The undrained shear strength anisotropy of four Jurassic to Eocene stiff clays

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The shear strength of heavily overconsolidated, stiff-to-hard plastic clays is crucial to their stability and also influential on the ground movements they develop in many geotechnical engineering applications. This paper considers the shear strength anisotropy of the London, Gault, Kimmeridge and Oxford clays through advanced hollow cylinder experiments on multiple high-quality samples taken at similar depths from inland sites where the geotechnical profiles have been established by comprehensive laboratory and in situ testing. Suites of undrained tests are reported, which loaded specimens from their in situ stress states to reach ultimate failure at pre-defined final major principal stress axis orientations defined in the vertical plane, while also controlling or monitoring the intermediate principal stress ratio, b . Both stress path and simple shear tests were undertaken with the hollow cylinder apparatus, which offers key advantages over conventional simple shear equipment. The interpretation reveals patterns of marked shear strength anisotropy that impact significantly on numerous geotechnical engineering applications.

KEYWORDS: anisotropy; clays; laboratory tests; shear strength; stress path

INTRODUCTION

Much of the UK's civil engineering infrastructure and several major cities are built on stiff, overconsolidated, geologically aged, high-overconsolidation-ratio (OCR) plastic clays, which can be remarkably brittle in shear. Slope instability is frequently encountered in such clays, particularly in coastal areas. Progressive failure is common, in which the shear surface microstructure reorients towards low-shear-strength, residual conditions; see Morgenstern & Tchalenko (1967) or Potts *et al.* (1997). These features also render stability assessment difficult with deep excavations, foundations, tunnels, cut slopes, embankments and dams; see, for example, Potts *et al.* (1990). Representative information on shear strength behaviour from peak to ultimate conditions is vital to current UK developments that will cross high-OCR, stiff, plastic clay strata, including the main north–south high-speed rail line (HS2) and at least one new nuclear power station.

Natural stiff clays often manifest directionally oriented macro- to microstructures (Hight *et al.*, 2007; Hosseini Kamal *et al.*, 2014). Tectonic shears and fissure discontinuities can affect their mass shear strength profoundly. Assuming isotropy in shear strength and assuming that peak triaxial compression shear strengths apply in all cases could be misleading in clays that are either anisotropic or brittle when sheared.

Shear strength and its anisotropy can also affect the displacements developed in engineering works where ground movement considerations dominate design. Geotechnical structures can develop zones of contained failure under

fully stable, small-displacement conditions that, together with the often anisotropic non-linear stiffness of the soil mass, control the overall patterns and scales of ground movements (Lo, 1965; Jardine *et al.*, 1986; Addenbrooke *et al.*, 1997; Potts & Zdravkovic, 2001; Jardine *et al.*, 2005).

Hight & Jardine (1993) demonstrated the importance of: (a) high-quality sampling; (b) addressing natural structure; and (c) applying appropriate effective stress states when characterising stiff clay behaviour. Fig. 1(a) reproduces the shear failure envelopes they established from large numbers of K_0 reconsolidated undrained triaxial compression tests on rotary and block samples of London clay, showing an en-echelon family of curved surfaces that expanded upwards with sample depth. Samples taken from any individual depth showed marked reductions in the stress ratio t/s' at peak shear strength when consolidated to effective stresses greater than those acting in situ and the opposite when swelled to lower stresses. The simple $c' = 0$, $\phi' = 19^\circ$ criterion provided a lower bound for all samples that failed on pre-existing planar fissure discontinuities aligned at angles near to the optimum $45^\circ - \phi'/2$ orientation from the vertical at which t/σ'_n is greatest under active triaxial conditions. Gasparre *et al.* (2007a) report similar patterns from triaxial tests conducted on high-quality London clay specimens from a wide depth range at the Heathrow T5 site. After normalising the peak $q-p'$ stresses by the Hvorslev equivalent mean effective stresses p_c^* , as defined by oedometer tests performed on reconstituted specimens from similar depths, they concluded that the clay's state boundary surfaces varied with depth in the similar en-echelon sequence reproduced in Fig. 1(b).

The Oxford, Kimmeridge, Gault and London clay formations considered in this study comprise high-OCR clays deposited between the Jurassic and the Eocene in broadly similar marine environments. Sampling depths around 10 m below the clays' current eroded surfaces were selected at the inland sites identified in Fig. 2 to reduce the potential local effects of prior tectonic disturbance, post-depositional weathering, glacial or tree action. High-quality block and Geobor-S rotary cored samples were taken at each site. Experiments on Gault, Kimmeridge and Oxford clays formed part of the integrated study detailed by Wilkinson

Manuscript received 13 October 2015; revised manuscript accepted 11 January 2017.

Discussion on this paper is welcomed by the editor.

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Fig. 1. (a) Shear strength envelopes for London Clay derived from triaxial compression (TC) tests, after Hight & Jardine (1993). The $c' = 0$, $\phi' = 19^\circ$ line represents the fissure shear strength. **(b)** Normalised shear strength envelopes for London Clay derived from TC tests, after Gasparre *et al.* (2007a)

(2011), Brosse (2012) and Hosseini Kamal (2012); Hosseini Kamal *et al.* (2014) presented full geotechnical profiles for each site and reported on the in situ testing undertaken at each location. The London Clay hollow cylinder apparatus (HCA) experiments were performed as part of the Heathrow Terminal 5 study; Hight *et al.* (2007) provided full details of this site's geotechnical profile.

Tables 1 and 2 summarise key aspects of the four clays' descriptions, micro- and meso-fabrics, index properties and estimated in situ effective stresses. None of the clays presented macrostructural features such as clearly visible laminations or tectonic shear bands. Their mesostructures include features such as fissures or bedding planes that are visible by eye, yet small enough for their effects to be homogenised and perceived as characteristics of representative soil 'elements'. Also listed are the peak triaxial compressive undrained shear strengths, S_u , obtained from carefully K_0 reconsolidated, 100 mm dia. specimens and residual angles of shearing resistance, as determined in ring shear experiments. The Oxford clay samples had the clearest bedding features, but no fissures, and developed markedly higher triaxial compression shear strengths and post-peak brittleness. The peak triaxial shear strengths of natural samples of the other clays were

Fig. 2. Location map of the studied material sampling sites, modified from Brosse (2012). The Kimmeridge Clay outcrop is a thin band bordering the south of the Oxford Clay outcrop so that the two have been combined on the map

strongly affected by their greater fissure intensity. All four soils are markedly brittle in shear. Hosseini Kamal *et al.* (2014) discuss the sample sizes required to conduct tests with representative element volumes (REVs) and report, perhaps surprisingly, no clear correlation between the clays' large strain behaviours and their variable ages or maximum prior burial depths. After removing as far as possible the potential effects of other depositional and post-depositional factors, they concluded that mesostructure was the most important determinant of the natural clays' large-scale yielding behaviour under triaxial conditions.

This paper explores the four clays' shear strength anisotropy as established by two types of HCA experiments on high-quality natural specimens. The first type loaded specimens from in situ stresses to failure at a range of final major principal stress axis orientations (defined as α the orientation of the σ_1 axis to the vertical), while also maintaining fixed intermediate principal stress conditions. The second comprised simple shear HCA tests on samples tested from in situ and other effective stress conditions. Nishimura (2006), Nishimura *et al.* (2007, 2008) and Brosse (2012) provide further details of the experiments and interpretation, while Brosse *et al.* (2016, 2017) report on the four clays' highly non-linear, pressure-dependent and anisotropic pre-failure stiffness characteristics.

