Journal of Zhejiang University-SCIENCE A (Applied Physics & Engineering)
ISSN 1673-565X (Print); ISSN 1862-1775 (Online)
www.zju.edu.cn/jzus; www.springerlink.com
E-mail: jzus@zju.edu.cn

Characterization of mudrocks: a practical application of advanced laboratory testing*

Richard J. JARDINE
(*Department of Civil and Environmental Engineering, Imperial College London, London SW7 2AZ, UK*)
E-mail: r.jardine@imperial.ac.uk

Received Oct. 8, 2010; Revision accepted Nov. 5, 2010; Crosschecked Dec. 7, 2010

Abstract: An integrated approach to geomaterial characterization is advocated that combines geology, in-situ testing, fabric studies, routine index experiments and advanced laboratory testing. It is shown that advanced laboratory testing can explore features such as kinematic yielding and anisotropy in stiffness or shear strength that would otherwise be impossible to quantify. A detailed study performed in London clay at the new Heathrow Terminal 5 site is used to illustrate the arguments made. It is shown that the London clay has strong anisotropy in stiffness, is highly non-linear over the strain range of engineering interest, has markedly anisotropic shear strength characteristics and exhibits a pronounced degree of brittleness. These features can impact significantly on the practical design and analysis of civil engineering works including shallow and deep foundations, tunnels and excavations, and the stability of slopes.

Key words: Mudrock, Fabric, Anisotropy, Hollow cylinder tests
doi:10.1631/jzus.A1000420 **Document code:** A **CLC number:** TU311

1 Introduction

Improvements in computational power have made it possible to employ increasingly sophisticated numerical modelling techniques for practical geotechnical engineering purposes. Fully non-linear treatments can now be made of problems ranging from foundations and deep excavations to tunnels, embankments and slopes. Potts and Zdravkovic (1999; 2001), for example, described how intricate aspects of soil behaviour such as plastic flow, progressive failure, pre-yield non-linearity, post-peak brittleness and non-linear permeability may be considered in analyses of ground stability, stresses, strains, displacements and soil-structure interaction. A debate has started as to whether geotechnical education should place much greater emphasis on modern numerical techniques, raising important issues regarding computer code development, verification and training of engineers (Potts, 2003).

However, it is already clear that complimentary improvements in practical site characterization and soil property determination are crucially important if geotechnical numerical modelling is to realise its full potential. This paper argues the case for applying ground characterization techniques that can provide the information required for modern geotechnical numerical analysis and also point the way for the future development of improved capabilities. Recent work by the Imperial College Geotechnics Group on UK mudrocks is reviewed to demonstrate the benefits of an integrated geological and geotechnical approach. The focus is placed on the series of geologically old stiff-to-hard clays, or mudrocks that underlie much of the Southern UK, as shown in Fig. 1. Major cities, including London, are founded on the mudrock strata, as are critical facilities such as transport routes, nuclear power stations and dams.

* Project supported by the UK Engineering and Physical Sciences Research Council (EPSRC), the British Airports Authority (BAA), and the London Underground Limited (LUL), UK

Among others, Jardine and Potts (1988), Jardine *et al*. (1991), St. John *et al*. (1992), Hight *et al*. (1992; 1997), Hight and Jardine (1993), Potts *et al*. (1997), Harris (2002), Jardine *et al*. (2005) and Kovacevic *et al*. (2004; 2007) described relevant case histories where integrated site characterisation and advanced laboratory testing have delivered considerable benefits to significant civil engineering projects.

One example of a high value construction project where accurate numerical modelling was essential to designers is outlined in Fig. 2. As described by Harris (2002), a 30.5 m deep excavation had to be formed in London clay immediately adjacent to the UK's Houses of Parliament to provide a new metro station and accommodate new running tunnels. The ground movements had to be limited strictly. Fig. 3 shows a comparison between field measurements of the lateral deflections of a central section of the right-hand diaphragm wall and predictions made with a highly non-linear numerical model. The input parameters for the latter were derived from advanced triaxial testing carried out at Imperial College London. In this case the field measurements fell close to the predictions. Jardine *et al*. (2005) considered the circumstances in which a close agreement between other comparable predictions and measurements has

