest and highest steel roughness (0·4 μm and 34 μm), whereas the results for the dry and saturated tests against 7·2 μm steel only appear to exceed this at the highest interface angles. This suggests that, in the case of chalk, the normal stress level affects the results obtained from the IST when compared to tilt table testing, although this is

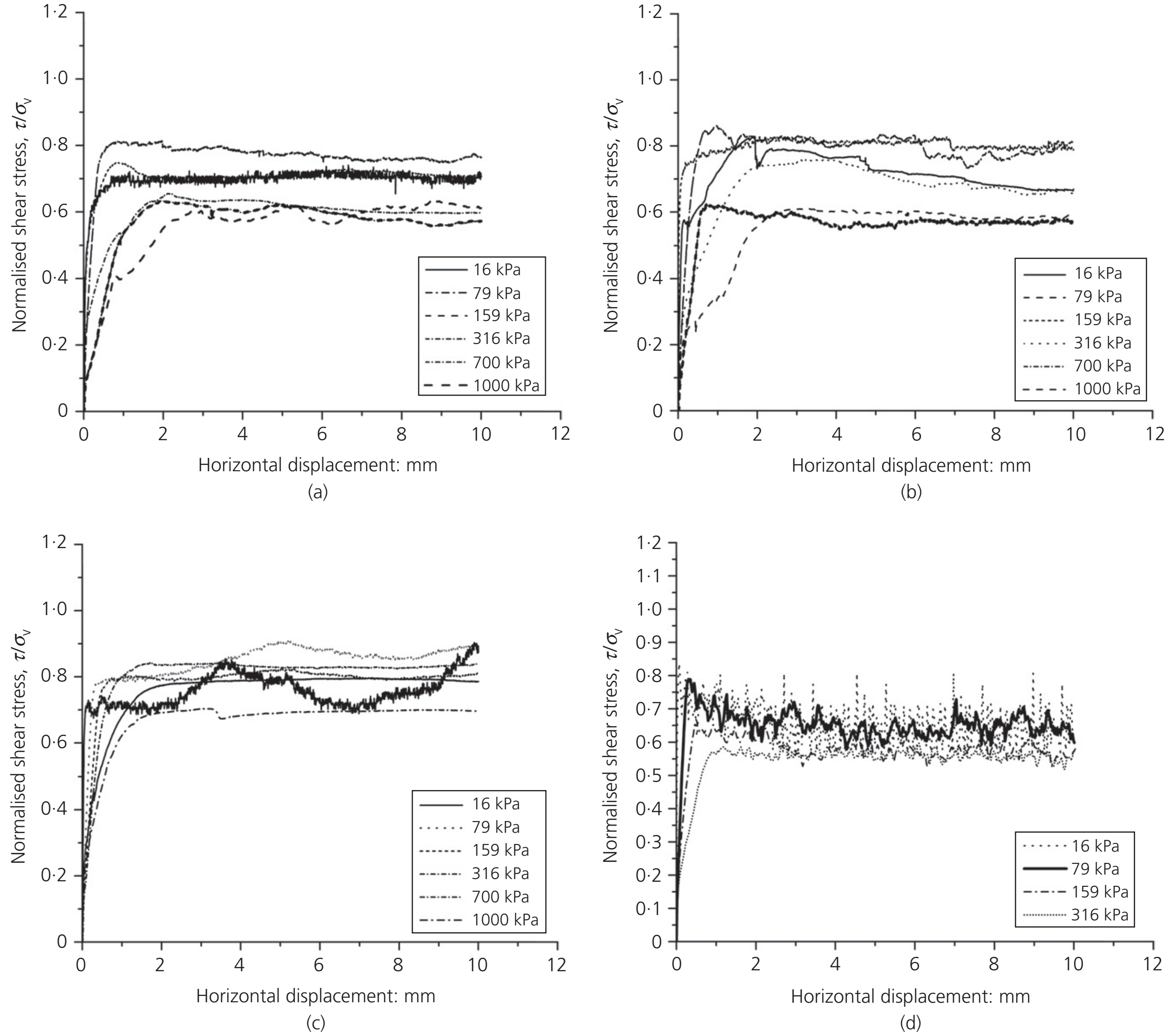

Figure 2. Normalised shear stress plotted against horizontal displacement for saturated chalk samples against: (a) steel $R_a = 0{\cdot}4$ μm; (b) $R_a = 7{\cdot}2$ μm; (c) $R_a = 34$ μm; and (d) dry sandstone samples against steel $R_a = 7{\cdot}2$ μm

not the case for other higher strength rocks. This variation in behaviour suggests a transition in moving from 0·4 μm to 7·2 μm steel interface, which is reflected in both the IST and tilt table testing.