APPARATUS AND PROCEDURES

Principles

Hollow cylinder apparatus tests can be employed to investigate the directional dependency, or anisotropy, of soil properties by varying independently the axial force (F), torque (T) and inner and outer cell pressures (p_i and p_o , respectively) applied to test specimens. These boundary changes allow operators to control the average vertical, radial, circumferential

Table 1. Soil description and estimated in situ stresses

Soil	Age: million years ago	In situ stresses			Microstructure†	Mesostructure‡
		K_0^*	p'_0 : kPa	$\sigma'_{v0} - \sigma'_{h0}$: kPa		
Oxford Clay	Upper Jurassic 161–156	3.2 (3–7)	250	–223	Strong horizontal orientation of particles and shells	No significant fissuring but highly bedded. Horizontal shell beds
Kimmeridge Clay	Upper Jurassic 156–151	1.8 (1.5–3)	186	–97	No strong preferred orientation due to presence of larger silt particles	Closely spaced fissuring (2–5 cm spacing) sub-horizontal and 40°–60° to vertical
Gault Clay	Lower Cretaceous 112–99	1.8 (1.5–3)	163	–85	No strong preferred orientation due to presence of larger micro-fossils	Closely spaced fissuring (spacing of 2–5 cm) sub-horizontal and sub-vertical with frequent zones of intense fissuring. Presence of nodules
London Clay (Unit B2c)	Eocene 56–49	1.8 (1.5–3)	323	–165	Moderate particle orientation	Highly fissured (spacing typically 2–10 cm) in sub-vertical and sub-horizontal directions

*The estimated value is given first. The range obtained from a variety of methods is indicated in brackets. Note that in the case of Oxford Clay, the estimated value could not be reached without over-straining the sample and so a K of 1.8 was used ultimately (see Table 4).

†Wilkinson (2011)

‡Hosseini Kamal *et al.* (2014)

and torsional shear stresses σ_z , σ_r , σ_θ and $\tau_{z\theta}$, as illustrated in Fig. 3. Provided that a uniform pore pressure u can be maintained in fully drained tests, the effective stresses σ'_z , σ'_r and σ'_θ may also be varied as required. Hight *et al.* (1983) explained how the control is achieved physically and derived the expressions listed in Table 3 to relate the average sample strains and total stresses to the boundary measurements. The latter, which have been widely adopted since then as standard protocols for HCA operation and interpretation, have been applied in this study. HCA systems maintain $\tau_{zr} = \tau_{r\theta} = 0$ and provide four degrees of freedom that offer independent control of the p , q , α (or $\alpha_{d\sigma}$), and b stress parameters defined as

$$p = \frac{(\sigma_1 + \sigma_2 + \sigma_3)}{3} \quad (1)$$

$$q = \sigma_1 - \sigma_3 \quad (2)$$

$$\alpha = \frac{1}{2} \tan^{-1} \left(\frac{2\tau_{z\theta}}{\sigma_z - \sigma_\theta} \right) \quad (3)$$

$$\alpha_{d\sigma} = \frac{1}{2} \tan^{-1} \left(\frac{2d\tau_{z\theta}}{d\sigma_z - d\sigma_\theta} \right) \quad (4)$$

$$b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} \quad (5)$$

α is the angle between the direction of the major principal stress σ_1 and the vertical, whereas $\alpha_{d\sigma}$ is the direction of the axis of the major principal stress increment. The effective stresses (denoted by ') are obtained simply by subtracting the measured or imposed pore pressure, u . Establishing the effects of α alone on material behaviour provides a direct way of investigating shear strength anisotropy. Variations in the $\tau_{z\theta} - (\sigma_z - \sigma_\theta)/2$ plane can be studied by conducting multiple tests that adopt fixed intermediate principal stress ratios b between 0 and 1.

Equipment

The two computer-controlled HCA systems illustrated in Figs 4 and 5 and described by Nishimura *et al.* (2007), Minh *et al.* (2011) and Brosse (2012) were employed. Both can apply mixed stress and strain control. The nominal outer diameters, inner diameters and heights of their test specimens were 70 mm, 38 mm and 170–190 mm for the Imperial College resonant column hollow cylinder apparatus (ICRCHCA) and 100 mm, 60 mm and 200 mm for the Imperial College hollow cylinder apparatus mark II (ICHCA II). Hosseini Kamal *et al.* (2014) discuss the impact of the samples' meso-fabric and discontinuity spacings on laboratory shear strength measurements, showing that the HCA specimens were sufficiently large to provide broadly REV's.

The ICRCHCA is equipped with a Hardin torsional resonant column oscillator to obtain dynamic $G_{z\theta}$ shear moduli. Sample deformation was measured globally, apart from the semi-local (platen-to-platen) torsional rotations, which were sensed with a proximity transducer system. The ICHCA II employed a full suite of high-resolution local transducers. Axial displacements and torsional rotations were measured with enhanced electro-level inclinometers, while changes of the inner and outer radii were measured with a linear variable differential transducer (LVDT) and three proximity transducers, respectively. The average stresses were calculated according to Table 3 from the current sample dimensions, as updated from global displacement

Table 2. Basic index and strength properties

Soil	Water content: %	Plastic limit: %	Liquid limit: %	Plasticity index	S_u : kPa	Residual ϕ' : degrees
Oxford clay	24–27	32–35	64–67	30–34	200–300	10
Kimmeridge clay	16–21	18–22	46–55	28–33	80–100	7
Gault clay	28–30	28–31	71–77	40–46	70–90	10
London clay (Unit B2c)	23–25	26–27	68–70	41–44	120–170	12

Fig. 3. Schematic diagram of the stress and strain states imposed in a hollow cylinder sample

Table 3. Average stresses and strains obtained from boundary measurements in an HCA; after Hight *et al.* (1983)

	Stresses	Strains
Vertical	$\sigma_z = \frac{F}{\pi(r_o^2 - r_i^2)} + \frac{p_o r_o^2 - p_i r_i^2}{r_o^2 - r_i^2}$	$\epsilon_z = -\frac{\Delta H}{H}$
Radial	$\sigma_r = \frac{p_o r_o + p_i r_i}{r_o + r_i}$	$\epsilon_r = -\frac{\Delta r_o - \Delta r_i}{r_o - r_i}$
Circumferential	$\sigma_\theta = \frac{p_o r_o - p_i r_i}{r_o - r_i}$	$\epsilon_\theta = -\frac{\Delta r_o + \Delta r_i}{r_o + r_i}$
Shear	$\tau_{z\theta} = \frac{3T}{2\pi(r_o^3 - r_i^3)}$	$\gamma_{z\theta} = \frac{2\Delta\theta(r_o^3 - r_i^3)}{3H(r_o^2 - r_i^2)}$
Major principal	$\sigma_1 = \frac{\sigma_z + \sigma_\theta}{2} + \sqrt{\left(\frac{\sigma_z - \sigma_\theta}{2}\right)^2 + (\tau_{z\theta})^2}$	$\epsilon_1 = \frac{\epsilon_z + \epsilon_\theta}{2} + \sqrt{\left(\frac{\epsilon_z - \epsilon_\theta}{2}\right)^2 + \left(\frac{\gamma_{z\theta}}{2}\right)^2}$
Intermediate principal	$\sigma_2 = \sigma_r$	$\epsilon_2 = \epsilon_r$
Minor principal	$\sigma_3 = \frac{\sigma_z + \sigma_\theta}{2} - \sqrt{\left(\frac{\sigma_z - \sigma_\theta}{2}\right)^2 + (\tau_{z\theta})^2}$	$\epsilon_3 = \frac{\epsilon_z + \epsilon_\theta}{2} - \sqrt{\left(\frac{\epsilon_z - \epsilon_\theta}{2}\right)^2 + \left(\frac{\gamma_{z\theta}}{2}\right)^2}$

H , specimen height; r_i and r_o , inner and outer radii; F , forces measured by the load cell placed beneath the specimen, minus half the specimen's submerged weight; $\Delta\theta$ = relative torsional angle between the specimen top and bottom. The other variables are defined in Fig. 3.

measurements. The average strains reported for individual test stages were calculated with respect to the specimen dimensions applying at the beginning of that stage. Careful calibration of the employed pressure and force transducers

against precise comparators over adequate ranges resulted in measurements of stresses with accuracies between 0.5 and 4.0 kPa (and far finer resolutions), depending on the individual sensors deployed. The displacement measuring

Fig. 4. Schematic diagram of the Imperial College Resonant column hollow cylinder apparatus (ICRCHCA) (Nishimura, 2006; Brosse, 2012)

system calibrations allowed measurement of strains with accuracies in the order of 10^{-4} or finer.

Specimen preparation

The HCA specimens were prepared by trimming cylinders to the specified outer dimensions with a wire saw in a rotary soil lathe. The inner cylinder void was then formed by jacketing the sample in an aluminium mould and using a metal working lathe to advance six drill bits of gradually increasing diameters to achieve the desired inner diameter with minimal disturbance. The specimens were carefully installed with both internal and external latex membranes and systems of surface filter paper drains in place, and were set on the edges of shear vanes anchored into the HCAs' sintered bronze vertical platens. The latter ensured that sufficient traction could be applied during torsional shearing to failure.

