Effects of polymer and bentonite support fluids on concrete–sand interface shear strength

C. LAM*, S. A. JEFFERIS† and C. M. MARTIN‡

Support fluids are widely used for the construction of deep bored piles and diaphragm walls. Specifications for the use of these fluids vary, and a thorough understanding of their effect on pile shaft resistance has not yet been developed. This paper presents the results of a set of concrete–sand interface shear tests carried out using both polymer and bentonite support fluids, with water as a reference fluid. It was found that polymer fluids had little effect on the interface shear strength when compared with water. Furthermore, in contrast to the results of earlier studies, concrete curing time was found to have little effect on the results. However, when bentonite slurry was used, the interface shear strength was found to decrease approximately linearly with the square root of the filtration time, until the strength of the pure filter cake was reached. This was due to the development of a bentonite filter cake at the interface so that only aggregate protruding through the filter cake made contact with the sand. It was found that the full concrete–sand shearing resistance could be mobilised when the concrete–sand contact area was greater than about 50% of the total area.

KEYWORDS: laboratory equipment; laboratory tests; piles; shear strength

INTRODUCTION

Bentonite support fluids have been used for the construction of diaphragm walls and bored piles since the pioneering work of Veder (1953). It is now known that in permeable soils such as sand and gravel, the stabilising mechanism involves the formation of a filter cake on the side walls. Cake formation may be assisted by rheological blocking – that is, penetration of the slurry into the ground until the gel strength of the slurry acting over the penetrated area of soil particles is sufficient to prevent further slurry penetration. The bentonite filter cake is of relatively low permeability, so it reduces fluid loss into the ground and provides a membrane against which the hydrostatic pressure of the slurry can act, so stabilising the excavation (Nash, 1974). The formation of a filter cake is necessary for a stable excavation with bentonite slurry in permeable soils. However, for friction piles and barrettes, the filter cake creates an undesirable layer of soft interface material between concrete and soil, and can reduce the interfacial resistance. Fig. 1 shows a diaphragm wall panel constructed at a site in London, with the top few metres of the wall surface exposed. A layer of filter cake with adhering soil can be seen on the wall surface below the level of the guide wall.

Although many studies have been carried out to investigate the effect of bentonite filter cake on interface resistance, a definitive understanding has not yet been developed, and unsatisfactory pile performance continues to be reported. For example, Randolph (2003) described load test results of two bored piles constructed using bentonite fluids in Vietnam. The shaft resistance of the first test pile was below expecta-

tion, so several modifications were made to the construction procedure for the second pile, including: (*a*) steps to reduce the delay between excavation and concreting; (*b*) a reduction in the head of bentonite above river level; (*c*) mechanical scarifying of the side wall prior to casting the pile. These measures were all aimed at reducing the thickness of the filter cake and were reported to have led to an improvement of about 30% in the shaft resistance. This case history poses questions as to whether such steps are also necessary for other projects, and if not, whether current specifications are already adequate for minimising the weakening effect of bentonite filter cake at the concrete–soil interface.

The current UK specification (Institution of Civil Engineers, 2007) requires that, for the construction of bored piles, concrete should be placed within 12 h of the start of the excavation. In the USA, there can also be a time requirement, but it is slightly differently specified. For example, Brown *et al.* (2010) recommend that the time during which the bentonite fluid is left undisturbed in a pile bore should be less than 4 h. However, no such time restriction can be found in the foundation manuals of countries such as Canada and Hong Kong (Canadian Geotechnical Society, 2006; Geotechnical Engineering Office, 2006). The discrepancies between these requirements, or the lack of them, indicates that further work is needed to quantify the effect of fluid support time for better quality control.

This paper presents the results of a laboratory investigation into the effect of support fluids on the interface shear strength between sand and cast in situ concrete. A review of the relevant literature is first given, followed by a description of the equipment used and the test results. Both bentonite and polymer fluids were included in the test programme, with water as the control fluid. The main effects of interest were those of support fluid filtration time, concrete surface roughness and concrete curing period.

PREVIOUS STUDIES

Summary of existing results

Many laboratory studies have been carried out to investigate the effects of excavation support fluids on interface
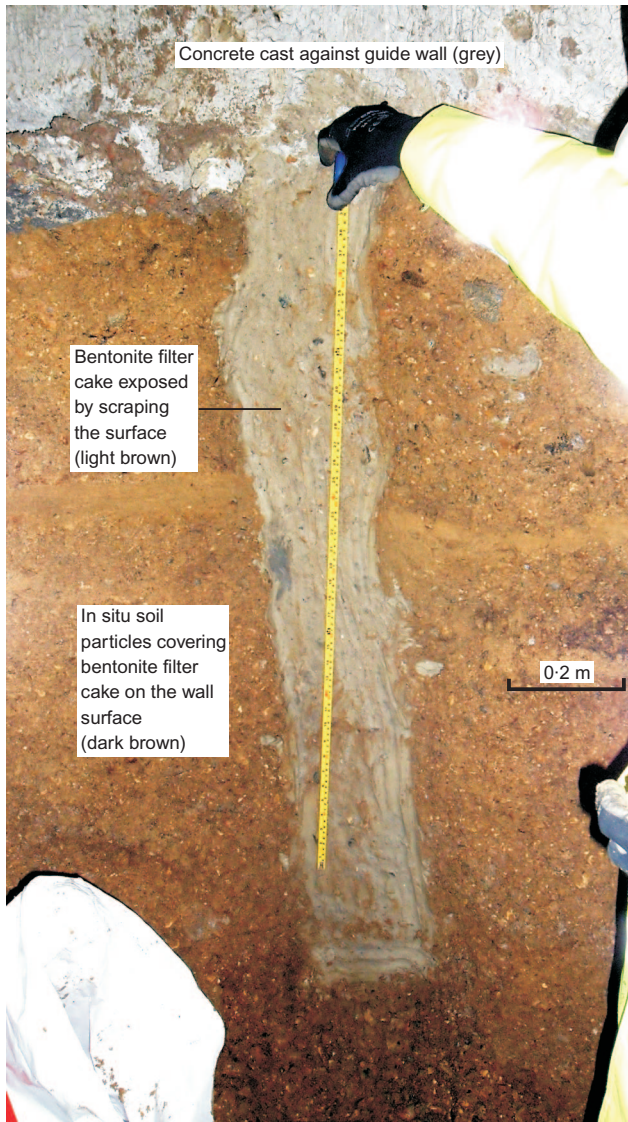
Fig. 1. Exposed bentonite filter cake on surface of diaphragm wall at site in London (courtesy of T. P. Suckling)

shear strength. Table 1 summarises the key results and the experimental techniques used. A brief critical review of these studies is given below.

At the outset it can be stated that almost all previous studies on the effects of bentonite slurry report a reduction in the interface shear strength compared with that for concrete in contact with a clean soil. However, the magnitude of the reported effect differs significantly. For example, Henry *et al.* (1998) reported that the interface strength of their specimens reduced to the strength of pure filter cakes after 6 h of filtration, but Cernak *et al.* (1973) found a similar effect only after 168 h. The time-dependence of strength reduction is also unclear. For example, Cernak *et al.* (1973) and Hu *et al.* (2006) found that increasing the filtration time led to lower interface strength, but Majano *et al.* (1994) reported a V-shaped variation of strength with time (reduction followed by increase). Similarly, publications on the effects of polymer fluids report outcomes ranging from a noticeable reduction (Tawfiq & Broughton, 1996; Shakir & Zhu, 2010) to a significant improvement (Hu, 2003; Hu *et al.*, 2006). These results do not provide a consistent baseline from which to establish and predict the effects of excavation fluids.