In the IST, irrespective of the steel type, the interface resistance exhibits a low value at normal stress of 16 kPa, which may indicate poor interlocking between the chalk and steel interface as the applied stress is not adequate to bring the two solid bodies into intimate contact and shearing is occurring on the top of the asperities (steel and chalk). As the normal stress increases, the interface gains higher shear strength, as better interlocking is established between the normal stresses of 79 and 159 kPa (7·2 to 16·6% of the chalk tensile strength, T_0). At these stress levels the shearing is accompanied by observable damage on the chalk surface, seen as a layer of powder (dry samples) or chalk putty (saturated tests) on the steel interface on post-test sample separation. This behaviour is similar to that noted for rock analogues (cement blocks) by Ziogos *et al.* (2015b). For normal stresses from 316 kPa to 1000 kPa (i.e. 0·31 to $1{\cdot}0T_0$) the ultimate interface friction angle reduces to values typically between 30 and 35° and in the majority of cases appears to be approaching the basic chalk–chalk friction angle value noted for the 0·4 μm interface. This suggests that

Normal stress: kPa	Initial sample state[a]	Post-test sample condition[b]	R_a: μm					
			0·4		7·2		34·0	
			Peak shear stress: kPa	Ultimate shear stress: kPa	Peak shear stress: kPa	Ultimate shear stress: kPa	Peak shear stress: kPa	Ultimate shear stress: kPa
16	D	I	13·9	7·5	13·5	11·0	12·62	11·5
16	S	I	11·5	10·5	13·0	10·5	13·5	12·0
79	D	I	75·5	57·5	76·9	60·5	72·5	70·5
79	S	I	65·0	59·0	70·2	58·0	72·0	69·5
159	D	I	136·0	106·0	147·5	131·0	143·0	141·0
159	S	I	119·0	112·0	132·5	125·5	131·0	127·0
316	D	I	287·0	273·5	235·6	215·5	318·0	316·0
316	S	I	207·5	188·0	237·2	215·0	266·0	263·5
700	D	SD	500·0	433·5	476·3	410·6	614·0	606·5
700	S	SD	450·0	399·0	432·5	400·0	558·0	546·0
1000	D	NI	748·0	739·5	650·3	600·5	803·5	643·0
1000	S	NI	600·0	560·5	608·5	582·0	705·0	694·0

[a]S, saturated; D, dry samples
[b]I, intact; SD, surface damage; NI, non-intact samples

Table 4. Summary of results from interface testing of chalk–steel interface utilising the IST device