Test programme

Figure 6 illustrates the general scheme of the stress paths applied in the undrained HCA tests. The influences of b and α were studied independently in the London clay HCA experiments by Nishimura (2006) and Anh Minh (2006), who investigated a wide range of depths at their single test site. Nishimura (2006) and Brosse (2007) also performed undrained HCA simple shear tests on London clay specimens from a range of depths and, at one depth, at a spread of effective stress levels; Nishimura *et al.* (2008) report a summary of the results. Brosse's (2012) work covered three different sites and clays, Oxford, Kimmeridge and Gault, focusing on a narrow depth range and on the effect of α by maintaining $b = 0.5$ in all the 'stress-path' tests. Undrained HCA simple shear tests were also undertaken on each clay.

This paper reports 32 successful HCA tests, each of which took around one month to complete. Tables 4–7 specify each

Fig. 5. Schematic diagram of the Imperial College hollow cylinder apparatus mark II (ICHCA II) (modified after Nishimura *et al.* (2007)) (PMMA, poly(methyl methacrylate))

test's code, sampling depth, reconsolidation effective stresses and shearing effective stress path. The tabulated Gault, Kimmeridge and Oxford clay sites were all free from any other type of overburden. However, the London Clay at Heathrow at T5 had been overlain by 6 m of Quaternary Thames River Terrace gravels that had been removed at the block sampling location some 70 years before sampling, but was still present at the rotary coring site. The reported London clay block specimens were taken at ≈ 10 m depth, while the rotary samples from the same unit were retrieved from 16 m below current ground level. To maintain compatibility, both types of specimen were tested at the same effective stress levels, which represent those applying currently at the rotary coring site and 70 years earlier at the block sampling location. The London clay specimens were therefore tested from higher effective stresses than the three other clays. The implications and importance of reproducing the in situ effective stress levels in the tests in assessing the strength anisotropy are discussed later.

Test procedures

All tests started by applying a cell pressure greater than the estimated in situ mean effective stress to induce a positive pore pressure once full contact was made between the sample and the platens. Saturation stages ensued, in which back pressures were increased to attain acceptable initial undrained pore pressure response ratios. It can be hard to obtain high B values with the high bulk stiffness ($K' > 100$ MPa) clays tested, but initial B values exceeding 0.9 were obtained in most of the specimens reported and only marginally lower ratios in a few cases. Subsequently, all specimens were reconsolidated under high back pressures (200–400 kPa) to effective stress states reflecting those estimated in situ and further saturation took place over these extended test stages. All but four of the specimens considered in this paper were reconsolidated to anisotropic stress conditions.

Fig. 6. Schematic diagram of the undrained stress paths followed during shearing at constant- $\alpha_{d\sigma}$ from in situ stresses in the HCA test series

Table 4. Summary of HCA tests for Oxford clay series

Test reference	Sample type	Specimen depth: mBGL	Type of test	Initial stresses		$\alpha_{d\sigma}$: degrees	$b^\dagger$	Stress state at peak q		
				p'_0 : kPa	$\sigma'_{v0} - \sigma'_{h0}$: kPa			p'_f : kPa	q_f : kPa	α_f : degrees
OA0005	Block	10	Constant $\alpha_{d\sigma}$	250	-130	0	0.5	202	350	0
OA2305	Block	10	Constant $\alpha_{d\sigma}$	250	-130	15	0.5	151	290	22
OA4505	Block	10	Constant $\alpha_{d\sigma}$	252	-125	30	0.5	213	255	43
OA6705	Block	10	Constant $\alpha_{d\sigma}$	250	-130	57	0.5	250	290	69
OA9005	Block	10	Constant $\alpha_{d\sigma}$	252	-125	90	0.5	371	520	90
OA0005* ‡	Block	10	Constant $\alpha_{d\sigma}$	252	-125	0	0.5	203	318	0
OASS	Block	10	Simple shear	250	-130	—	0.58	218	224	57

All tests were performed in the ICRCHCA apparatus.

† For simple shear tests the b value at peak q is given.

‡ Test OA0005* reproduced OA0005 conditions with a different specimen.

Table 5. Summary of HCA tests for Kimmeridge clay series

Test reference	Sample type	Specimen depth: mBGL	Type of test	Initial stresses		$\alpha_{d\sigma}$: degrees	b^*	Stress state at peak q		
				p'_0 : kPa	$\sigma'_{v0} - \sigma'_{h0}$: kPa			p'_f : kPa	q_f : kPa	α_f : degrees
KA0005	Rotary core	11.4	Constant $\alpha_{d\sigma}$	186	-97	0	0.5	172	199	0
KA2305	Rotary core	11.2	Constant $\alpha_{d\sigma}$	186	-97	14	0.5	166	195	21
KA4505	Rotary core	11.5	Constant $\alpha_{d\sigma}$	186	-97	30	0.5	200	252	40
KA6705	Rotary core	11.9	Constant $\alpha_{d\sigma}$	186	-97	55	0.5	248	323	63
KA9005	Rotary core	11.7	Constant $\alpha_{d\sigma}$	186	-97	90	0.5	278	350	90
KASS	Rotary core	9.0	Simple shear	186	-97	—	0.43	231	302	53

Constant- $\alpha_{d\sigma}$ tests were performed in the ICHCA II and the simple shear test in the ICRCHCA.

*For simple shear tests the b value at peak q is given.

Table 6. Summary of HCA tests for Gault clay series

Test reference	Sample type	Specimen depth: mBGL	Type of test	Initial stresses		$\alpha_{d\sigma}$: degrees	$b^\dagger$	Stress state at peak q		
				p'_0 : kPa	$\sigma'_{v0} - \sigma'_{h0}$: kPa			p'_f : kPa	q_f : kPa	α_f : degrees
GA0005	Rotary core	9.5	Constant $\alpha_{d\sigma}$	163	-85	0	0.5	146	212	0
GA2305	Rotary core	9.8	Constant $\alpha_{d\sigma}$	163	-85	15	0.5	163	207	21
GA4505	Rotary core	11.8	Constant $\alpha_{d\sigma}$	163	-85	28	0.5	157	189	39
GA6705	Rotary core	10.6	Constant $\alpha_{d\sigma}$	163	-85	55	0.5	200	224	65
GA9005	Rotary core	10.3	Constant $\alpha_{d\sigma}$	163	-85	90	0.5	206	252	90
GA2305* ‡	Rotary core	12.5	Constant $\alpha_{d\sigma}$	163	-85	15	0.5	—	—	—
GASS	Rotary core	10.9	Simple shear	163	-85	—	0.45	162	198	53

All tests were performed in the ICRCHCA apparatus.

† For simple shear tests the b value at peak q is given.

‡ Test GA2305* was interrupted during shearing due to a power outage. Data are presented up to that point.

Table 7. Summary of HCA tests for London clay series

Test reference	Sample type	Specimen depth ‡ : mBGL	Type of test	Initial stresses		$\alpha_{d\sigma}$: degrees	b^*	Stress state at peak q		
				p'_0 : kPa	$\sigma'_{v0} - \sigma'_{h0}$: kPa			p'_f : kPa	q_f : kPa	α_f : degrees
LA0005	Block	16.3	Constant $\alpha_{d\sigma}$	323	-165	0	0.5	282	318	0
LA2305	Block	16.3	Constant $\alpha_{d\sigma}$	323	-165	15	0.5	296	303	23
LA4505	Block	16.3	Constant $\alpha_{d\sigma}$	323	-165	30	0.5	302	242	48
LA6705	Block	16.3	Constant $\alpha_{d\sigma}$	323	-165	55	0.5	365	381	67
LA9005	Block	16.3	Constant $\alpha_{d\sigma}$	323	-165	90	0.5	415	484	90
LASS	Rotary core	16.2	Simple shear	315	-171	—	0.56	301	245	53
LASS1	Rotary core	15.8	Simple shear	252	-120	—	0.42	314	323	48
LASS2	Block	16.3	Simple shear	323	-165	—	0.64	317	277	53
LISS	Rotary core	15.6	Simple shear	285	0	—	0.51	322	278	51
LISS1	Block	16.3	Simple shear	50	0	—	0.44	79	96	54
LISS2	Block	16.3	Simple shear	150	0	—	0.48	207	202	51
LISS3	Rotary core	15	Simple shear	450	0	—	0.47	438	359	48

All tests were performed in the ICRCHCA apparatus.