Table 1. Summary literature reports of laboratory techniques and results for the interface shear strength of soil or rock previously exposed to support fluids

References	Test method	Soil type	Sample dimension: mm	Filtration method	Filtration pressure: kPa	Filtration time: h	Bentonite filter cake thickness: mm	Concrete mix	Casting method*	Effect of bentonite fluid on interface strength	Effect of polymer fluid on interface strength
Farmer *et al.* (1971)	Model pile tested in a triaxial cell	Fine gravel	Pile: ⌀150, 410 long Soil: ⌀440, 460 long	Hydrostatic head of the fluid column in the model drill hole	n/s (Sand is partly saturated; but no drainage path for filtrate)	1–1·5	5 (before concreting)	1–10 mm aggregate, 150 mm slump	Wet	Little difference at low confining pressure but reduced by up to 10% at high pressure	n/a
Cernak *et al.* (1973)	Direct shear – details unknown	Coarse sand	150 × 440	n/s	25	18, 168	n/s	n/s	n/s	No effect if exposed for 18 h but reduced to pure filter cake strength if exposed for 168 h	n/a
Wates & Knight (1975)	Model pile tested in a large container	Medium sand	Pile: ⌀160, 1200 long Soil: unknown	Hydrostatic head of the fluid column in the model drill hole	n/s (Water table in soil unknown)	24	Decreasing from 9 at top to 4 at base (after concreting)	9 mm coarse aggregate	Wet/ pressure[†]	Reduced by 30% compared to model pile cast 'in the dry'	n/a

Reference	Test	Soil	Dimensions	Method				Material	Condition	Result (wet/primary)	Result (secondary)
Majano & O'Neill (1993); Majano et al. (1994)	Model pile tested in a triaxial cell	Fine to medium sand	Pile: Ø25, 150 long Soil: Ø76, 165 long	Hydrostatic head of the fluid column in the model drill hole	3	0·5, 4, 24	n/s	Plasticised cement mortar, 250 mm slump	Wet	Reduced by an average of 65% compared to cased excavation; largest reduction occurred after 4 h of filtration	Reduced by an average of 40% compared to cased excavation; highest resistance achieved after 24 h of filtration
Tawfiq & Broughton (1996)	Direct shear – soil on concrete	Fine to medium sand	101 × 101	Filtration cell (Ø254 mm)	n/s	36	n/s	n/s	Dry	Reduced by 20% compared to dry sand	Reduced by 7–15% compared to dry sand
Cheng & Haberfield (1998)	Direct shear – concrete on rock	Soft rock	75 × 160 and 80 × 600	Trowelling bentonite paste and painting polymer fluid on the rock surface	n/a	n/a	<0·1 for dry cast, 1–3 for pressure cast	n/s	Dry, wet, & pressure	Reduced by up to 50% when filter cake thickness approaches 70% of roughness asperity height	Very little
Henry et al. (1998)	Direct shear – soil–bentonite mix on sand	Sand	101 × 101	Applying a head difference between the fluid in the upper half of shear box and the soil in the lower half	21	6	4	(Soil–bentonite mix)	Pressure	Reduced to pure filter cake strength after 6 h of filtration	n/a
Ata & O'Neill (1997, 2000)	Direct shear – concrete on sand	Fine sand and stiff clay	Ø64	Static fluid column above the soil sitting in the lower half of shear box	3	3	n/a	Cement mortar	Pressure	n/a	Similar to the unexposed samples if cement mortar was cured for 3 days but 28% to 46% higher if cured for 7 days
Thasnanipan et al. (2003)	Direct shear – sand on concrete	Sand	n/s	Filtration cell	n/s	1	n/s	Cement mortar	Dry	Reduced by 30% compared to unexposed samples	Reduced by 20% compared to unexposed sample (polymer–bentonite mix)
Hu (2003); Hu et al. (2006)	Model pile in a centrifuge	Fine to medium sand (gravel in prototype)	Pile: Ø1525, 7620 and 10670 long‡	Scaled hydrostatic head of the fluid column in the model drill hole	11·4 for bentonite, 34 for polymer‡	11·2, 134·4‡	12·7‡ 71·1‡ (before concreting)	Plasticised cement grout	Pressure (grout sets while spinning)	Little difference in torsional capacity for filter cake thickness of 12·7 mm compared to cased excavation but reduced by 50% for filter cake thickness of 71·1 mm‡	Significant improvement (45%) in the torsional resistance in dense sand compared to predictions
Arwanitaki et al. (2007)	Direct shear – sand on concrete	Medium sand	300 × 300	Model diaphragm wall trench	50	24	1–2 (near cement injection point), 8 (away from injection point)	Cement mortar	Wet (cast vertically)	$\delta' = 22.9°$ compared to $\phi' = 42.5°$ as measured by triaxial tests	n/a
Shakir & Zhu (2010)	Simple shear – soil on concrete	Clay and sandy clay	Ø60	Manually applying bentonite and polymer fluid to the soil surface by spatula	n/a	n/a	n/s	Cement mortar	Dry	Reduced by 23% for clay; increased by 10% for sandy clay	Reduced by 17% for clay; increased by 10% for sandy clay

* Dry, wet and pressure cast mean respectively that the concrete was cast separately from the soil, onto the soil surface under atmospheric pressure and onto the soil surface under the test pressure.

† This test satisfies the conditions of both wet and pressure cast, as the model pile was cast, cured and tested all under atmospheric pressure.

‡ Prototype dimensions.

Ideal test conditions

Although some of the discrepancies between these previous results can be explained by the different properties of the materials used, many are actually attributable to the experimental techniques employed. Before discussing the individual investigations, it is useful to consider the 'ideal' conditions for laboratory interface shear tests involving support fluids.

(*a*) Filtration: the filtration pressure is the difference between the stabilising fluid pressure and the free-field pore-water pressure. It should correspond to that which is experienced in the field, and could range from 15–20 kPa at sites with a high groundwater table to over 200 kPa at depth in a soil with a low groundwater table.

(*b*) Concrete mix: the concrete mix design should be similar to that used in the field.

(*c*) Casting method: the concrete should be cast and cured under a wet concrete pressure representative of field conditions.

(*d*) Shearing procedure: the shearing rate should correspond to that of a typical field load test. According to the Institution of Civil Engineers (2007), this may range from an effective loading rate of approximately $3.7 \times 10^{-3}\%$ design verification load (DVL) per second (or 100% DVL in 7·5 h) in a load-controlled maintained load test (MLT), to 0·01–0·02 mm/s in a displacement-controlled constant rate of penetration (CRP) test.

Evaluation of experimental techniques

If the experimental techniques used in previous studies are judged against the above criteria, it is found that none of the studies satisfies all of them. First, most studies either did not report the filtration pressure, or only used a low pressure. In two cases, the filter cake was formed by manually plastering the soil or rock surface with a bentonite paste. Such a technique cannot properly replicate the field process of cake formation, and in particular the effect of time on filter cake thickness. Second, most studies either did not specify the concrete mix or used a cement mortar. This limits the possibility of investigating the effect of surface roughness – an important effect identified by Wates & Knight (1975) and also in this research. Wates & Knight used a concrete mix containing 9 mm coarse aggregate and found that the aggregate particles protruded from the concrete surface, which increased the surface roughness. Finally, fewer than half of the previous investigators cited in Table 1 cast and cured the concrete under an applied pressure. This procedure, termed 'pressure casting' by Cheng & Haberfield (1998), is essential for the formation of a realistic interface.

In addition to the cake formation procedure and the type of concrete, differences in the test geometry are important. In the review of previous work it was found that all investigations, except for Hu *et al.*'s study which used a geotechnical centrifuge, either tested model piles in a pull-out device or used an interface shear box (Hu *et al.*, 2006). Although a centrifuge can simulate the soil stresses more realistically, it is not without problems. First, the effective sizes of the bentonite and soil particles are increased by a factor of *n*, where *n***g** is the centrifugal acceleration. This would result in the formation of a filter cake on a gravel side wall at the prototype scale (fine sand at model scale) but in fact under field conditions a filter cake can only form on the surface of a gravel bed following a rheological blocking process in the pores (Hutchinson *et al.*, 1975; Jefferis, 1992). Second, in a centrifuge model, it is difficult to use any concrete mix other than a highly plasticised cement mortar to form the model pile, because of the

constraints of centrifuge package size and the need to cast the pile in-flight. This limits the potential for modelling the important effect of aggregate protrusion.

The principal advantages of testing scale model piles in a pull-out device are that the geometry can be based on that of real piles, and the boundary stress conditions can be controlled. However, since the hydrostatic pressure in a model drill hole will be limited by the depth of the model hole, the filter cake cannot be formed under a representative pressure unless an external pressurisation device is used. Model piles tested in the laboratory typically suffer from the same size limitations as the centrifuge, and therefore use of a site-type concrete mix is not possible unless the diameter of the model pile is increased to about 150–200 mm. However, if this is done, the size of the surrounding soil bed also needs to be increased, and the assembly can no longer be accommodated in a modified triaxial cell such as the one used by Majano *et al.* (1994).

Given the limitations of model pile test methods, it was concluded that the most appropriate equipment for the investigation of interface shearing is a purpose-designed shear box, as only the materials immediately at and adjacent to the interface are modelled. This can readily be done at 1:1 scale, as opposed to testing a whole pile but at a reduced scale. The specimen can therefore be kept reasonably small, while avoiding many of the limitations of the other methods. Therefore, an interface shear box with extended capabilities was designed and used to carry out the research described in this study.