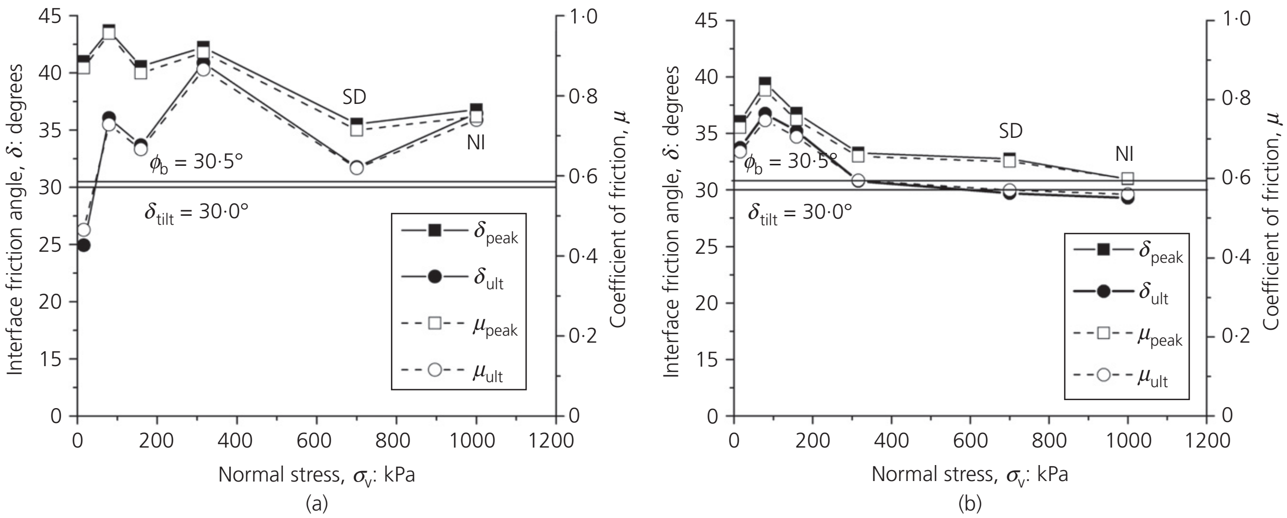

Figure 3. Variation of interface friction angle and coefficient of friction for chalk–steel interface test for steel with $R_a = 0{\cdot}4$ μm against (a) dry and (b) saturated chalk

damage at the interface may be filling the rough surface of the rougher steel samples and reducing their apparent interface roughness to that approaching the smoothest interface tested here. Significantly more damage was noticed in some samples tested at 700 kPa and 1000 kPa, where parts of the perimeter of the sample were chipped off (labelled as surface damage, SD in Table 4), or at the highest normal stress level (1000 kPa) resulted in complete tensile failure of the sample (NI) as shown in Figure 6. Therefore in the case of chalk the upper limit to the interface strength appears to be linked to the local

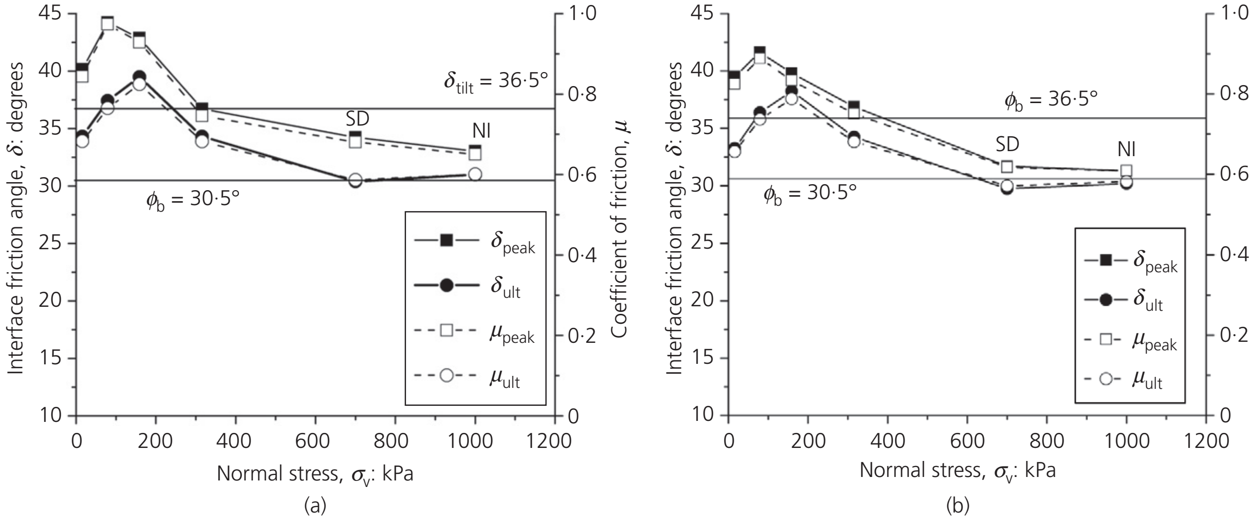

Figure 4. Variation of interface friction angle and coefficient of friction for chalk–steel interface test for steel with $R_a = 7{\cdot}2$ μm against (a) dry and (b) saturated chalk