*For simple shear tests the b value at peak q is given.

‡ For block samples, the indicated depth includes 5.8 m of Thames River Terrace gravel that had been removed 70 years before sampling (see Nishimura *et al.*, 2007).

Estimating in situ K_0 is not straightforward in these heavily overconsolidated clays. Three approaches were followed in initial scoping tests, as described below.

- (a) Equation (6), proposed by Mayne & Kulhawy (1982), was evaluated after assessing the in situ OCRs of the four clays. However, this approach was hampered by uncertainties in the maximum burial depths and in the effects of recent stress history, including glaciation and tree action.

$$K_0 = (1 - \sin \phi') \text{OCR}^{\sin \phi'} \quad (6)$$

- (b) K_0 values were interpreted from suction measurements made on multiple triaxial and HCA samples, making allowance for deviator stress release and carefully establishing the in situ vertical effective stress.
- (c) In situ measurements were considered that had been made with a Marchetti dilatometer in the Kimmeridge clay and a self-boring pressuremeters for the Oxford clay.

The K_0 ranges estimated from these above sources are listed in Table 1. In selecting the finally chosen test values (see Table 1), it was noted that several estimates exceeded the theoretical lower bound passive coefficient $K_p \approx 2$ as evaluated with triaxial compression ϕ' values obtained from either optimally fissured natural samples, or the critical state angles from tests on reconstituted samples (Hosseini Kamal *et al.*, 2014). It was also noted that trying to apply $K > 2$ led to excessive shear straining during reconsolidation of the first clay tested in Brosse's (2012) study, Oxford clay. As a result, Oxford clay samples were tested from $K = 1.8$ instead of the estimated values of 3.2, so that the shear stage started from higher vertical stresses than in situ for this soil (see Tables 1 and 4). Uniform K values of 1.8 were therefore adopted for the remaining Gault and Kimmeridge clays that represented consistent best estimates and simplified test interpretation with the earlier London clay LA series, which adopted the same K_0 value ($= 1.8$).

Drained isotropic reconsolidation was conducted first to reach the desired p' values and then $(\sigma_z - \sigma_\theta)/2$ was reduced, keeping constant p' , to reach the estimated in situ effective stress conditions. Small-strain non-destructive resonant column tests were conducted during all ICRCHCA tests to monitor how $G_{z\theta}$ varied during reconsolidation and shearing; see Gasparre *et al.* (2007b) and Brosse *et al.* (2016, 2017). The latter had no discernible effect on permanent strains or pore water pressures and so did not affect the subsequent strength measurements.

Stress-strain measurements can be severely compromised by any residual creep straining that may apply prior to shearing; see Jardine (1985), Gasparre *et al.* (2014) or Nishimura (2014). Sufficient pause times were therefore imposed after all consolidation stages for creep rates to stabilise before commencing undrained shearing. Several days were required for the axial strain rates to fall below the adopted 0.002%/h criterion.

The shearing stages of the experiments termed 'stress-path' or 'constant- α_{ds} ' tests all involved an intermediate stage to change b from its initial value of unity (which applies with $K_0 > 1$) to the desired test value under undrained conditions, while keeping p , q and α constant. Nishimura *et al.* (2007) explored a wide range of b values in experiments on London clay, but only α_{ds} tests involving $b = 0.5$ are considered here. After allowing 12 h for creep, the final undrained shearing was initiated with constant p , b and α_{ds} values, where α_{ds} is the direction of the axis of the major principal stress increment, as defined by equation (4). This 'tangent' expression refers to the angles developed over small

increments of stress change. However, the α_{ds} directions were kept constant during the shearing stages of all tests, except the simple shear experiments in which the samples' vertical and circumferential stresses were able to respond spontaneously to the imposed kinematic control conditions of zero vertical, radial and circumferential strains. With the exception of the experiments imposing $\alpha_{ds} = 0^\circ$ and 90° , tests that started from $K > 1$ experienced continuous variations in α during undrained shearing, as illustrated in Fig. 6. The imposed values of α_{ds} were selected so that for each test peak q developed at α values close to targets of 0° , 23° , 45° , 67° and 90° . The latter were achieved accurately, as demonstrated in Tables 4–7 by the peak q angles, which are denoted as α_f . The shearing stages of the constant- α_{ds} tests were conducted under a mixed stress and strain control arrangement to obtain a constant average deviatoric strain rate of 0.15%/h (Brosse, 2012). Future investigations may be able to consider whether strain rate has any effect on shear strength anisotropy. In experiments applying $23^\circ < \alpha_{ds} < 67^\circ$ the torsional loading was driven at a constant rate, while the axial load and cell pressures were controlled to keep the prescribed stress path. For the 0° and 90° conditions the axial displacement was driven at a constant rate, and the cell pressures controlled to follow the desired stress path. The driving rates of the 'principal' loading devices were adjusted after trial experiments to achieve the target deviatoric strain rate. Examples are shown in Fig. 7 of how each stress component was varied (following the equations presented in Fig. 6) during the b -change and q -change stages of test OA6705.

The HCA simple shear tests were set up and reconsolidated in the same way as the 'stress-path' tests. No b -change stage was included. Instead, spontaneous b changes were observed as part of the specimens' response to the imposed simple shearing conditions. Most involved K_0 reconsolidation to in situ stresses except for Brosse's (2007) 'variable stress' London clay tests, which were swelled/consolidated to a range of p' and shear from isotropic conditions. The experiments progressed directly from their equilibrium reconsolidated states to undergo undrained torsional shear straining at 0.15%/h. The specimen heights and inner cavity volumes were kept constant by mechanical constraints (closing the drainage valves and restricting the axial shaft displacements). The outer cell pressure was constant and a servo system controlled the axial force to compensate for any effects of apparatus compliance. As described by Nishimura (2006), Minh *et al.* (2011) and Brosse (2012) this arrangement overcame the considerable boundary condition difficulties, non-uniformities, sample size effects and uncertainties implicit in conventional simple shear testing and led to results that could be expressed fully in terms of the four-dimensional stress and strain tensors, along with direct measurements of the samples' pore water pressure responses. The simple shear tests typically failed with $50^\circ < \alpha < 60^\circ$ and b between 0.4 and 0.6.

Stress and strain measurements

Accurate measurement, from the very early stages up to failure, of the four-dimensional stress and strain states developed in HCA tests poses considerable technical difficulties. The HCA cells illustrated in Figs 4 and 5 were set in a tightly temperature-regulated laboratory. Very stable, 16-bit resolution auto-ranging analogue-to-digital converters were employed to track the stress and strain measurements. Both HCA systems employed high-quality inner and outer cell pressure measurements as well as purpose-designed, dual-axis, pressure-compensated, internal load cells that measured the axial force and torque applied to the samples.

These biaxial cells were carefully calibrated for both direct and cross-sensitivity under the load paths applied employed for testing. As described above, the two HCA sets employed a mix of local or semi-local (platen-to-platen) strain measurements in conjunction with global measurements of the axial displacement and volume changes. Apparatus compliance effects were alleviated in the ICRHCA by making

platen-to-platen torsional shear strain measurements and applying calibrated compliance corrections to external axial strain measurements, while the rigid HCA connection systems reduced any loading eccentricity or tilting. Full local strain instrumentation was deployed in the ICHCA II experiments. Membrane penetration was not significant with the fine-grained clays tested. The sample platen contacts were made as effective as possible by using sintered bronze porous discs, shear vanes and consolidation stages that imposed pre-shearing vertical loads. The resolutions and accuracies of the various instruments are summarised in Table 8. The electro-mechanical systems allowed precise controls of all the pressures (± 0.2 kPa) and the axial force (± 0.2 N). These systems, along with fully automated feedback personal computer (PC) controls, permitted precise application of p (± 0.5 kPa), q (± 1 kPa), α ($\pm 0.1^\circ$; except at stresses very close to isotropic states, where α is indeterminate) and b (± 0.005), even over test stages that extended for several hours.

EXPERIMENTAL OUTCOMES

The following sections outline general aspects of the shear strength and deformation characteristics observed in HCA tests on all four clays. Nishimura (2006), Nishimura *et al.* (2007, 2008) and Brosse (2012) provide numerous two-dimensional (2D) and three-dimensional (3D) plots to describe the effective stress paths and stress-strain relationships provided by the four degree of stress and strain freedom HCA experiments. This paper focuses on the sub-set of outcomes that are essential to characterise undrained shear strength anisotropy, while Brosse *et al.* (2016) describe the anisotropic and highly non-linear stiffness characteristics revealed by the HCA tests.