INTERFACE SHEAR BOX APPARATUS
Basic design and dimensions

Figure 2 shows a photograph of the interface shear box apparatus. In essence, the apparatus developed for this study is a medium-sized shear box, the top half of which can first be filled with pressurised support fluid and then with fresh concrete. The plan internal dimensions are 175 mm by 275 mm, the upper and lower halves of the box are respectively 100 mm and 70 mm high, and the gap between the two halves of the box is adjustable. These dimensions have no particular theoretical basis, but are simply chosen to enable the box to perform the required functions at a reasonable scale with respect to the 20 mm maximum aggregate size in the concrete (a maximum size common for piling concrete). For example, the top half of the box is large enough to act as a fluid reservoir during the filtration stage, and to accommodate a real concrete mix containing coarse aggregate.

As the concrete surface was expected to be rough due to aggregate protrusion, an interface gap height of 20 mm was used for the tests. This was achieved by the use of four steel spacer strips secured between the two halves of the box. The spacers form the middle part of a pressure vessel during the filtration and concreting stages, but can be removed to form the interface gap prior to the shearing stage of the test.

Water saturation and support fluid filtration

Figure 3(a) shows a schematic diagram of the apparatus during the water saturation and support fluid filtration stages. Unlike some previous investigations which used a separate filtration cell to form a filter cake (Tawfiq & Broughton, 1996; Thasnanipan *et al.*, 2003), the new design with a pressurised slurry box and a filtrate drain allows the filtration stage to take place with the test soil already seated in the lower half of the box. The test soil was first flushed with water with the bottom drainage port open, then flooded with either the bentonite or polymer support fluid (except for the
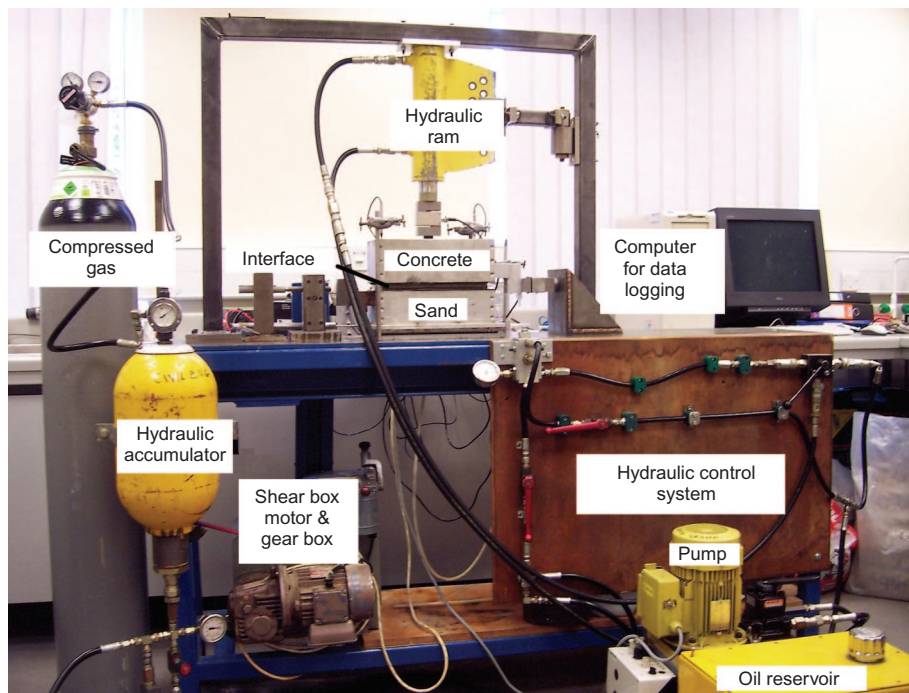
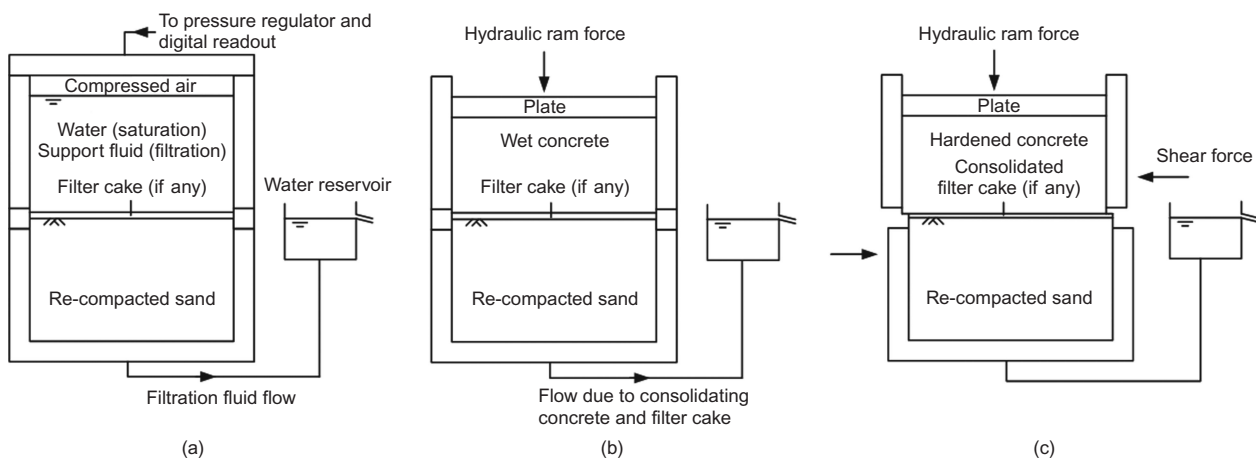**Fig. 2. Interface shear box apparatus**

Fig. 3. Schematic diagrams showing arrangement of the interface shear box during: (a) soil saturation and support fluid filtration; (b) pressure casting and curing; (c) shearing

reference water test). The support fluid was then pressurised at the desired pressure. The benefit of this approach is that it reduces the need for sample movement and the potential for soil and filter cake disturbance. Ata & O'Neill (1997, 2000) employed a similar arrangement but with a short fluid column, which provided only rather limited filtration pressures. The present design with a slurry box supplied with the support fluid by way of a hydraulic accumulator facilitated the use of much higher, more realistic pressures. This loading arrangement was also used by Peña Duarte (2007) to test highly compressible sand–foam mixtures.

Concreting

After the formation of a filter cake, the top of the cell was removed and the remaining fluid was siphoned off. The drainage port at the bottom of the box was left open, but the pipe was connected to an open-topped water reservoir in which the water level was set to match that of the top surface of the soil. This ensured that the soil remained fully satu-

rated, and also allowed free water movement in and out of the soil during the subsequent concrete curing and shearing stages. The concrete, a superplasticised, self-consolidating mix (see Table 2), was then placed on top of the filter cake (if any) and then, as described below, cured under an appropriate wet concrete pressure. This procedure simulates field conditions in which the filter cake is formed undisturbed for a period of time but, importantly, is not scoured away by the rising column of wet concrete. It was believed that scouring of the filter cake by rising concrete does occur, but Farmer *et al.* (1971) and Wates & Knight (1975) found the effect to be negligible to non-existent. The scouring that occurs during tremie casting will depend on the flow regime of the rising concrete (Littlechild & Plumbridge, 1998) and the properties of the filter cake – a material which typically grades from bulk slurry to a tight clay material in contact with the trench wall. Soft slurry may be removed by the rising concrete but stiffer material is likely to persist unless previously removed by the mechanical action of a digging or cleaning tool.

Table 2. Concrete mix design used for laboratory tests

Constituent	Quantity: kg/m³	Remarks
Cement	405	Portland cement with 30% fly ash
Coarse 20 mm aggregate	668	10–20 mm Thames Valley uncrushed aggregate
Coarse 10 mm aggregate	316	5–10 mm Thames Valley uncrushed aggregate
Fine aggregate	839	0–5 mm sharp sand
Water	162	Water/cement ratio = 0·4
Admixture	3	Superplasticiser
Density	2393	–

Concrete pressurisation and curing

Figure 3(b) is a diagram of the apparatus during the concrete pressurisation and curing stage. As shown, the normal pressure was provided by a hydraulic ram with the capacity to apply up to 800 kPa. This is sufficient to replicate most field situations, as for a 30 m deep fluid-supported excavation the wet concrete pressure should only be about 450 kPa, according to the prediction equation given by Lings *et al.* (1994). After the required curing period, the ram pressure could be adjusted to provide the required normal stress for the shearing tests. However, for the tests reported in this study the ram pressure was left unchanged, effectively assuming that the full concrete pressure was 'locked in' after the curing process. In the field, the validity of this assumption will depend, among other factors, on the degree of saturation of the soil surrounding the pile. If the soil is fully saturated, the pile concrete will expand during curing due to water absorption and the contact pressure will increase; the opposite will be true for soil holding little water in the pores (Lam & Jefferis, 2012). In this study, the concrete was cured inside the box at a constant room temperature of 20°C. The curing period was typically 7 days except for one test where the curing period was 3 days.