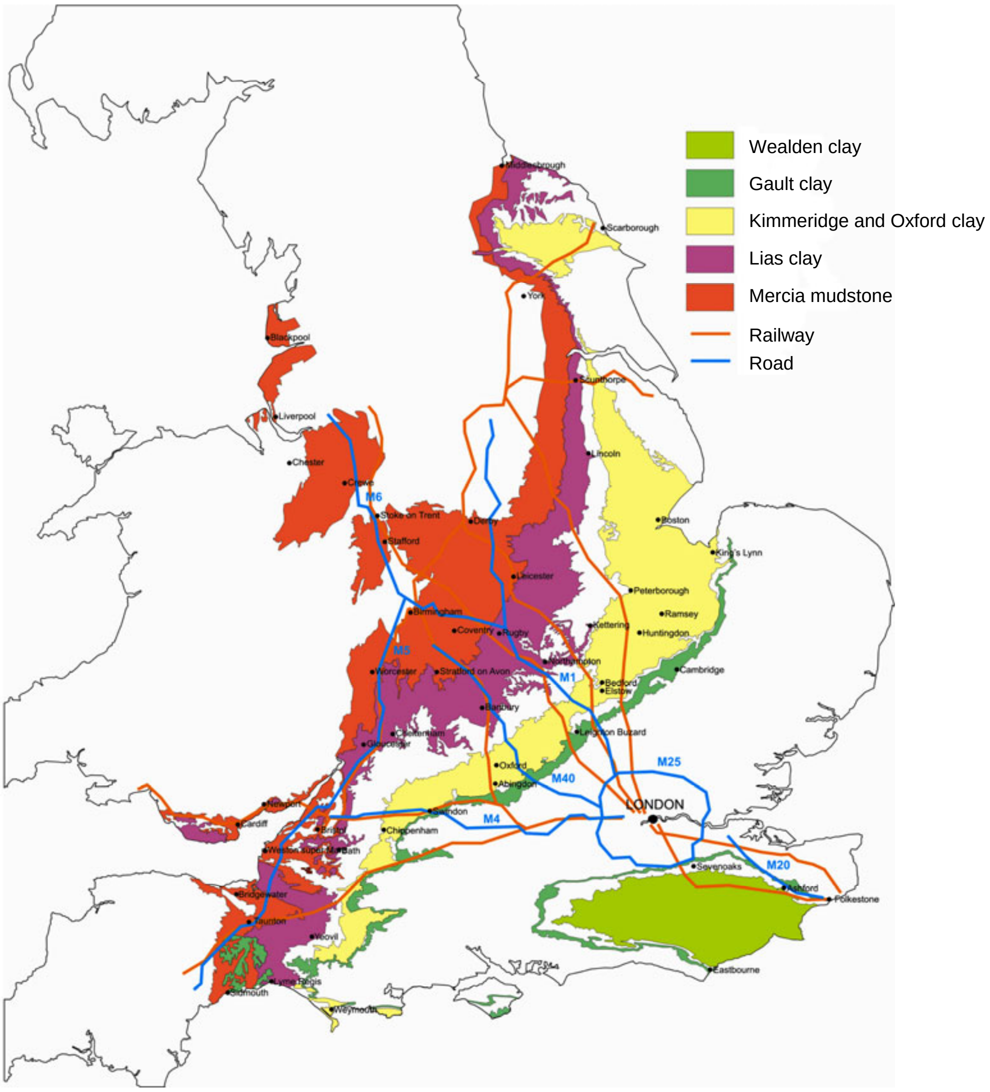

Fig. 1 Geographical distribution of Triassic-to-Eocene mudrocks in England
Strata considered in current Imperial College studies include London, Oxford, Gault, Kimmeridge, and Lias clays

been obtained. They also discussed cases where still more elaborate modelling appears to be required to make similarly accurate forecasts of ground movements and soil-structure interaction, noting the potential need to address the anisotropy, brittleness, creep and strain rate dependency of natural soils.