Shear localisation

Shear localisation had a vital impact on the clays' shearing behaviour. The onset of any shear localisation could be detected during HCA shearing by inspecting the deformation of a grid pattern drawn on the outer membrane and is designated as 'first rupture' herein. The optical magnification caused by the cell water allowed shear displacements of about 0.5 mm to be resolved by eye along any grid line, allowing shear band formation to be identified before the shear strain had grown by 0.25% when continuous observations were made so that the q value at first rupture could be estimated and identified on the stress-strain plots. Ranges are given for those cases where shear localisation started overnight.

Fig. 7. Example of stress control showing theoretical and measured values during (a) b -change and (b) constant- α stages in test OA6705. For ease of reading, only the stress changes from the beginning of each stage are plotted. Initial stress states for each stage are indicated on the respective figures

Table 8. Resolution and accuracy of the instrumentation used in the ICRHCA and ICHCA II

Parameters	Units	Location	ICRHCA		ICHCA II	
			Resolution	Accuracy*	Resolution	Accuracy*
Axial force	N	External	0.004	2.02	0.007	3.76
Torque	N m	External	0.0002	0.32	0.0005	0.69
Pore water pressure	kPa	External	0.04	0.41	0.006	0.92
Inner cell pressure	kPa	External	0.04	0.34	0.013	0.88
Outer cell pressure	kPa	External	0.01	1.02	0.006	0.99
Axial displacement	mm	External	0.0004	0.02	0.0005	0.01
Sample volume change	cm ³	External	0.0008	0.04	0.0019	0.06
Inner cell volume change	cm ³	External	0.0008	0.03	0.0009	0.08
Rotation	degrees	External	0.0007	0.23	0.0005	0.02
Outer wall displacement	mm	Internal	—	—	0.0004	0.0084
Inner wall displacement	mm	Internal	—	—	0.0003	0.0018
Axial displacement	mm	Internal	—	—	0.0004	0.011
Shear strain	degrees	Internal	—	—	0.0002	0.041

*Accuracy is defined as the 95% confidence range (two standard deviations from the true value).

Shear localisation and consequent strain non-uniformity is a well-recognised problem in soil testing, particularly with overconsolidated plastic clays such as those considered in this study. There is no perfect way to (a) take account of their development and so (b) deduce what might apply if the samples' response was perfectly uniform. Recognising this difficulty, the current authors report when shear localisation commenced in each test, as revealed by the grids marked on the specimens' external membranes and identify which parts of each test can be regarded as representing broadly continuous element behaviour and which parts represent the locally non-uniform global response. Two envelopes are therefore reported later in Figs 11, 12 and 15. Shear localisations can occur at microscales from the very early stages of shearing, as tomography by Viggiani & Hall (2008) has shown. Any soil specimen, even if reconstituted, is inhomogeneous at some scale and it is usual to consider such 'non-uniform' aspects of behaviour as part of the element tests' overall response, provided that the specimens possess a sufficiently representative volume. Given the difficulties of characterising post-peak behaviour, the present authors offer no interpretation from the HCA tests of the clays' post-rupture, critical state or residual shear strengths. However, Hosseini Kamal *et al.* (2014) report how these aspects of behaviour were investigated in parallel triaxial and ring-shear experiments.

Stress-strain behaviour

This study involves experiments in which the stresses and strains have four degrees of freedom. The shear strengths

developed can be expressed in a variety of ways. One approach is to adopt stress invariants to give a due consideration to all three principal stresses. However, in these particular test series with b fixed at 0.5, the following simple relationships hold between commonly used parameters p' , s' , q and t , and the first invariant I_1 of the Cauchy stress tensor and J_2 the second invariant of the deviatoric stress tensor

$$p' = \frac{I_1}{3} \quad (7)$$

$$J = \sqrt{J_2} = \frac{1}{\sqrt{3}} \sqrt{1-b+b^2} q = \frac{q}{2} = t \quad (8)$$

In the same way, when $b=0.5$, the following relationship exists between the commonly used undrained shear strength, S_u , and the generalised peak deviatoric stress, J

$$S_u = \frac{\sigma_1 - \sigma_3}{2} = J_{\text{peak}} \quad (9)$$

Therefore a peak of q or t coincides with that of J , and a peak of t/s' coincides with that of J/p' . This paper adopts peak S_u and peak t/s' to quantify the shear strength, as these more widely employed measures of shear strength can be compared simply and directly with other types of experiment, such as triaxial tests.

The deviatoric stress-shear strain relationships, $q-(\varepsilon_1 - \varepsilon_3)$, observed during the undrained shear stages of the suites

Fig. 8. Stress-strain curves in $q-(\varepsilon_1 - \varepsilon_3)$, observed during the undrained shear stages of the suites of the constant- α_d and simple shear HCA tests conducted from in situ stress conditions: (a) Oxford clay; (b) Kimmeridge clay; (c) Gault clay; (d) London clay

Fig. 9. Stress ratio-strain curves in $t/s'-(\varepsilon_1 - \varepsilon_3)$, observed during the undrained shear stages of the suites of the constant- $\alpha_{d\sigma}$ and simple shear HCA tests conducted from in situ stress conditions: (a) Oxford clay; (b) Kimmeridge clay; (c) Gault clay; (d) London clay

of the constant- $\alpha_{d\sigma}$ and HCA simple shear tests conducted from in situ stress conditions are shown in Figs 8(a)–8(d), while Figs 9(a)–9(d) present the corresponding $t/s'-(\varepsilon_1 - \varepsilon_3)$ plots.

The figures distinguish between the tests' pre-and-post shear localisation stages. It can be seen that first visible localisation generally preceded the peak $S_u = q/2$ and peak t/s' conditions. The peaks in q and t/s' were not always concurrent, but all developed over the 1.5–9% shear strain range. The dominantly torsional shear mode tests (that failed with $20^\circ < \alpha_f < 70^\circ$) provoked shear discontinuity formation at smaller shear strains than the other experiments. The observed failures were brittle, differing from those seen in tests on ductile, low-plasticity, overconsolidated clays, but the HCA tests exhibited far less marked displacement-softening than triaxial tests on the same clays. Neither well-defined 'post-rupture' strengths (Burland, 1990) nor residual strength conditions were developed. It should be borne in mind that, as with other types of soil shear testing, HCA tests represent a non-uniform boundary value problem. The behaviour observed after shear surface formation is a consequence of interactions between the intact soil, the shear band material that is undergoing marked micro-fabric changes and the apparatus geometry. In most cases shear localisation started along one of the many natural discontinuities. The point of 'first rupture' is taken as the point at which shear localisation was first evident. However, the first shear localisation features did not necessarily persist and become parts of the final failure

mechanism. In some cases, the progress of relative displacement skipped to different locations, and/or they were connected by a newly formed shear zone propagating through apparently intact parts. The process of localised shear mechanisms trying to find weaker zones, and when they could not, disrupting intact parts with higher resistance, took place progressively and led to the relatively large overall strains observed between first localisation and peak resistance. Nishimura (2006) and Brosse (2012) give further details of how the shear bands propagated during each experiment and how they interacted with the eventual final failure mechanisms.