Shearing

Figure 3(c) is a diagram of the apparatus during the shearing stage. After the specified concrete curing period, the spacers between the top and bottom halves of the box were removed to form the interface gap. Any filter cake that was formed at the interface during the filtration stage could now be seen from the side. For the shearing test the lower half of the box was displaced at a constant rate of 2 mm/min (0·033 mm/s) until a maximum displacement of 15 mm was reached. This shearing rate is similar to that of a typical displacement-controlled (CRP) field test (Institution of Civil Engineers, 2007). At this rate, coarse-grained soils in the lower box will remain fully drained, but any bentonite filter cake that is present at the interface will be sheared undrained due to its significantly lower permeability (in the order of 10^{-10} m/s according to Jefferis, 1992). It may be noted that Henry *et al.* (1998) used a shearing rate of 0·0025 mm/min for their tests in an effort to ensure complete drainage of both the soil and the filter cake. The present tests could have been carried out at a similar rate, but there would have been a risk of drying of the filter cake as its outer surface was exposed by the removal of the spacers prior to shearing.

TEST CONDITIONS AND MATERIAL PROPERTIES
Test conditions

For interface shear tests involving support fluids, no one set of test conditions applies to all cases since the material properties and pressures (pore water, support fluid, wet concrete) are all site specific. For this study, a site at Stratford in East London was used as a representative UK case. This site is located 0·5 km south-east of the Stratford International station and bored piles were installed down to 27 m below ground level. The ground conditions consist of 6·5 m of made ground, 13 m of Lambeth Clay and at least 14 m of Thanet Sand (Lam *et al.*, 2010a). The groundwater table in the deep aquifer was reduced to about 30·5 m below ground level due to dewatering at the nearby Stratford International station box. The pore-water pressure in the Thanet Sand, over the length of the pile, was therefore zero. The bentonite fluid pressure within the pile bore was hydrostatic and was calculated using a fresh slurry density of 1·02 g/cm³. The wet concrete pressure was calculated using the method proposed by Lings *et al.* (1994), in which the concrete pressure is assumed to have a simple bilinear profile and the point of the transition, known as the critical depth, is taken as one-third of the excavation depth. The average bentonite and concrete pressures for the pile section in the Thanet Sand layer were estimated to be 230 and 360 kPa respectively. These values were used as the filtration and concrete curing pressures in the laboratory tests.

Concrete properties

Details of the concrete mix used for the tests are given in Table 2. This is an actual piling concrete mix as developed by a UK concrete supplier, containing a superplasticiser and designed to be self-consolidating. The maximum aggregate size is 20 mm, which is typical for high-workability foundation concrete. The target slump of this mix is 175 mm (slump class S4) and the 28-day cube strength is about 50 MPa.

Thanet Sand properties

Samples of Thanet Sand were taken from the East London site. Based on a sieve analysis, the D_{10}, D_{30} and D_{60} particle sizes are 0·07, 0·1 and 0·14 mm respectively. These sizes are fine enough to prevent any deep penetration of the bentonite fluid (Jefferis, 1992) but a filter cake will form as the permeability is relatively high ($\approx 2 \times 10^{-5}$ m/s, determined by the constant head method on recompacted samples) compared with that of a bentonite filter cake (in the order of 10^{-10} m/s).

The shear strength parameters of the Thanet Sand were determined using both a standard shear box (60 mm × 60 mm) and the extended interface shear box (175 mm × 275 mm) working in sand-on-sand mode by filling the upper half with sand instead of concrete. The sand was compacted using a vibrating hammer to a dry density of 1400 ± 25 kg/m³. This corresponds to a relative density of 68 ± 7% (the minimum and maximum dry densities of this sand have been

determined as 1200 and 1520 kg/m^3 respectively). The peak angle of shearing resistance, ϕ'_p, was found to be 36° over the stress range of interest. Troughton (1992) also reported the same peak angle on recompacted samples from a site in Canary Wharf. According to Ventouras & Coop (2009), the ϕ'_p of undisturbed Thanet Sand can range from 30° to 39° in triaxial tests.

Polymer fluid properties

The polymer used was a synthetic polymer system known as SlurryPro CDP supplied by KB International. First, 1 kg/m^3 of CDP base polymer was mixed at low shear for 45 min with water to which a small amount of potassium hydroxide had been added, which raised the pH to 11. LA-1 thickener at a quantity of 0·08 kg/m^3 was then added to improve the viscosity of the fluid further. Other additives such as weighting and fluid loss control agents are available for challenging ground conditions but were not used in this study. The properties of the prepared fluid were determined using the test methods prescribed by the Federation of Piling Specialists (2006) and the results are given in Table 3. There are currently no commonly accepted compliance values for polymers, so no comparison is made between the properties of bentonite slurry and the polymer fluid, although it may be noted that the polymer fluid did not form a filter cake and had a much higher fluid loss than the bentonite slurry. Discussion of the stabilisation mechanism of polymers, which Lam et al. (2010b) and Jefferis et al. (2011) explain is not by filter cake formation, is beyond the scope of this paper.

Bentonite fluid properties

The bentonite used was a sodium-activated bentonite known as Berkbent 163, a civil engineering grade product supplied by Tolsa UK and commonly used for foundation projects in the UK (e.g. Suckling et al., 2011). It was mixed with water at 40 kg/m^3 using a 5000 r/min laboratory mixer for 30 min, allowed to hydrate for 24 h, and then briefly remixed before use. The properties of the bentonite slurry were determined using the same methods as for the polymer fluid, and the results are included in Table 3. It can be seen that its properties fulfil all the requirements of the Institution of Civil Engineers (2007) specification. The thixotropic property of this bentonite slurry can be found in the paper by Jefferis & Lam (2012).

Bentonite filter cake properties

To aid the interpretation of the interface shear test results, the properties of the bentonite filter cake under the simulated field conditions of the East London site were studied. Filter cakes were formed in a standard filter press (American

Petroleum Institute, 2003) under the chosen filtration pressure of 230 kPa and then consolidated uniformly under the estimated concrete pressure of 360 kPa. At the end of each stage, the thickness was measured and this was repeated over a range of the filtration times. The results are shown in Fig. 4. As expected, during the filtration stage, the thickness of the filter cake increased linearly with the square root of time (Nash, 1974) and the thickness reduced significantly when consolidated after the filtration stage. The consolidated cake thickness is of particular relevance to the shear box results.

INTERFACE SHEAR TEST RESULTS AND DISCUSSION

A total of nine interface shear tests were carried out; details are given in Table 4. The first test was a reference test performed on a water-saturated sample without any support fluid filtration. The remaining eight tests were carried out using either polymer or bentonite fluids under various test conditions. The objectives of the test programme were to investigate: (a) the effects of polymer fluid and of concrete curing time; (b) the effects of bentonite fluid following various filtration times; (c) the effect of concrete aggregate protrusion through bentonite filter cakes.

Effect of polymer fluid

Figure 5 shows the shear stress–displacement curves for Tests 1 to 3 (with water and polymer at two concrete ages) as set out in Table 4. It can be seen that the three curves have similar peak values, although the water-saturated sample shows somewhat higher initial stiffness due to its slightly higher initial dry density (1425 kg/m^3) compared with the other two samples (1389 and 1396 kg/m^3). The similar peak values shown by the polymer-saturated samples at the two
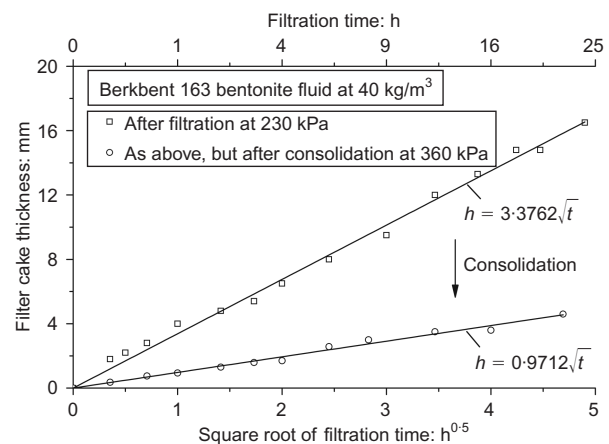

Fig. 4. Thickness of bentonite filter cakes as a function of square root of filtration time

Table 3. Properties of polymer and bentonite support fluids used for laboratory tests

Property	Test method	Units	Polymer test results	Bentonite test results	Compliance values for bentonite fluid according to Institution of Civil Engineers (2007)
Density	Mud balance	g/cm^3	1·00	1·02	<1·10
pH	Electrical pH meter	–	11·0	10·5	7–10·5
Fluid loss at 30 min*	Filter press	ml	>200	26	<30
Filter cake thickness at 30 min*	Micrometer	mm	0	2·5	<3
Marsh funnel viscosity	Marsh funnel	s	70	34	30–50
10-min gel strength	Fann viscometer	N/m^2	≈0	5·5	4–40

* Measured using the standard test pressure of 690 kPa (100 psi).