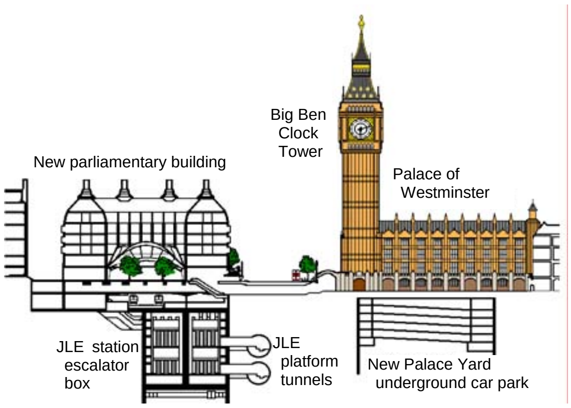

Fig. 2 Jubilee Line Extension (JLE) Metro Station case history
Deep excavation and tunnels in London clay mudrock are opposite UK's Houses of Parliament, London (Harris, 2002)

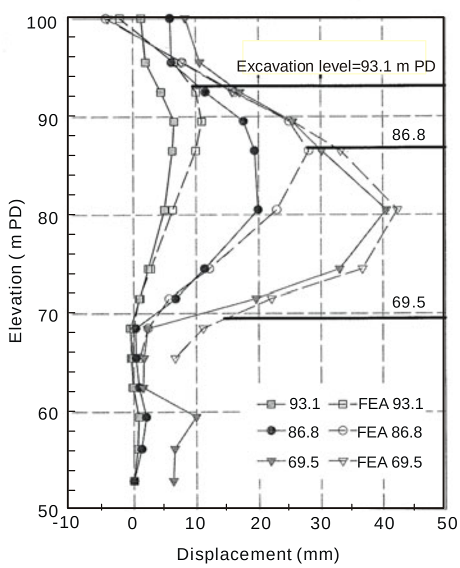

Fig. 3 Non-linear finite element analysis (FEA) based on advanced laboratory testing and measurements for lateral movements of diaphragm wall after excavation from original ground level (taken as 100 mPD) to elevations of 93.1, 86.8, and 69.5 m, respectively (Harris, 2002)

Recent research at Imperial College has investigated further details of the behaviour of the Eocene London clay, concentrating on the 48.5 m thick sequence strata found at the new Terminal 5 (T5) site at London's Heathrow Airport. Gasparre (2005), Anh-Minh (2006), Nishimura (2006), Gasparre *et al*. (2007a; 2007b), and Nishimura *et al*. (2007) provided the detailed descriptions that are the main sources for this paper, while Hight *et al*. (2007) gave an overview of the integrated site characterization undertaken. This paper reviews the results obtained and emphasises particularly the role played by advanced laboratory testing. A new research project is currently underway into the more extended set of older mudrocks identified in Fig. 1, considering the Cretaceous Gault clay and the Jurassic Oxford, Kimmeridge, and Lias clays.

2 Integrated characterization

It is natural that the starting point for improving ground characterization is geology. In the context of the London clay, stratigraphy studies by King (1981) have helped to explain significant variations of soil properties with depth, such as those noted by Hight and Jardine (1993) and Hight *et al*. (2003) at locations across the London basin. It is now understood that the London clay can be divided into 5 major units (Fig. 4) that correlate with main episodes of sedimentation. Each involved a cyclic sequence, starting with relatively coarse near coastal sediments that become finer as the sea level rose and the coastline retreated, before showing a reversing trend as continuing sedimentation led to coastal advance and shallower water depths. De Freitas and Mannion (2007) described how these trends can be followed by inspecting the micro-fossils present in samples. Close analysis of the T5 profile indicated the subdivision as shown in Fig. 4, which correlates very clearly with and helps to explain the variations in composition, state and fabric (Gasparre *et al*., 2007a; Hight *et al*., 2007). Careful visual description was crucial to mapping the pronounced sets of discontinuities present (primarily fractures or fissures, bedding features and sand partings) and to understanding the significant variations of mass permeability with depth. Geology also plays a vital role in understanding the possible variations with a depth of K_0, a parameter that is highly influential in numerical analyses of practical problems such as deep excavations, bored piles or progressive slope failures.

Field geophysics and in-situ testing are also vital to the integrated characterisation of the mudrocks found at T5. Vertical and horizontal shear wave velocities were determined in the field, adopting orthogonal polarisations for each, using an array of three boreholes as described by Hight *et al*. (2007). In-situ cone penetration (CPT) testing was also performed to help identify stratigraphic changes, as advocated by Hight and Jardine (1993), although concretions in particular layers and the friction resistance developed along the CPT rods led to excessive surface reaction force requirements. These difficulties were overcome by advancing the CPT ahead of a dedicated borehole, applying a cumbersome alternating sequence of drilling and CPT probing stages. Specialist cones that release drilling mud at locations well above the CPT probe's friction sleeve may overcome such difficulties by reducing the shear and normal stresses acting on the system of rods.