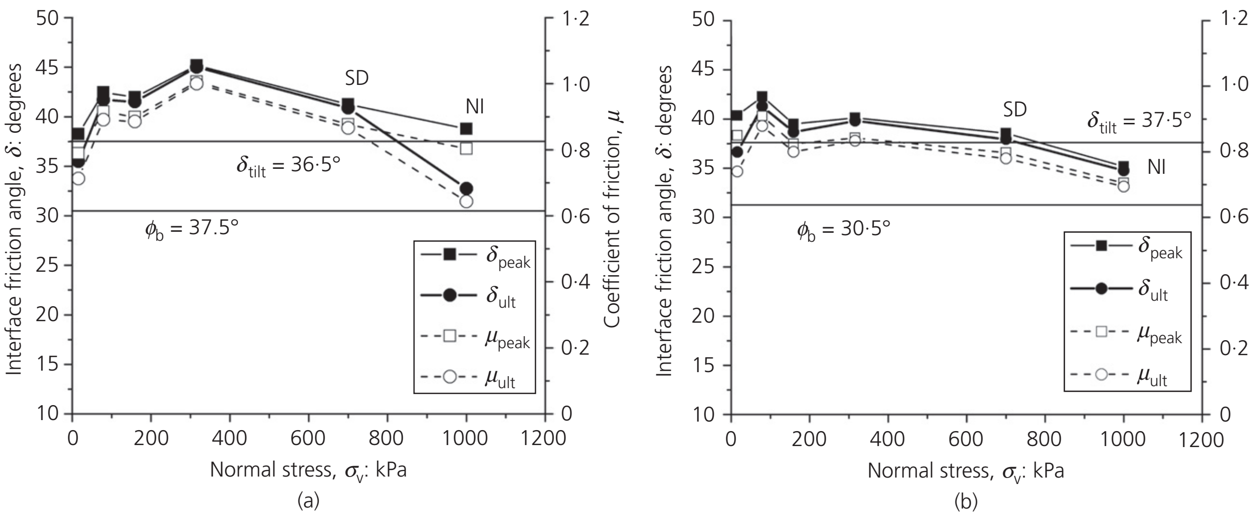

Figure 5. Variation of interface friction angle and coefficient of friction for chalk–steel interface test for steel with $R_a = 34$ μm against (a) dry and (b) saturated chalk

surface strength of the material (similar to the grain size of sand exceeding the roughness of steel as discussed earlier) with potential for catastrophic disruption of the interface at higher normal stresses on approaching the tensile strength of the chalk ($T_0 = 0{\cdot}96$ to $1{\cdot}1$ MPa). Although some of the samples tested at the higher stress of 1000 kPa were not intact after removal of the sample from its clamp, it is believed that the interface shearing behaviour is valid as the clamping system maintained the integrity of the sample and shearing surface during testing. The reduced shear stress noted during testing at these stresses reflects the increased interface damage. The roughness of the steel interfaces tested was measured before and after testing for the low-displacement tests and no significant variation in roughness was noted for either rock type. The roughness of the rock samples was also measured and this remained within the variability of the average values typically measured. A similar procedure was undertaken for the large-displacement tests, which showed that the steel tested against chalk exhibited no significant change in roughness after careful removal of the chalk residue. Testing of the steel with the

Figure 6. Tensile failure of a dry chalk sample sheared at 1000 kPa

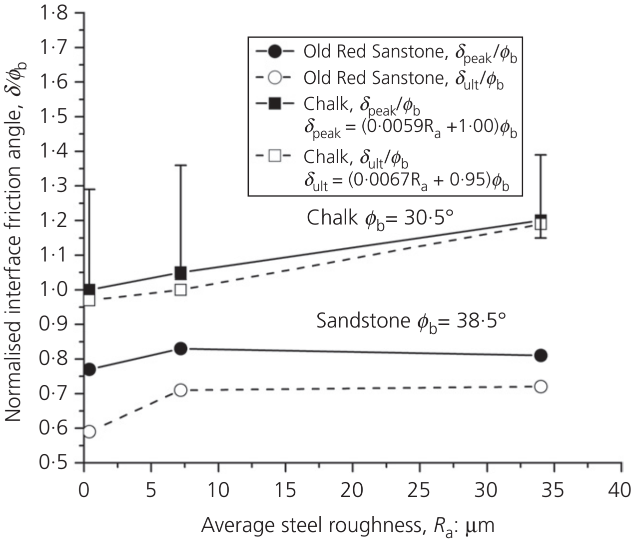

Figure 7. Variation of normalised friction angle with increasing steel roughness, for saturated chalk samples and dry Old Red Sandstone

chalk residue in place was not undertaken, as the surface was relatively uneven and non-continuous over the steel surface making roughness determination difficult. Visual degradation of the steel surface of the large-displacement tests against sandstone was noticed, with post-test roughness measured as 5·5 µm as opposed to pre-test roughness of 7·2 µm. The sandstone itself did not show any significant change in roughness, but black residue was noticed on its surface which was assumed to come from the steel.