Table 4. List of interface shear box tests

Test number*	Support fluid	Filtration time: h	Max. aggregate size in concrete: mm	Age of concrete at test: days	Purpose of test
1	Water	0	20	7	Reference
2	Polymer	7·5	20	3	To investigate the effects of polymer fluid and of concrete curing time
3	Polymer	7·5	20	7	
4	Bentonite	0·5	20	7	To investigate the effect of filtration time to bentonite fluid
5	Bentonite	3	20	7	
6	Bentonite	7·5	20	7	
7	Bentonite	12	20	7	
8	Bentonite	24	20	7	
9	Bentonite	12	5†	7	To show the effect of concrete mix

* Non-interface direct shear tests on Thanet Sand and bentonite filter cake are not included in this table.
† Microconcrete does not contain any coarse aggregate larger than 5 mm.
Note: For all the tests, the sand was compacted to a dry density of 1400 ± 25 kg/m³ and the Stratford (East London) site conditions were simulated, that is a filtration pressure of 230 kPa and wet concrete pressure of 360 kPa. The normal stress used during the shear test was also maintained at 360 kPa. The shear tests were carried out at a constant displacement rate of 2 mm/min.
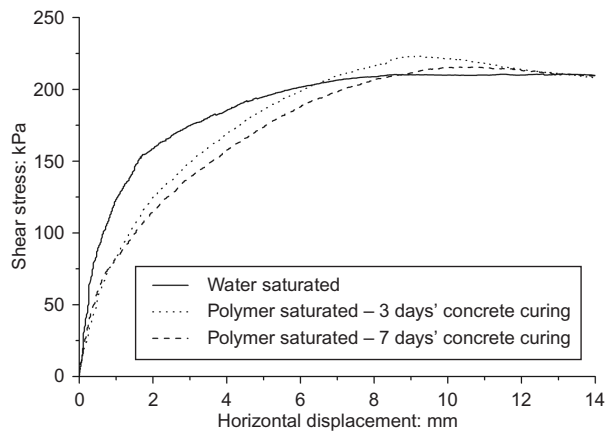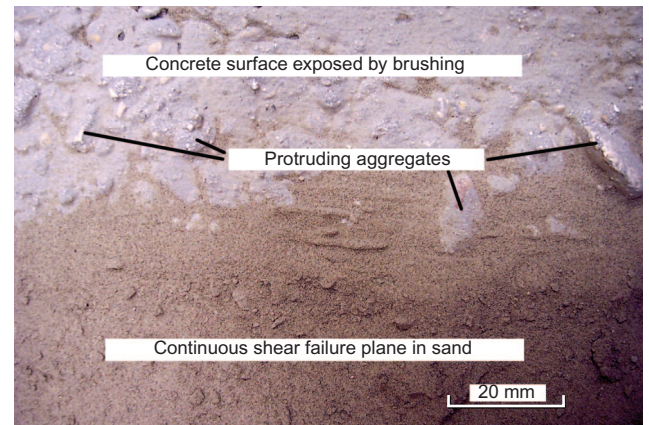

Fig. 5. Shear stress−displacement curves of Tests 1 to 3 showing the effects of polymer fluid and concrete curing time
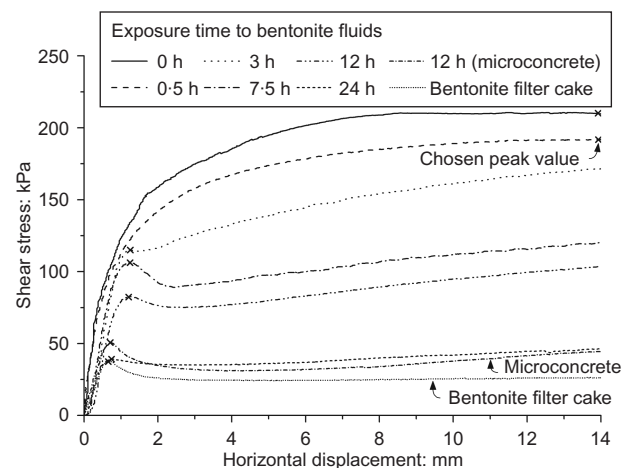

Fig. 6. Photograph showing exposed concrete surface (top) and shear failure plane (bottom) of an interface shear test using polymer support fluid

concrete ages (3 and 7 days) are as expected because by 3 days the strength of the concrete ($\approx$30 MPa) already greatly exceeds the interface shear stress. However, the findings are contrary to the results of Ata & O'Neill (2000), who found that samples cured for 7 days had higher shear strengths than those cured for 3 days. They attributed this to polymer fluid initially acting as a lubricant at the soil−cement mortar interface, but subsequently roughening the mortar surface by chemical reaction and causing the failure plane to develop in the sand rather than at the interface. However, as Tests 1 to 3 were carried out using a real piling concrete mix rather than a cement mortar, the surface was rough in all three cases due to aggregate protrusion and hence the failure plane was always in the sand. This explains why there is little difference between the three tests, but a significant difference from those of Ata & O'Neill (2000). Fig. 6 shows a photograph of the concrete surface exposed after one of the polymer tests. The shear failure surface in the soil can be seen in the bottom half of Fig. 6.

Effect of bentonite fluid
Figure 7 shows the shear stress−displacement curves from Tests 1 (water and piling concrete), 4 to 8 (bentonite and piling concrete) and 9 (bentonite and microconcrete). The curve for a pure consolidated filter cake is also included for

Fig. 7. Shear stress−displacement curves of Tests 1 and 4 to 9 showing effects of bentonite filtration time and concrete mix

comparison. It can be seen that the curve for the water-saturated sample (Test 1) forms the upper bound and the curve for the pure filter cake forms the lower bound, with the remainder plotting between them. It can also be seen that as the filtration time increases, the shearing resistance

decreases, and that after 24 h of filtration the resistance approaches that of the filter cake. It is interesting to note that Henry *et al.* (1998) reported a similar finding but after only 6 h of filtration. This difference is possibly because Henry *et al.* did not use concrete but a soil–bentonite mix, and thus the effects of aggregate protrusion may have been more limited. Lastly, it can be seen that, for Tests 5 to 8 (3 to 24 h of filtration), the shearing resistance increased again after the initial peak – a behaviour also observed by Arwanitaki *et al.* (2007) in some of their direct interface shear tests. The post-peak rise seen in this study is believed to be an experimental effect, which is due to the sand resistance being mobilised by the slightly eccentric vertical load at large shear displacements. Had the tests been conducted in ring shear mode, the post-peak rise probably would not have occurred. As it is the initial peak strength that is of interest, the post-peak rise is ignored and the 'true' inferred peak values are marked by crosses on the plot. It is worth noting that Broms & Hill (1973) also found a post-peak rise in the shaft resistance of a step-tapered pile, which was driven into a hole pre-drilled under bentonite slurry, and that no such behaviour was found in a similar pile installed in a hole formed with water. The reason for the post-peak rise in the first pile was probably the resistance of the sand being increasingly mobilised by the upper sections of the tapered pile at large displacements, at which point the shear strength of the filter cake had been overcome and was less significant.

Figure 8 summarises the peak shear stress results of Tests 1 to 8. The peak shear stresses of recompacted Thanet Sand and of the filter cake are also shown as upper and lower bounds on the plot. It can be seen that for those samples subject to bentonite filtration, the interface shear strength reduces approximately linearly with the square root of the filtration time. This is because, as previously shown in Fig. 4, the thickness of the filter cake also increases linearly with the square root of time. The practical implication is that when bentonite fluid is used to support an excavation in granular soils, the rate of filtration and hence the rate of interface strength reduction will be the greatest within the first few hours. The results in Fig. 8 may also explain why Brown (2002) found a pronounced strength reduction for bored piles (drilled shafts) which had only very limited exposure time to bentonite – a finding not expected by Brown at the time.