The trajectories of the above tests' effective stress-paths are divided in two parts in Figs 10(a)–10(d) and Figs 11(a)–11(d), plotting the projections of the paths in the $[(\sigma_z - \sigma_\theta)/2] - p'$ plane and the $\tau_{z\theta} - [(\sigma_z - \sigma_\theta)/2]$ plane, respectively. The mean effective stresses applying at final failure tended to increase with α_f , rising from minima $\alpha = 0^\circ$ towards markedly higher p' when failing at $\alpha = 90^\circ$, reflecting the negative $d(\sigma_z - \sigma_\theta)/d(2p')$ slopes applying from the start of shearing up to the final dilative failure stages. The general trend of pre-failure 'dilative' behaviour for extensional loading ($\alpha_{d\sigma} > 45^\circ$) and 'contractive' behaviour for compressional loading ($\alpha_{d\sigma} < 45^\circ$) is a factor that contributed significantly to the clays' S_u anisotropy, as discussed below. Nishimura *et al.*'s (2007) wider ranging HCA study of London clay showed that similar patterns apply irrespective of the b value at which samples are taken to failure.

$$\begin{Bmatrix} \delta \varepsilon_r \\ \delta \varepsilon_\theta \\ \delta \varepsilon_z \\ \delta \gamma_{\theta z} \\ \delta \gamma_{zr} \\ \delta \gamma_{r\theta} \end{Bmatrix} = \begin{bmatrix} \frac{1}{E_h} & \frac{-\mu_{hh}}{E_h} & \frac{-\mu_{vh}}{E_v} & 0 & 0 & 0 \\ \frac{-\mu_{hh}}{E_h} & \frac{1}{E_h} & \frac{-\mu_{vh}}{E_v} & 0 & 0 & 0 \\ \frac{-\mu_{hv}}{E_h} & \frac{-\mu_{hv}}{E_h} & \frac{1}{E_v} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{G_{hv}} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{G_{vh}} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{2(1+\mu_{hh})}{E_h} \end{bmatrix} \begin{Bmatrix} \delta \sigma_r \\ \delta \sigma_\theta \\ \delta \sigma_z \\ \delta \tau_{\theta z} \\ \delta \tau_{zr} \\ \delta \tau_{r\theta} \end{Bmatrix} \quad (10)$$

Fig. 10. Projection of the effective stress paths on the $[(\sigma_z - \sigma_\theta)/2] - p'$ plane for the constant- $\alpha_{d\sigma}$ and simple shear HCA tests conducted from in situ effective stresses: (a) Oxford clay; (b) Kimmeridge clay; (c) Gault clay; (d) London clay

The pre-failure ‘dilatancy’ characteristics appear to be related predominantly to the $(\varepsilon_z - \varepsilon_\theta)$ mode of deformation, with the $\gamma_{z\theta}$ mode having comparatively little effect. Nishimura (2006) and Brosse (2012) made theoretical estimates for the effective stress-path inclinations, $d(\sigma_z - \sigma_\theta)/d(2p')$ by applying the cross-anisotropic elastic model given in equation (10) and employing parameters adopted from the small-strain stiffness measurements of Gasparre *et al.* (2007b), Hosseini Kamal (2012) and Brosse *et al.* (2017), who found that $E'_h > E'_v$ and anisotropic sets of μ_{vh} , μ_{hv} and μ_{hh} Poisson ratios applied to all four clays. It should be noted that according to the cross-anisotropic theory, the inclinations $d(\sigma_z - \sigma_\theta)/d(2p')$ vary with α except for the special case employed in the presented tests where $b = 0.5$.

The theoretical $d(\sigma_z - \sigma_\theta)/d(2p')$ estimates and measurements are listed in Table 9. The input elasticity parameter values were measured on different specimens from the same block samples. The theoretical values of $d(\sigma_z - \sigma_\theta)/d(2p')$ are predominantly determined by the ratio E'_h/E'_v , which can be determined through suites of triaxial tests involving local strain and shear wave velocity measurements; see Gasparre

et al. (2007b), Nishimura (2014) or Brosse *et al.* (2017). Ratios between 1.6 and 3.1 are found for the four clays, with the upper and lower limits applying to the London and Gault clays, respectively. The four clays’ initial gradients varied between tests but gave average values in the -2.0 ± 0.3 range. While predictions from the simplified analysis have the same negative sign, they overestimate the initial the gradients and so under-predict the influence of changes in $(\varepsilon_z - \varepsilon_\theta)$ on p' , except for the Gault clay case. The adopted anisotropic ratios of clay stiffness were established at very small strains (less than 0.001%) and may not be fully applicable to the whole range represented in Fig. 10, although the $d(\sigma_z - \sigma_\theta)/d(2p')$ inclinations did not vary greatly from small strains, over the non-linear, elasto-plastic phase, until failure was approached.

It might be argued that the predicted and measured ratios may have differed because the input stiffness ratios were measured under axisymmetric stress states that were altered in the HCA tests by applying non-symmetric stresses in the b -change stage. Semi-quantitative evaluation can be made following the anisotropic Young’s moduli model based on the effective stress raised by constant exponents, as outlined, for example, by Hoque *et al.* (1996). However, Nishimura (2006), Gasparre *et al.* (2007b), Hosseini Kamal (2012) and Brosse *et al.* (2017) report that the clays’ elastic stiffness anisotropy is insensitive to small changes in effective stress states, as quantified by the exponent values set for the above model. Applying, for example, Nishimura’s LA series London clay stiffness data indicates that the b -change stage effective stress changes would lead to a 1.6% change in E'_h/E'_v that would not affect any of the clays’ $(\sigma_z - \sigma_\theta)/d(2p')$ inclinations significantly.

Table 9. Predicted and measured $d(\sigma_z - \sigma_\theta)/d(2p')$ stress-path gradients from undrained shearing stages of HCA tests maintaining $b = 0.5$

Soil	Measured stress path gradients		Ranges predicted from anisotropic elastic stiffness parameters
	Range	Average	
Oxford clay	-1.5 to -1.9	-1.7	-2.1 to -3.3
Kimmeridge clay	-1.8 to -3.5	-2.3	-1.9 to -3
Gault clay	-1.6 to -2.9	-2.2	-1.2 to -1.7
London clay (Unit B2c)	-1.9 to -2.7	-2.2	-4.2

Anisotropy in undrained shearing resistance

First the HCA test outcomes are reported in total stress terms, considering the clays within the framework of a purely cohesive (Tresca) material whose shear strength is

Fig. 11. Projection of the stress paths on the $\tau_{z\theta}$ - $[(\sigma_z - \sigma_\theta)/2]$ plane for the constant- $\alpha_{d\sigma}$ and simple shear HCA tests conducted from in situ effective stresses: (a) Oxford clay; (b) Kimmeridge clay; (c) Gault clay; (d) London clay

Fig. 12. Variations in undrained peak shear strength S_u with the orientation of major principal stress at failure α_f covering both the constant- $\alpha_{d\sigma}$ and simple shear HCA tests conducted from in situ stresses: (a) Oxford clay; (b) Kimmeridge clay; (c) Gault clay; (d) London clay

independent of p . For an isotropic material following this framework, the peak strength envelope forms a circle centred on the origin in Figs 11(a)–11(d). All of the four clays deviate markedly from this pattern. Figs 12(a)–12(d) detail the clays' variations of undrained shear strength, S_u , with α_f , covering both the constant- $\alpha_{d\sigma}$ and simple shear tests conducted from in situ effective stress conditions, emphasising the differences between the strengths developed at the first rupture and ultimate peak conditions. The variation has also been fitted on Figs 12(a)–12(d) using the empirical equation proposed by Bishop (1966)

$$S_u = S_{u0} [1 - a_1 \sin^2(\alpha)] [1 - a_2 \sin^2(2\alpha)] \quad (11)$$

where S_{u0} is the undrained strength at $\alpha = 0^\circ$, and a_1 and a_2 are fitting parameters.

A summary is made in Fig. 13 of the four clays' trends with α of peak $S_u = q/2$. Although the samples were all taken from broadly similar depths, the London clay specimens were tested at higher effective stresses than the other clays (see Tables 4–7) because, as noted earlier, they had been covered by 6 m of River Terrace gravels. It is evident that the Gault and Kimmeridge clays developed generally lower undrained shear strengths than the Oxford clay, and the London clay specimens tested at significantly higher effective stresses. The Gault clay showed the least variation of S_u with α , with its minimum applying when the σ_1 axis was rotated by around 40° from the vertical and its maximum when $\alpha = 90^\circ$. The Kimmeridge clay developed a similar $\alpha = 0^\circ$ S_u value to the Gault, but manifested its minimum at a lower α (in the range 0° to 20°) and a 50% higher maximum S_u at $\alpha = 90^\circ$. The Oxford and London clays developed their minima with α around 45° , and S_u maxima at $\alpha = 90^\circ$ that were double their intermediate α minima.

Fig. 13. Summary of the four clays' trends of variation of undrained peak strength S_u with α_f from constant- $\alpha_{d\sigma}$ and simple shear HCA tests conducted from in-situ effective stresses

Overall, the four clays share similar hierarchies in undrained shear strength: S_u is lowest when α lies between 10° and 45° , when the specimens' failure mechanisms could best exploit pre-existing horizontal fissures or bedding features, while S_u is highest when $\alpha = 90^\circ$. The most intensively fissured Gault showed the least variation between these limits, while the London and Oxford clays showed the most marked S_u anisotropy. The simple shear tests, which failed at α around 50° – 60° and $b \approx 0.4$ – 0.6 , gave relatively low S_u values that fit well with the constant- $\alpha_{d\sigma}$ experiments anisotropic S_u trends.