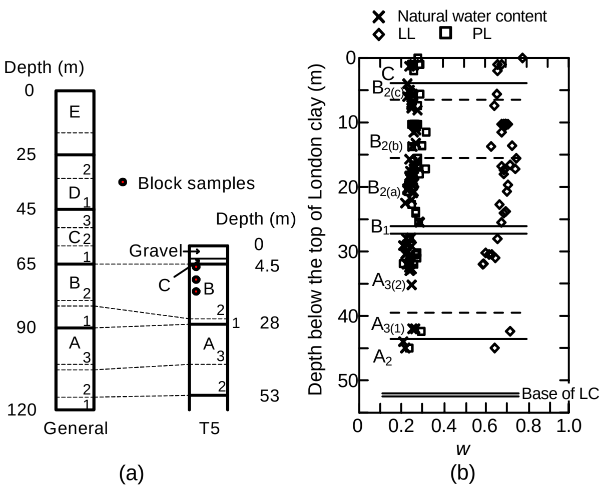

Fig. 4 London clay (LC) stratigraphy (King, 1981; Hight *et al*., 2003) (a) and liquid limit (LL), plastic limit (PL), and water content (*w*) profiles for Heathrow Terminal 5 (T5) site (Gasparre *et al*., 2007b) (b)

In Fig. 4a, full succession of LC is shown on the left, and T5 series A to E is shown on the right, with block sampling levels

Advanced laboratory testing of the type undertaken at T5 is only worthwhile if high quality samples are available. Three sampling styles were applied in the study described. Firstly, 100 mm outside diameter (OD) thin walled tube samplers were employed to provide cores that could be extruded rapidly on site, before any pore pressure equalisation could take place across their width, allowing suction measurements to be made with special probes (Ridley and Burland, 1993) that gave information on the in-situ mean effective stress, and hence K_0 (Hight *et al*., 2007). While these samples could also be used for index tests and descriptions, they were considered unsuitable for mechanical testing. The advanced testing described in the above previous studies was focused primarily on carefully taken (and preserved) rotary cores and on block samples retrieved from the stepped T5 deep excavations. The Geobore S wireline triple core rotary technique was applied, with carefully controlled drilling muds. The 100 mm outside diameter samples were preserved on site immediately after retrieval to the surface by removing all drilling mud and softened material, and then sealing in layers of plastic film and wax. The block sampling was conducted by hand-sculpting cubes with approximately 300 mm side widths, which were preserved in a similar manner to the rotary cores, being sealed in film and wax before being set in polyurethane foam which expanded as it was placed to fill specially made wooden boxes. Later laboratory specimen preparation commenced by cutting smaller prisms from the protected composite blocks with a powered band-saw, re-sealing the main blocks carefully after each such cut. The cut prisms of mudrock could then be trimmed with a soil lathe to produce cylindrical samples for triaxial or hollow cylinder apparatus (HCA) testing.

The main focus of the remainder of this paper is the advanced mechanical testing performed on the samples. Comprehensive programmes were conducted on both intact samples and reconstituted specimens. Readers are referred to Gasparre *et al*. (2007a) for descriptions of the fabric studies involving scanning electron microscope (SEM) and other techniques, and to De Freitas and Mannion (2007) for an account of the micro-palaeontology studies that correlated the abundant micro-fossils present to the geological setting with the stratigraphy of the sediments.

3 Advanced laboratory testing

The key aspects of mechanical behaviour investigated through laboratory testing were:

1. The effects of micro- and macro-structure developed by the mudrocks over geological time on their compressibility, yielding and shear strength characteristics, as manifested in triaxial and oedometer experiments on samples with appropriate dimensions;

2. How the same features affect the mudrock's anisotropy, considering stiffness behaviour from very small to moderate strains and the potential anisotropy in shear strength;

3. The brittleness of the clay, which can soften rapidly from relatively strong peak resistances to low residual strengths once shear bands develop with an oriented micro-fabric;

4. How field and laboratory measurements of soil stiffness compare, and the relationships between the behaviours of intact and reconstituted specimens;

5. The effects of recent stress history and ageing on the stiffness and shear strength characteristics;

6. Based on the above, how geological structure and age affect the properties of the mudrock sediments present at Heathrow T5.