Comparison of the results in Fig. 8 with field data from various authors, which are plotted in Fig. 9, provides further support for the reported strength reduction effect. The shaft resistance results of Littlechild & Plumbridge (1998) and Thasnanipan *et al.* (1998) are presented as ratios of actual to
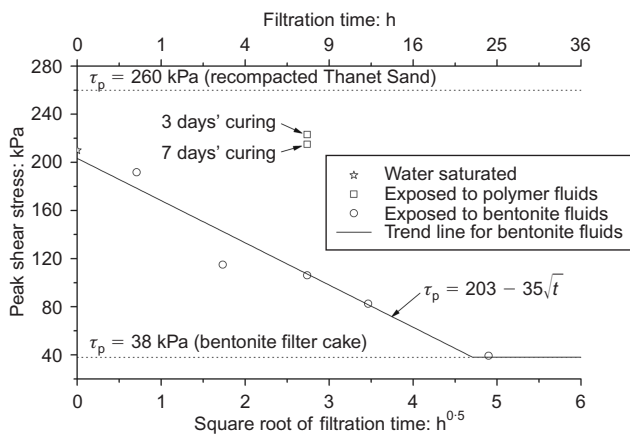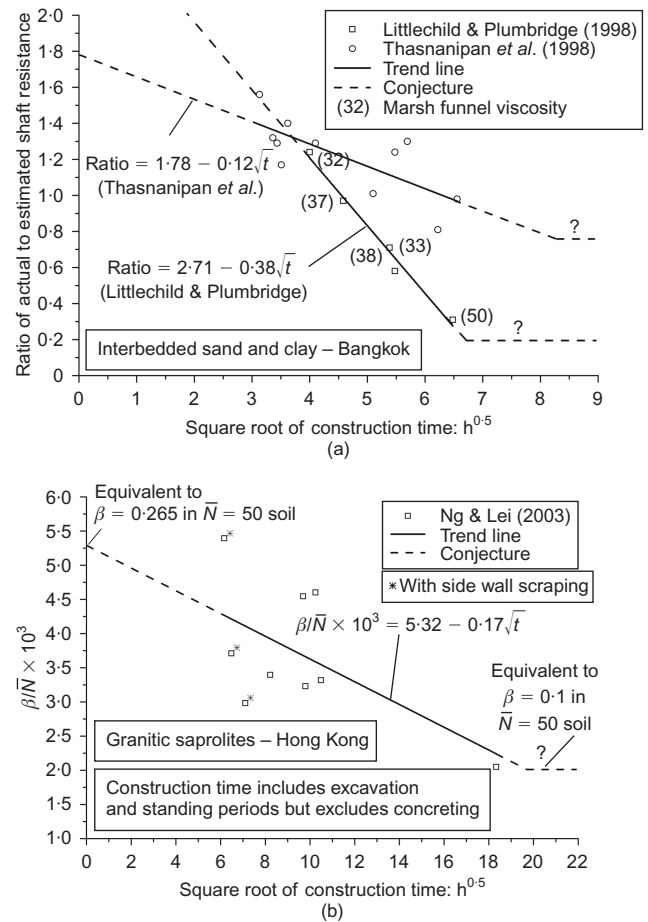

Fig. 9. Effect of construction time on shaft resistance: (a) data after Littlechild & Plumbridge (1998) and Thasnanipan *et al.* (1998); (b) data after Ng & Lei (2003)

estimated values, whereas those by Ng & Lei (2003) are given as $\beta/\bar{N}$, where β is ratio of maximum shaft resistance to mean effective vertical stress and $\bar{N}$ is the uncorrected mean standard penetration test (SPT) N value. First, it is interesting to see that the two studies in Bangkok (Fig. 9(a)) give rather different results. Littlechild & Plumbridge found that their results were influenced by both construction time and the Marsh funnel viscosity of the slurry, and hence the slope on the plot was exaggerated as the lowest viscosity (32 s) pile was built in the shortest time (16 h), while the highest viscosity (50 s) pile was built in the longest time (42 h). However, Thasnanipan *et al.*, whose results include a larger data set, did not find any correlation with slurry viscosity; hence the values are not shown on the plot. Nevertheless, it is still worth mentioning that the time required to reach the postulated minimum strength is about 40–60 h in the first two cases (Fig. 9(a)) and 380 h in the third case (Fig. 9(b)), both of them being significantly longer than the 24 h found in the present laboratory tests. This is because during a real excavation the filter cake on the side wall can be removed or made thinner by the scraping action of the digging tool (depending on the bucket and tool design), and hence a longer construction time is required for a similar thickness of bentonite to be deposited. Deliberate side-wall scraping using specifically designed tools prior to concreting can greatly reduce this effect, as noted by Randolph (2003).

Effect of aggregate protrusion
To investigate the mechanism of strength reduction induced by a bentonite filter cake, the concrete surfaces of

Fig. 8. Summary of peak shear stresses for Tests 1 to 8 using piling concrete

Tests 1 and 4 to 8 were examined after the tests. It was found that as the filtration time increased, the filter cake became thicker and fewer aggregate particles were able to penetrate through and make contact with the underlying sand, thus causing the concrete–sand contact area to decrease. To illustrate this, Fig. 10 shows photographs of the samples with 0·5, 3, 12 and 24 h of filtration. The concrete–sand contact area was assessed and is shown, as a fraction of the full area of the box (percentage), in the bottom right-hand corner of each photograph.

To confirm the theory about aggregate protrusion, an additional test (No. 9) was carried out using microconcrete, which contained no aggregate larger than 5 mm. The resulting shear stress–displacement curve is shown in Fig. 7. Comparing this curve with the curve for Test 7 (12 h filtration test for concrete made with 20 mm aggregate) indicates that the protrusion of the larger aggregate particles causes an increase of 63% in the peak interface shear stress. This effect must be purely due to the effect of surface roughness, as there were no other changes to test procedure. A visual examination of the concrete surfaces showed that when microconcrete was used, almost the entire surface was covered by the filter cake. This finding confirms that the development of shearing resistance in this type of 'sandwiched' system is controlled not only by the filter cake thickness but also by the surface roughness of the concrete, which is influenced by the maximum aggregate size in the concrete mix.

To quantify the effect of concrete–sand contact area, the peak strength values of Tests 1 and 4 to 9 were used to back-calculate a fractional concrete–sand contact area, b, using the following equation