As mentioned earlier, the adoption of a linear failure envelope for chalk does not seem appropriate for design purposes, since the interface friction angle is affected by the normal stress level. Although, in order to allow comparison, linear failure envelopes for peak and ultimate interface resistance were calculated. These are based upon the average peak or ultimate resistance determined over the range of effective stresses tested. To allow the effect of normal stress and potential for surface degradation and damage to be represented, the range of normalised friction angles obtained are denoted by vertical error bars, as shown in Figure 7 (shown for chalk peak values only for clarity). The results suggest that, over the steel roughness range investigated ($R_a = 0{\cdot}4$–34 µm), on average the interface becomes stronger as the steel roughness increases, without reaching a 'plateau' as seen in other studies (Barmpopoulos *et al.*, 2010; Jardine *et al.*, 1993; Ziogos *et al.*, 2015b) and as shown for sandstone. However, when increasing normal stress is considered there appears to be a tendency for the chalk–steel interfaces to tend towards the basic chalk–chalk interface properties.

3.1 Large-displacement interface testing

Results from the large-displacement tests can be seen in Figure 8 compared with the result from a similar test undertaken on sandstone for comparison. It is clear from the tests on chalk at all normal stress levels that there is continuous degradation of the shear surface throughout the test. This is similar to the behaviour observed by Barmpopoulos *et al.* (2010) testing sand against concrete to large displacements, which was attributed to observed sand particle crushing and generation of fines. At the lowest stress of 100 kPa the friction angle has fallen from an initial peak of 43° to 37° at 7·0 m with the rate of degradation appearing to reduce. This value is similar to that noted from the low-stress tilt table testing of chalk–steel (Table 3). At 300 kPa normal stress, reduced initial degradation is observed with a relatively constant interface friction angle of 30–31° being reached after 2·4 m of displacement, which tends to the basic chalk–chalk interface friction angle and which may be explained by the degradation behaviour described above for the low-displacement tests. In contrast, at a normal stress of 700 kPa there is a significant reduction in the interface friction angle below the basic chalk–chalk interface friction angle until reaching a relatively constant value of 17° ($0{\cdot}56\phi_b$) at 6·5 m. Previous low-displacement testing results shown earlier may have led to the recommendation of a lower safe bound design interface friction angle of approximately 29° ($0{\cdot}95\phi_b$) which could be determined from the basic chalk–chalk interface friction angle. In the case of large-displacement events this may be a suitable approach where the normal stresses do not exceed 300 kPa ($0{\cdot}31T_0$). Where this value is exceeded then a more conservative interface resistance must be assumed. The behaviour observed in Figure 8 for the large-displacement test on sandstone shows very different behaviour, with increasing resistance with increasing displacement up to 1·7 m displacement and then a more gradual increase with increasing displacement which

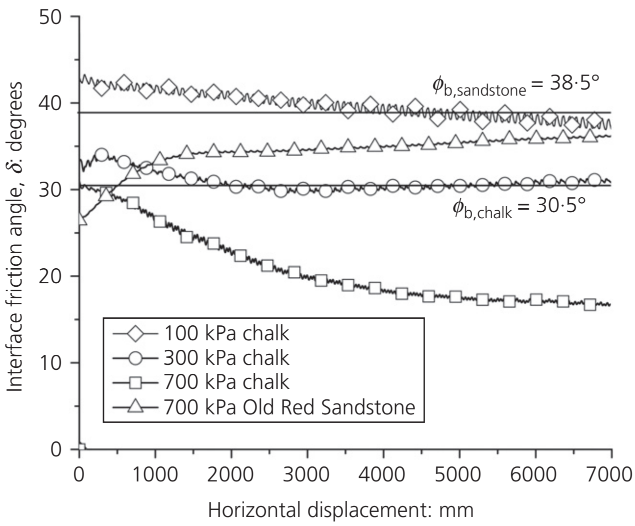

Figure 8. Interface friction angle plotted against horizontal displacement for saturated chalk samples

again appears to be tending to the sandstone–sandstone basic interface friction angle. This may be due to the removal of weak exposed sandstone asperities (individual weakly cemented grains), but this behaviour requires further investigation. What is apparent from the testing is that, unless normal stresses are high enough to cause significant interface and sample damage, large-deformation tests on steel ($R_a = 7{\cdot}2$ μm)–rock interfaces result in interface behaviour that tends to the basic low-stress rock–rock interface behaviour (for both chalk and sandstone–steel interfaces).