Naturally, the study required large numbers of routine oedometer and index tests, which are reported in the previous studies mentioned above. However, we focus below on the more advanced stress path triaxial tests and HCA tests conducted by Gasparre (2005), Ann-Minh (2006) and Nishimura (2006). These works provided detailed profiles through the full sequence of sediments, working with the rotary core samples, as well as conducting intensive studies on multiple sets of similar samples cut from the numerous block samples taken at depths of 5.2 and 10.5 m below original ground level.

3.1 Triaxial test characteristics

Most of the triaxial testing was conducted on 100 mm OD, 200 mm high samples in hydraulic stress path cells built by Imperial College following the principles set out by Bishop and Wesley (1975). In the current design illustrated in Fig. 5, the upper cell body fits over the internal tie rods and top plates, so easing sample assembly and dismantling. The automated control system employs both electro-pneumatic stress actuators and hydraulic pump 'constant rate of strain' systems, and a very flexible computer control and data logging system is applied. High resolution linear variable differential transformer (LVDT) based local axial and radial strain sensors were installed that had been developed at Imperial College following the scheme initially proposed by Cuccovillo and Coop (1997). Dual axis bender element transducers were employed to assess the shear wave velocities V_{hv} and V_{hh} non-destructively at multiple stages during the tests, employing devices built at Imperial College London. A mid height pore pressure probe was employed to assess the degrees of excess pore-pressure development applying during all drained and undrained test stages, following the original proposal of Hight (1982). Similar, but smaller cells were deployed to test some 38 mm OD, 76 mm high samples, and a limited number of high pressure triaxial experiments were also performed (Gasparre, 2005).

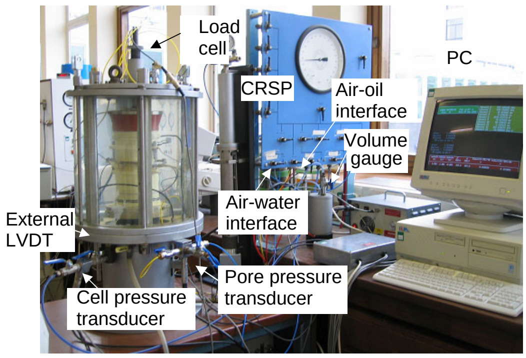

Fig. 5 Triaxial stress path equipment developed at Imperial College for testing 100 mm OD, 200 mm high Terminal 5 London clay samples
Note on sample local strain, bender element and pore-water pressure probe instrumentation: CRSP signifies constant rate of strain pump

The larger specimens were statistically more likely to include fissures that could be mobilised as failure approached. Scale effects apply to the shear strengths of any soil that is brittle, especially when preformed discontinuities are present. Careful inspection prior to testing identified any pre-existing fissures on specimens that might contribute to the eventual failure mechanism. Even tightly closed fissures have a clear influence on peak shear strength, by a pair of undrained compression tests on two similar 38 mm diameter specimens, where one contained favourably oriented fissures that were absent in the other (Fig. 6). All specimens developed rupture bands at or slightly before achieving peak strength. Those that remained intact until just before reaching peak strength showed far more marked post-peak brittleness. While the fissured and intact tests were separated during subsequent interpretation, the post-peak shear strengths mobilised by both types of samples at modest axial strains (5% to 10%) showed less variation, and gave ϕ' values (ϕ' is the angle of shearing resistance) that were comparable to those of reconstituted samples tested to large strains after normal consolidation, falling near to the interpreted critical state ϕ' values. Tightly closed fissures did not appear to affect the samples' initial stress-strain behaviour (at small to moderate strains) significantly.

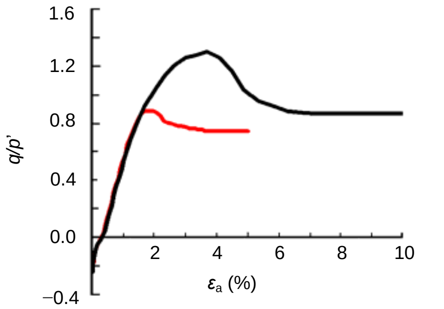

Fig. 6 Pair of triaxial tests on Terminal 5 London clay samples

ε_a is the axial strain; $q=(\sigma_1'-\sigma_3')$, and $p'=(\sigma_1'+\sigma_2'+\sigma_3')/3$, where q is the deviatoric stress, p' is the effective mean normal stress, and σ_1', σ_2', and σ_3' are three effective principal stress components. Stronger specimen is intact while weaker specimen fails on pre-formed fissure (Gasparre *et al*., 2007b)

The large volume of undrained tests performed in both triaxial compression and extension on 100 mm diameter specimens led to a series of envelopes, defining peak and post rupture effective stress shear strengths in both compression and extension for the various geological units. Fig. 7 summarises the interpretation given by Hight *et al*. (2007). Three main features are clear:

1. The intact peak strength envelopes of the deeper strata fall above those of the shallower layers, reflecting their greater degree of natural 'structure' and lower sensitivity to 'damaging' factors such as swelling after unloading, frost action or surface chemical weathering.