Influence of initial effective stress level on peak shear strength

As was demonstrated in Figs 8–10, the four brittle, plastic, overconsolidated clays reach their peak shear strengths shortly after developing shear discontinuities and do not manifest any extended period of dilative behaviour. Hence S_u depends on the effective stress state under which the envelope is first engaged, and is not uniquely related to water content or void ratio (see, for example, Jardine *et al.* (2004)). Specimens possessing a geologically controlled effective stress failure envelope can exhibit a variety of S_u values at similar water contents, depending principally on variations in their initial effective stress states. Imposing the correct effective stress and K_0 conditions is therefore critical when attempting to measure the S_u anisotropy that would apply in the field. It is generally not possible to impose such conditions when conducting triaxial compression tests on inclined samples.

As noted at the outset and illustrated in Fig. 1, the stiff clays manifest en-echelon families of curved effective stress peak failure envelopes and state boundary surfaces under triaxial test conditions. The envelopes vary with each soil's structure and depth and each envelope presents reductions in t/s' with increasing mean effective stress. While it was not possible to explore in detail how these features affected each clay's effective stress shear strength anisotropy, Brosse (2007) and Nishimura *et al.* (2008) studied effective stress level dependency of the strength in simple shear tests on London clay samples as outlined in Table 7. Fig. 14 summarises the peak t – s' conditions developed in four simple shear tests conducted on samples taken from a level 16 m below the original gravel surface, 10 m below the top of the clay. Four were tested from in situ stress conditions, two from lower initial effective stresses and one from elevated consolidation stresses. The observed curved failure trend envelope represents that applying under the $45^\circ < \alpha_f < 55^\circ$ and $0.4 < b < 0.6$ 'simple shear' range. This curved envelope has comparable shapes and scales to the triaxial compression envelopes for London clay samples at 6–12 m depth outlined in Fig. 14. The t/s' ratios applying to tests involving α rotation are also likely to decline with increasing mean effective stress p' . This trend has been confirmed in subsequent additional HCA tests on Kimmeridge clay samples.

Peak shear strength anisotropy in terms of mobilised t/s' ratios

As shown in Fig. 10, all four clays showed significant variation with α in their peak failure p' and s' values as a result of the pre-failure $d(\sigma_z - \sigma_\theta)/d(2p')$ gradients listed in

Fig. 14. Peak t – s' conditions developed in simple shear HCA tests conducted on London clay samples taken from the 16 m depth below original gravel surface level, considering tests sheared from in-situ and other effective stress levels (modified from Nishimura *et al.*, 2008). The $c' = 0$, $\phi' = 19^\circ$ line represents the fissure shear strength

Table 9. This feature contributed much to the different anisotropy patterns seen for S_u (Fig. 13) and peak t/s' (shown subsequently in Fig. 16), with lower α leading to a smaller S_u for a given t/s' due to a reduced s' value at failure. This relationship between S_u and peak t/s' is further skewed by the curvature of the peak failure envelopes associated with each α value, with t/s' decreasing as s' increases. Further testing is required to provide a comprehensive assessment for each clay of the curved effective stress peak shear strength envelopes corresponding to fixed values of b and α .

The first rupture and peak conditions identified for each clay in undrained tests from in situ stress conditions are reported in Figs 15(a)–15(d) by plotting t/s' against α , while an overall summary plot for the peak t/s' points is given in Fig. 16 that shows at its right-hand side equivalent values of $\phi'_{mob} = \sin^{-1}(t/s')$. As with the S_u traces, the t/s' minima develop at intermediate α , although the range of angles is relatively larger, falling between 45° and 70° . The clays show their t/s' maxima at $\alpha_f = 0^\circ$, rather than the 90° orientation applying to S_u , reflecting the lower effective stresses applying at smaller α values that were discussed above. The mobilised ϕ' angles are all relatively high, reflecting envelope curvature and potentially limited cohesive bonding in the geologically aged clays. As with the S_u plots, the simple shear experiments led to peak t/s' points that were compatible with the $\alpha_{d\sigma}$ experiments conducted over the same α range.

Brosse's (2012) detailed analysis showed that the two $\alpha_f = 0^\circ$ Oxford clay tests both developed limited tensile circumferential stresses, with $\sigma'_3 = \sigma'_\theta$, and also exhibited a clear pattern of vertical tensile cracks in the samples. The Oxford clay $\alpha_f = 22.5^\circ$ test also failed close to the no-tension line (that is the line corresponding to $\sigma'_3 = 0$ and $t/s' = 1$). As summarised in Table 1 the Oxford clay samples' meso-fabrics were unique in displaying primarily bedding and no fissuring; these features enhanced its shear strength at low α values. Although impossible under ordinary triaxial conditions where pore water pressures cannot exceed the cell pressure and negative σ'_3 values can never be applied, negative effective circumferential stresses can develop in HCA tests conducted with $b > 0$ and low α values, as the annular geometry allows true circumferential tension. However, the non-uniformities developed by samples at late test stages, and the limited magnitudes (a few kPa) of the recorded negative stresses make it difficult to draw reliable detailed conclusions regarding this Jurassic clay's true cohesive strength, if any. Triaxial compression tests performed on Oxford clay by Hosseini Kamal (2012) on similar samples led to much higher S_u , with samples manifesting a more extended dilative phase (see Table 2 and Fig. 12). Tests conducted at low effective stress under axi-symmetric triaxial conditions on strong, heavily overconsolidated clays may overestimate the undrained strength.

It should be noted that the observed variation of the strength with α_f is not solely due to the meso-fabric. Nishimura *et al.* (2007) indicated for London clay that even reconstituted specimens exhibited significant strength anisotropy, but with a different pattern to the natural specimens. The anisotropy of the natural samples represents the combined outcome of the intrinsic microstructure, as applying at deposition, and the subsequent geological history which imparted, among other processes, burial and compaction, diagenesis, fissuring, stress relief, vegetative impact, freeze–thaw cycles and weathering.

Influence of intermediate principal stress on peak shear strength

As noted previously, the influence on shear strength of σ'_2 , as expressed by the imposed b value, was not the principal

Fig. 15. Variations in stress ratio t/s' with the orientation of major principal stress at failure α_f covering both the constant- $\alpha_{d\sigma}$ and simple shear HCA tests conducted from in situ stresses: (a) Oxford clay; (b) Kimmeridge clay; (c) Gault clay; (d) London clay. Right-hand side shows equivalent mobilised angle of shearing resistance. Note that when $b = 0.5$, $t/s' = q/(2p')$, see equations (7) and (8)

Fig. 16. Summary for all four clays of peak t/s' trends against α_f in undrained tests sheared from in situ stresses. The right-hand side shows equivalent mobilised angle of shearing resistance. Note that when $b = 0.5$, $t/s' = q/(2p')$, see equations (7) and (8)

focus of the present study, where most tests were conducted with $b = 0.5$. Nonetheless, the peak shear strengths obtained from triaxial compression ($b = 0$, $\alpha = 0^\circ$) and extension ($b = 1$, $\alpha = 90^\circ$) tests are plotted for comparison in Figs 12 and 15. The lower strengths developed in triaxial extension as compared to the $b = 0.5$ $\alpha = 90^\circ$ case are noticeable, especially in terms of S_u . This feature may reflect the more restricted set of planes on which samples can fail in HCA tests under conditions where the intermediate principal stress direction is constrained to always be radial.

Influence of structure on shear strength anisotropy

As highlighted above, wherever a purely cohesive (S_u) or a purely frictional (t/s') model is considered, the lowest strengths were recorded at intermediate α values, usually in the 30 – 70° range. Brosse (2012) and Nishimura (2006) observed that these tests usually led to strain localisation along predominantly horizontal or low-angled ($\pm 20^\circ$ from the horizontal) shear planes or shear bands. This is to be expected as in these loading orientations one of the set of planes of maximum shear stress ratio is near horizontal. This suggests that horizontal planes are weaker against shear. This is coherent with Wilkinson's (2011) observations on the microstructure of those clays, which show that the platy clay particles are preferably horizontally orientated. The Oxford clay tested presents the strongest alignment of particles, which combined with the presence of beddings observed at the meso-structure level to explain the strong strength anisotropy observed in this soil. London and Gault clay presented milder preferential orientation of particle while Kimmeridge clay had the lowest level. Although potential routes for developing constitutive models that match the experimental results are not discussed here, Brosse (2012) demonstrates how her experiments' anisotropic failure conditions can be characterised by following the criteria and approach set out by Pietruszczak & Mroz (2000).