It should be noted that the testing regime here is constant normal stress (similar to that adopted by Barmpopoulos *et al.* (2010) for ring shear tests on sand–steel and sand–concrete interfaces) whereas in the case of driven piles a constant normal stiffness regime may more adequately represent in situ conditions leading to a reduced potential for tensile strength linked degradation. This assumes, however, that the chalk in situ is intact and well confined without faults or voids/low-strength zones. In addition, constant normal stiffness conditions may lead to significantly higher in situ stresses than those tested here, which could result in a more rapid degradation with displacement.

4. Implications for testing and design

4.1 Utilisation of tilt table for simple interface characterisation

Table 3 compares the results of interface testing using the tilt table to those obtained from IST testing. It can be seen that there is good agreement between the peak interface friction angle measured in both types of test, especially for the rougher steel plates ($R_a = 7{\cdot}2$ and 34 μm) and for σ_v up to 700 kPa (i.e. $0{\cdot}7T_0$). Therefore, it is possible to utilise the tilt table test to estimate the peak friction angle for preliminary design purposes, and to use IST (or large-displacement direct shear box tests) in detailed design. Based on the IST tests here for chalk (Figures 3–5), and the apparent tendency to degrade to low friction angles with increasing normal stress levels and interface degradation, tilt tables tests for chalk against a relatively smooth interface may give a useful lower bound for design. For both the low and high displacements it would seem that the normal lower-bound interface friction angle should be taken as the basic chalk–chalk friction angle from tilt table testing, irrespective of the foundation material or roughness. For higher displacement tests, however, some caution has to be exercised when normal stress levels exceed $0{\cdot}31T_0$. Similarly, in the prototype deployment of tidal stream generator, foundations are likely to experience cyclic loading that has the potential to cause degradation at lower stress levels and lower displacements.

4.2 Potential design approaches

Previously, Ziogos *et al.* (2015a, 2015b) have proposed an adhesion factor (α) (shear stress normalised by UCS) type approach for monotonic rock–steel and cement–steel interface strength prediction similar to that developed for rock socket pile adhesion factors (Tomlinson, 2001). In this case, however, the magnitude of shear stress is lower by several orders of magnitude compared to pile applications, due to the unbonded nature of the interface and the CNS conditions for rock socketed piles. UCS was also normalised by the vertical stress during interface testing, as this was seen to have a significant effect over the relatively low stresses likely to be encountered at the rock–steel interface (as also observed in this study). It is assumed that such a normalisation is not applied for rock socket piles due to the high confining stresses and difficulty in determining the actual in situ stress at the bonded interface. Figure 9 shows lines that represent the adhesion factor values obtained from previous monotonic interface testing of various rock–steel interface combinations. The lines are described by Equation 4 and allow the calculation of the maximum shear stress capacity of the interface, for a given rock type (UCS), foundation footing (steel R_a) and the anticipated applied average normal stress (σ_v).

$$\text{4.} \quad \alpha = b\left(\frac{\text{UCS}}{\sigma_v}\right)^c$$

The lines previously determined for various steel roughness levels (R_a from 0·4 to 7·2 μm and UCS from 45 to 157·2 MPa) have been extrapolated to cover the range of testing undertaken in this study. For steel $R_a = 0{\cdot}4$ μm, fitting constants b and c may be taken as 1·78 and −1·26, respectively, whereas for $R_a = 7{\cdot}2$ μm, $b = 1{\cdot}25$ and $c = -1{\cdot}16$. Specific data points are also shown for the IST testing of Old Red Sandstone by way of comparison. It can be seen that the previously determined