$$A\tau_{m} = Ab\tau_{c-s} + A(1 - b)\tau_{c-b} \tag{1}$$

which can be rearranged to

$$b = \frac{\tau_{m} - \tau_{c-b}}{\tau_{c-s} - \tau_{c-b}} \tag{2}$$

where A is full the cross-sectional area of the box, τ is the interface shear stress and subscripts 'm', 'c–s' and 'c–b' denote measured, concrete-to-sand and concrete-to-bentonite respectively. Based on the previous results, τ_{c-s} and τ_{c-b} have been determined as 210 kPa and 38 kPa respectively at the applied consolidation and test pressures. Fig. 11 plots the back-calculated area ratios, b, against the actual assessed ratios. It is interesting to see that the back-calculated values are about twice the actual values, meaning that for each percentage point increase in the concrete–sand contact area, the actual effect is about two percentage points. The proportion of contact area required to achieve the full concrete-to-sand strength therefore appears to be only 50%. This is probably because the protruding aggregate not only generates friction but also induces a ploughing action in the sand. When a sufficient amount of protruding aggregate is present, the shear plane will develop entirely in the sand.
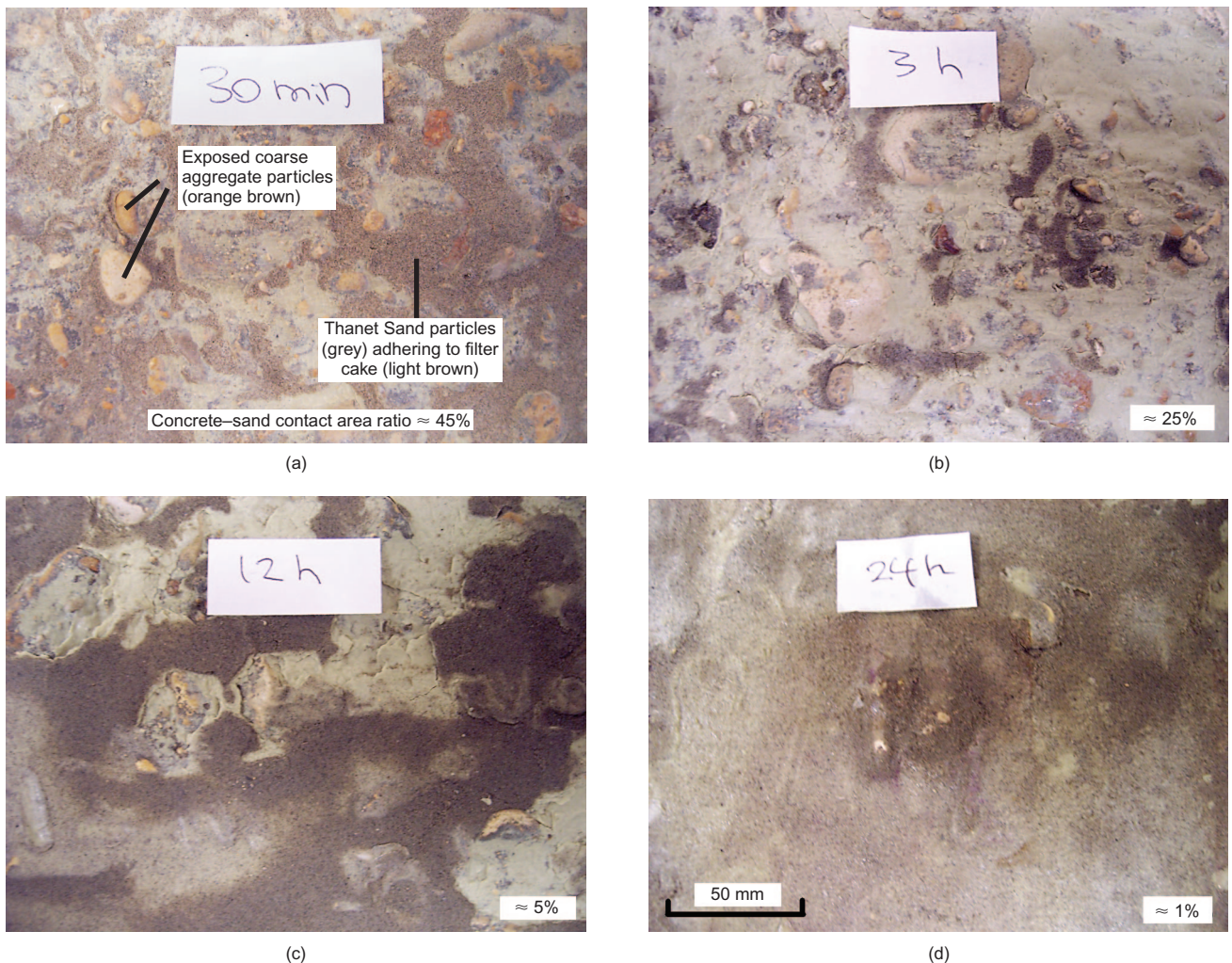

Fig. 10. Photographs of concrete surfaces showing the effect of filtration time on the fraction of the concrete–sand contact: (a) 0·5 h; (b) 3 h; (c) 12 h; (d) 24 h
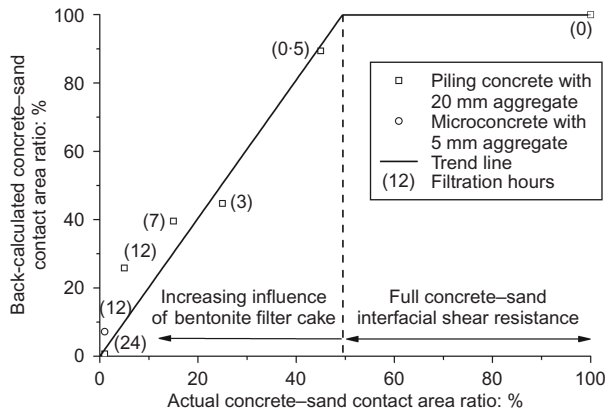
Fig. 11. Back-calculated plotted against actual fractional ratios of concrete–sand contact area

CONCLUSIONS

A series of laboratory tests has been carried out to study the effects of excavation support fluids on the interface shear strength between sand and cast in situ concrete. The main findings are as follows.

(a) The experiments show that, when compared to water, the presence of a polymer support fluid does not have a detrimental effect on the interface shear resistance, regardless of the age of the concrete. Field experience demonstrates that, unlike water, polymer fluids are capable of supporting sandy ground which would be liable to loosen and collapse in a water-filled excavation. This means that while the horizontal configuration of the shear surface is a valid means of testing the effect of polymer on shearing resistance of the sand–concrete interface, it is liable to overestimate the shearing resistance where water is used instead.

(b) Increasing the filtration time for bentonite fluid reduces the interface shear strength eventually, after about 24 h in the present experiments, to the shear strength of the filter cake. In a real excavation the interface shear strength will also depend on other factors such as the ground conditions, the properties of the bentonite slurry, the excavation tool used and whether the side wall is deliberately scraped after the completion of excavation and/or prior to cage insertion. It should be noted that in practice it may take a considerable time (typically 2–5 h) for reinforcing cages to be inserted and concreting to be completed, hence even for a scraped side wall a perfect concrete–soil bond should not be expected for a pile formed under bentonite slurry. The strength reduction relationship shown in Fig. 8 might be used to represent the worst-case indicator where little or no side scraping is undertaken or where there is a significant delay between the end of excavation or scraping and the start of concreting.

(c) A major implication of the test results is that, when bentonite is used, the degree of strength reduction depends on both the thickness of the consolidated filter cake and the amount of aggregate protruding through the filter cake into the soil. At present none of these effects is explicitly considered in common pile design approaches, although the effect of construction time is acknowledged in some textbooks (e.g. Tomlinson, 2001). The laboratory results indicate that only 50% (by area) of concrete–sand contact is required to mobilise the full concrete–sand interface shear strength. This is thought to be due to the ploughing resistance developed by aggregate protruding into the sand.

(d) In view of the above findings, it would appear that polymer-based support fluids provide an attractive alternative to conventional bentonite slurry, as they do not form a layer of soft filter cake at the pile–soil interface. The effect of different support fluids in silts and clays remains to be investigated. Based on the experience with polymer fluids so far, pile bore stability is usually not a problem provided the fluid is properly maintained on site (e.g. Jefferis & Lam, 2013). It can be expected that polymer fluids will play a much larger role in the foundation industry in the future as their use continues to increase around the world.

(e) Although this paper is focused on maximising the interface shear strength in piles, the findings will have relevance to situations where there is a need to reduce interface shear strength, such as lubrication of pipes in pipe jacking works.

NOTATION

A cross-sectional area of the interface shear box
b calculated fractional area of concrete–sand contact surface
D_x particle size at which x% of the grains are smaller
$\overline{N}$ uncorrected mean SPT N value
n centrifugal acceleration factor
ng centrifugal acceleration
β ratio of maximum shaft resistance to mean effective vertical stress
δ' interface friction angle
τ interface shear strength
τ_{c-b} interface shear strength, concrete to bentonite
τ_{c-s} interface shear strength, concrete to sand
τ_m interface shear strength, measured
ϕ'_p peak angle of shearing resistance

REFERENCES

American Petroleum Institute (2003). *Recommended practice for field testing water-based drilling fluids, API recommended practice 13B-1*, 3rd edn, ANSI/API 13B-1/ISO 10414-1. Washington DC, USA: American Petroleum Institute.

Arwanitaki, A., König, D. & Triantafyllidis, T. (2007). Zum Kontaktverhalten zwischen suspensionsgestützten Ortbetonwänden und dem anstehenden Boden. *Bautechnik* **84**, No. 11, 781–792 (in German).

Ata, A. & O'Neill, M. (1997). *Characterization of the effect of POLY-BORE polymer on the construction and performance of drilled shaft foundations. Phase II: laboratory investigations*. Houston, TX, USA: University of Houston.

Ata, A. & O'Neill, M. (2000). The physicochemical interaction between PHPA polymer slurry and cement mortar. *Geotech. Test. J.* **23**, No. 2, 225–235.

Broms, B. B. & Hill, L. (1973). Pile foundations for the Kuwait Towers. *Proceedings of the 8th international conference on soil mechanics and foundation engineering*, Moscow, vol. 2, pp. 33–38.

Brown, D. (2002). Effect of construction on axial capacity of drilled

foundations in Piedmont soils. *J. Geotech. Geoenviron. Engng* **128**, No. 12, 967–973.

Brown, D. A., Turner, J. P. & Castelli, R. J. (2010). *Drilled shafts construction procedures and LRFD design methods*, Report FHWA NHI-10–016. Washington, DC, USA: National Highway Institute, US Department of Transportation, Federal Highway Administration.