PRACTICAL IMPLICATIONS

The HCA experiments demonstrate varying degrees of anisotropy in all four clays. Referring the S_u trends displayed in Fig. 13 for near plane strain, $b = 0.5$, conditions to the

triaxial compression undrained shear strengths listed in Table 2 shows that the latter could be non-conservative over a critical range of intermediate α values. If free to form under foundations, dams, retaining walls or in slopes, undrained collapse mechanisms could exploit the experimentally proven anisotropy as they start the progressive failure that eventually leads to residual shear strength development (Potts *et al.*, 1997). Analyses based on peak triaxial compression shear strengths would be misleadingly non-conservative for the London, Gault, Kimmeridge or Oxford clays due to both their anisotropy and their brittleness. Equally, in cases that involve uniform and primarily vertical or horizontal plane strain loading, standard triaxial compression tests could underestimate peak field shear strengths.

As noted earlier, the shear strength anisotropy can also be expected to affect the displacements developed by fully stable engineering works. Geotechnical structures often develop zones of contained failure under safe working conditions that, together with the non-linear stiffness of the soil mass, control the overall patterns and scales of ground movements; see Jardine *et al.* (1986), Addenbrooke *et al.* (1997) or Potts & Zdravkovic (2001). New numerical analyses with anisotropic criteria (such as those discussed by Brosse (2012)) are required to explore how the stiffness and shear strength anisotropy proven by the HCA experiments will impact on a wide range of geotechnical engineering applications. Equivalent analyses of how shear strength anisotropy affects embankment foundation stability with low-OCR clays indicate that the impact will be considerable (Zdravkovic *et al.*, 2002). The same authors show that no single index strength can be adopted as fitting all geometries. For example, the direct simple shear (DSS) strengths, which are often adopted as 'practical' intermediate values between triaxial compression and extension, can either be safe or unsafe depending on the embankment slope angle. The anisotropy reported for the stiff clays shows quite different patterns in which, for example, passive failures invoke higher strengths than those under active conditions, and the implications for practice remain to be explored systematically.

SUMMARY AND CONCLUSIONS

Characterising the shear strength behaviour of heavily overconsolidated, stiff-to-hard plastic clays is crucial to assessing their stability and also influential when predicting ground movements in many geotechnical engineering applications. The anisotropy in shear strength of the London, Gault, Kimmeridge and Oxford clays has been established through advanced HCA experiments on multiple high-quality samples taken at similar depths from inland sites with well-established geotechnical profiles. Ten main conclusions are drawn.

- The clays' micro- and macro-fabrics led to variable degrees of shear strength anisotropy. Overall, the Oxford clay showed the most important anisotropy due to its clear horizontal bedding and absence of fissuring. The lowest degree of anisotropy in terms of the Tresca criterion was shown by the Gault, while the Kimmeridge clay showed the least anisotropy in terms of the Mohr–Coulomb criterion.
- Simple shear strength tests, which failed at intermediate α_f , gave lower range shear strengths (in terms of both S_u and t/s') in all but the Kimmeridge clay.
- All tests showed shear localisation before achieving peak shear strength. This developed earliest in intermediate α_f angle tests that could exploit sub-horizontal micro- and macro-fabric features.

- The mean effective stresses developed during undrained shearing from K_0 values >1 fell when $\alpha_f \leq 45^\circ$ and increased at higher α angles. These features reflected the clays' anisotropic micro-fabrics and stiffnesses.
- Relatively high peak t/s' and intermediate S_u values developed under low- α_f conditions. These results are closely connected to curvature of the effective stress envelopes, as clearly seen in the London clay. This feature is likely to be observed for the other clays. For example, the Oxford clay developed limited tensile circumferential effective stresses in 'active' undrained $b = 0.5$ tests with $\alpha_f = 0^\circ$.
- The clays' variations of peak t/s' and S_u with α_f manifest significantly different patterns (Figs 13 and 16), reflecting the combined effects of: (i) p' changes during shear; (ii) the limited resistance to tensile stress; and (iii) t/s' reducing as p' increases.
- The relatively high peak S_u and intermediate t/s' values found at $\alpha_f = 90^\circ$ reflect the same factors as above (i)–(iii). The soil micro-fabric was relatively resistant to horizontal loading, with the micro-fabric leading to p' increases that depressed the mobilised ϕ' applying in undrained 'passive' ($\alpha_f = 90^\circ$) HCA tests.
- The stiff clays' anisotropic shear strength and pre-failure stiffness characteristics have important implications for stability and ground movements.
- Further constitutive model development combined with programmes of rigorous numerical analysis are required to investigate how the experimentally observed behaviour will impact on specific geotechnical engineering applications.
- Current models that assume a circular bounding surface in the $\tau_{z\theta} - [(\sigma_z - \sigma_\theta)/2]$ plane, such as standard Cam clay, Drucker–Prager models, and so on, cannot capture the skewed envelopes manifested by the four studied overconsolidated clays. Brosse (2012) explores potential routes through which the observed shear strength anisotropy may be characterised more realistically.

ACKNOWLEDGEMENTS

The authors acknowledge the Engineering and Physical Sciences Research Council (EPSRC) for their funding of the research described under grants EP/E029957/1 and EP/D506387/1 to the contributions made by the Imperial College technicians Alan Bolsher, Steven Ackerley and Graham Keefe, as well as their colleagues Professor Matthew Coop and Drs Clark Fenton, Ramtin Hosseini-Kamal and Stephen Wilkinson who undertook the geological and micro-fabric studies that underpinned the work described. They also acknowledge Dr Nguyen Anh-Minh's contribution to parallel series of London clay HCA tests. Although not discussed in this paper, Dr Anh-Minh's experiments provided a vital contribution to the earlier London clay study.

NOTATION

B	pore pressure coefficient
b	intermediate principal stress factor
c'	effective cohesion
e	void ratio
E_v, E_h	effective vertical and horizontal Young's moduli, respectively
G_{hh}	shear modulus in horizontal plane
G_{hv}, G_{vh}	shear moduli in vertical plane
$G_{z\theta} (= G_{vh})$	shear moduli in vertical plane
I_1, I_2	first and second invariants of Cauchy stress tensor respectively
J	generalised deviatoric stress $J = \sqrt{J_2}$

J_2	second invariant of Cauchy deviatoric stress tensor
K	coefficient of earth pressure
K_0	coefficient of earth pressure at rest
K'	drained bulk stiffness
K_p	theoretical lower bound passive coefficient
K_0	coefficient of earth pressure at rest
p'	mean effective stress
p'_0	initial mean effective stress
p'_e	Horslev's equivalent pressure
q	deviatoric stress ($=\sigma'_1 - \sigma'_3$)
s'	$(\sigma'_1 + \sigma'_3)/2$
S_u	undrained shear strength
t	$(\sigma'_1 - \sigma'_3)/2$
u	pore water pressure
α	angle between the vertical and the direction of σ_1 axis
$\alpha_{d\sigma}$	angle between the vertical and the direction of $\delta\sigma_1$ axis
α_f	angle between the vertical and the direction of σ_1 axis at peak q
γ	bulk unit weight
$\gamma_{z\theta}$	shear strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	major, intermediate and minor principal strains, respectively
$\varepsilon_z, \varepsilon_r, \varepsilon_\theta$	axial, radial and circumferential strains, respectively
ζ	mathematical variable used for parametric representation of stress path during constant- $\alpha_{d\sigma}$ shearing stage
$\mu_{vh}, \mu_{hv}, \mu_{hh}$	Poisson's ratios in the cross-anisotropic elastic model
$\sigma_1, \sigma_2, \sigma_3$	major, intermediate and minor principal stresses, respectively
$\sigma_z, \sigma_r, \sigma_\theta$	axial, radial and circumferential stresses, respectively
ϕ'	effective angle of shearing resistance

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