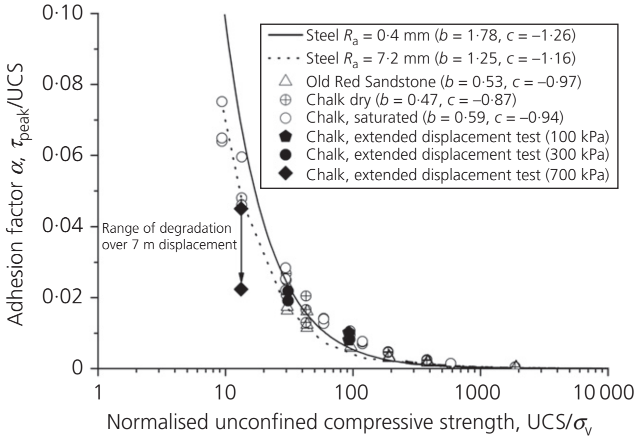

Figure 9. Adhesion factor values for chalk and sandstone compared to contours from Ziogos *et al.* (2015a) (steel $R_a = 7{\cdot}2$ μm)

relationship for rocks of much higher UCS seems applicable for the much lower strength chalk and offers an alternative design approach to an interface friction angle based approach, where low monotonic displacements occur (which could be attempted from the results in Table 4 or the lower-bound basic friction angle). Additionally, the results are shown for the large-displacement tests on chalk (Figure 8), which suggest that the approach based upon Equation 4 should be used with caution for chalk where large displacements and/or cyclic loading may occur at an interface during installation or service. Adhesion factor ranges from 0·001 ($\sigma_v = 16$ kPa) to 0·07 ($\sigma_v = 1000$ kPa) for saturated chalk samples are similar to the cohesion intercept/UCS ratio (ranges from 0·02 to 0·13) defined by Clayton and Saffari-Shooshtari (1990) from interface tests on bonded planar chalk–concrete interfaces. This suggests that the strength of the material is potentially controlling behaviour irrespective of whether or not the concrete is bonded or unbonded.

5. Summary and recommendations

Interface shear testing between a very high-density chalk and steel of varying roughness was undertaken to gain insights for tidal stream generator GBS foundation design. In addition, interface testing using Old Red Sandstone samples was undertaken for comparison purposes.

Low-displacement interface shear testing of chalk–steel interfaces at low normal stresses showed a tendency for increasing shear resistance up 159 kPa (i.e. $0{\cdot}16T_0$). At stress levels above this and in particular up to 316 kPa ($0{\cdot}33T_0$), the interface shear resistance begins to degrade and tends to the basic chalk–chalk interface behaviour (with increasing normal stress) determined at low stress from tilt table testing. This suggests damage at the chalk–steel interface, which was observed as chalk dust or putty on the steel interfaces after testing. As the normal stress was increased further the chalk displayed surface damage ($\sigma_v = 700$ kPa, $0{\cdot}76T_0$) and fracturing ($\sigma_v = 1000$ kPa, $1{\cdot}10T_0$). The average chalk–steel interface shear strength appears to increase linearly with increasing roughness of the steel, which is in contrast to the results of sandstone–steel interfaces, which reach a ‘plateau’ at an average steel roughness of 7·2 μm, but this behaviour is highly dependent on the normal stress levels.

Large-displacement monotonic tests of saturated chalk samples (up to 7·0 m) revealed a degradation of interface strength for increasing displacement with a tendency for resistance towards the basic chalk–chalk interface behaviour measured from tilt table testing ($\phi_b = 30{\cdot}5°$) at normal stress levels up to 300 kPa ($0{\cdot}33T_0$). At normal stress levels above this ($\sigma_v = 700$ kPa), degradation of the chalk–steel interface was more severe, tending towards half of the resistance at lower stress levels.

Tilt table interface tests were undertaken on chalk–steel interfaces with resulting very low normal stresses (less than 1 kPa), and showed good agreement with the IST results at low normal stresses ($\sigma_v = 16$ kPa), especially for the rougher steel plates used during this study ($R_a = 7{\cdot}2$ and 34 μm). The tilt table test was also used to determine the basic chalk–chalk interface friction angle, which seems to be a key indicator of behaviour for degraded chalk, which in some cases may be used as a lower-bound design approach. Results would suggest that the tilt table test is a useful, inexpensive method of characterising rock–steel interfaces that may give useful insights to behaviour for preliminary design.

Comparison of the results from this study with those of other higher UCS rocks would suggest that a previously developed alpha against normalised UCS based design approach to predicting chalk–steel interface resistance may be adopted, but that some care needs to be exercised when crushing or degradation of chalk could occur. For the particular application of tidal stream generator foundations identified in this paper this may be further exacerbated by the cyclic nature of loading encountered, which was not investigated in this paper.

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