Canadian Geotechnical Society (2006). *Canadian foundation engineering manual*, 4th edn. Richmond, BC, Canada: Canadian Geotechnical Society.

Cernak, B., Dvorak, A., Hlavacek, J., Klein, K. & Petrasek, J. (1973). New approaches to problems of bearing capacity and settlement of piles. *Proceedings of the 8th international conference on soil mechanics and foundation engineering*, Moscow, vol. 2, pp. 67–74.

Cheng, F. & Haberfield, C. M. (1998). Laboratory study of the effect of drilling fluids on piles socketed into soft rock. *Proceedings of the 2nd international symposium on the geotechnics of hard soils–soft rocks*, Naples, vol. 1, pp. 93–101.

Farmer, I. W., Buckley, P. J. C. & Sliwinski, Z. (1971). The effect of bentonite on the skin friction of cast in-place piles. *Behaviour of Piles: Proceedings of conference organised by the Institution of Civil Engineers, London*, pp. 67–72. London, UK: Institution of Civil Engineers.

Federation of Piling Specialists (2006). *Bentonite support fluids in civil engineering*, 2nd edn. Kent, UK: Federation of Piling Specialists.

Geotechnical Engineering Office (2006). *Foundation design and construction*, Publication 1/2006. Hong Kong SAR, China: Geotechnical Engineering Office.

Henry, L. B., Filz, G. M. & Davidson, R. R. (1998). Formation and properties of bentonite filter cakes. *Proceedings of the ASCE annual convention: Filtration and drainage in geotechnical/geoenvironmental engineering, Boston*, ASCE Geotechnical Special Publication No. 78, pp. 69–88. Reston, VA, USA: ASCE.

Hu, Z. (2003). *Determining the optimum depth of drilled shafts subject to combined torsion and lateral loads in saturated sand from centrifuge testing*. Master's dissertation, University of Florida, Gainesville, FL, USA.

Hu, Z., McVay, M., Bloomquist, D., Herrera, R. & Lai, P. (2006). Influence of torque on lateral capacity of drilled shafts in sands. *J. Geotech. Geoenviron. Engng., ASCE* **132**, No. 4, 456–464.

Hutchinson, M. T., Daw, G. P., Shotton, P. G. & James, A. N. (1975). The properties of bentonite slurries used in diaphragm walling and their control. *Proceedings of conference on diaphragm walls and anchorages, London*, pp. 33–39. London, UK: Institution of Civil Engineers.

Institution of Civil Engineers (2007). *ICE specification for piling and embedded retaining walls*. London, UK: Thomas Telford.

Jefferis, S. A. (1992). Grouts and slurries. In *Construction materials reference book* (ed. D. K. Doran), pp. 48/1–48/24. Oxford, UK: Butterworth-Heinemann.

Jefferis, S. A. & Lam, C. (2012). Letter to the Editor: Hydraulic characteristics of bentonite cake fabricated on cutoff walls. *Clays Clay Miner.* **60**, No. 6, 557–560.

Jefferis, S. A. & Lam, C. (2013). Polymer support fluids: use and misuse of innovative fluids in geotechnical works. *Proceedings of the 18th international conference on soil mechanics and geotechnical engineering*, Paris, vol. 4, pp. 3219–3222.

Jefferis, S. A., Troughton, V. & Lam, C. (2011). Polymer systems for fluid supported excavations. In *Geotechnical issues in construction: short paper series and proceedings of 2nd conference, London*, Report No. X513, pp. 7–12. London, UK: CIRIA.

Lam, C. & Jefferis, S. A. (2012). Reply to the discussion by Fellenius on 'Critical assessment of pile modulus determination methods'. *Can. Geotech. J.* **49**, No. 5, 622–629.

Lam, C., Troughton, V., Jefferis, S. & Suckling, T. (2010a). Effect of support fluids on pile performance – a field trial in east London. *Ground Engng* **43**, No. 10, 28–31.

Lam, C., Jefferis, S. A. & Goodhue Jr, K. G. (2010b). Observations on viscosity reduction of PHPA polymer support fluids. *Proceedings of sessions of the GeoShanghai 2010 international conference, Shanghai*, ASCE Geotechnical Special Publication No. 205, pp. 184–191. Reston, VA, USA: ASCE.

Lings, M. L., Ng, C. W. W. & Nash, D. F. T. (1994). The lateral pressure of wet concrete in diaphragm wall panels cast under bentonite. *Proc. Instn. Civ. Engrs – Geotech. Engng* **107**, No. 3, 163–172.

Littlechild, B. & Plumbridge, G. (1998). Effects of construction technique on the behaviour of plain bored cast in situ piles constructed under drilling slurry. *Proceedings of the 7th international conference exhibition on piling and deep foundations*, Vienna, pp. 1·6·1–1·6·8.

Majano, R. E. & O'Neill, M. W. (1993). *Effect of mineral and polymer slurries on perimeter load transfer in drilled shafts*, Report UHCE-93-1. Houston, TX, USA: University of Houston.

Majano, R. E., O'Neill, M. W. & Hassan, K. M. (1994). Perimeter load transfer in model drilled shafts formed under slurry. *J. Geotech. Engng.* **120**, No. 12, 2136–2154.

Nash, K. L. (1974). Stability of trenches filled with fluids. *J. Constr. Div. ASCE* **100**, No. 4, 533–542.

Ng, C. W. W. & Lei, G. H. (2003). Performance of long rectangular barrettes in granitic saprolites. *J. Geotech. Geoenviron. Engng., ASCE* **129**, No. 8, 685–696.

Peña Duarte, M. A. (2007). *Foam as a soil conditioner in tunnelling: physical and mechanical properties of conditioned sands*. DPhil thesis, University of Oxford, UK.

Randolph, M. F. (2003). Science and empiricism in pile foundation design. *Géotechnique* **53**, No. 10, 847–875, http://dx.doi.org/10.1680/geot.2003.53.10.847.

Shakir, R. R. & Zhu, J. (2010). An examination of the mechanical interaction of drilling slurries at the soil-concrete contact. *J. Zhejiang Univ. – Sci. A (Appl. Sci. Engng)* **11**, No. 4, 294–304.

Suckling, T. P., Lam, C., Jefferis, S. A. & Pantling, C. J. (2011). Evaluation of causes of bleeding of free water from a bentonite slurry. *Proceedings of the 36th annual conference on deep foundations*, Boston, pp. 401–407.

Tawfiq, K. & Broughton, H. L. III (1996). Friction characteristics of cohesionless soil penetrated by polymer and mineral slurries. *Proceedings of the 4th materials engineering conference*, Washington, DC, vol. 2, pp. 1170–1178.

Thasnanipan, N., Baskaran, G. & Anwar, M. A. (1998). Effect of construction time and bentonite viscosity on shaft capacity of bored piles. *Deep foundations on bored and auger piles: Proceedings of the 3rd international geotechnical seminar on bored and auger piles, Ghent, Belgium* (ed. W. F. Van Impe), pp. 171–177. Rotterdam, the Netherlands: Balkema.

Thasnanipan, N., Aye, Z. Z. & Boonyarak, T. (2003). Behaviour of polymer-based slurry for deep-seated bored piles in multi-layered soil of Bangkok. In *Deep foundations on bored and auger piles: Proceedings of the 4th international geotechnical seminar on bored and auger piles, Ghent, Belgium* (ed. W. F. Van Impe), pp. 267–274. Rotterdam, the Netherlands: Millpress.

Tomlinson, M. J. (2001). *Foundation design and construction*, 7th edn. Harlow, UK: Pearson Education.

Troughton, V. M. (1992). The design and performance of foundations for the Canary Wharf development in London Docklands. *Géotechnique* **42**, No. 3, 381–393, http://dx.doi.org/10.1680/geot.1992.42.3.381.

Veder, C. (1953). Method for the construction of impermeable diaphragms at great depth by means of thixotropic muds. *Proceedings of the 3rd international conference on soil mechanics and foundation engineering*, Zurich, vol. 2, pp. 91–94 (in French).

Ventouras, K. & Coop, M. R. (2009). On the behaviour of Thanet Sand: an example of an uncemented natural sand. *Géotechnique* **59**, No. 9, 727–738, http://dx.doi.org/10.1680/geot.7.00061.

Wates, J. A. & Knight, K. (1975). The effect of bentonite on the skin friction in cast-in-place piles and diaphragm walls. *Proceedings of the 6th Regional Conference for Africa on Soil Mechanics and Foundation Engineering*, Durban, vol. 1, pp. 183–188.