2. The post rupture strengths fall within a narrower band, indicating that brittleness increases with depth.

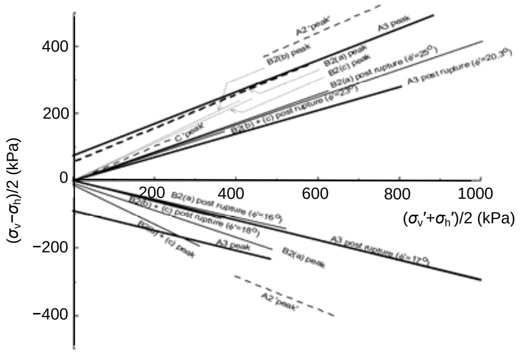

Fig. 7 Triaxial shear strength failure criteria in compression and extension for Terminal 5 London clay

σ_v and σ_h are the vertical and horizontal stresses, respectively; σ_v' and σ_h' are the effective vertical and horizontal stresses, respectively; variations between geological units and between peak and post rupture strengths are noted (Hight *et al*., 2007)

3. The failure envelopes for triaxial compression generally pass above those applying in triaxial extension.

The peak strength envelopes developed by intact samples in compression and extension delineate only parts of the samples overall yielding surfaces. The high pressure 'right hand' side of the yield surfaces were difficult to identify from high pressure probing tests because the mudrocks undergo a gradual structural break down and show progressive yielding similar to that seen in sands (Kuwano and Jardine, 2002; 2007). The sharp changes that can be seen with higher porosity natural soft clays as reported for Bothkennar clay (Smith *et al*., 1992) are not evident in either compression or swelling.

However, the peak strength envelopes can be interpreted by reference to the yielding behaviour of reconstituted samples (Smith *et al*., 1992), by normalising the stress variables with reference to p_e^*, the mean effective stress applying at the same void ratio on each unit's intrinsic compression line (ICL, established by oedometer tests performed on samples reconstituted at a water content of 1.25 times the liquid limit as proposed by Burland (1990)), as shown in Fig. 8. The local bounding surface as defined by Shibuya *et al*. (2003a; 2003b) and Jardine *et al*. (2004) which was established from undrained tests on isotropically consolidated reconstituted London clay, is also plotted. It is clear that the intact natural clay exists in a portion of normalised stress space that reconstituted samples cannot populate, showing the influence of the gradual geological ageing and partial lithification experienced by the Eocene London clay in-situ.

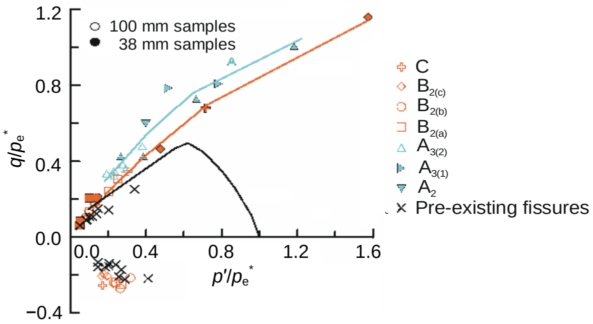

Fig. 8 Peak compressive shear strength failure criteria for units A and B normalised by the equivalent Hvorslev parameter p_e^*, and compared with local bounding surface (SBS*) for normally, isotropically, consolidated reconstituted London clay (Gasparre *et al.*, 2007a)

The suites of local strain transducers, mid-height pore pressure probes and bender element transducers mounted on the test specimens allowed the mudrocks' stiffness characteristics to be explored in detail. Behaviour became markedly in elastic once stress or strain increments exceeded very low threshold limits, typically around $p'/250$, or 0.001% respectively. Nevertheless, combining drained static axial and radial probing tests with dual axis bender element experiments allowed an analysis of the response seen within the quasi-elastic kinematic yield surface Y_1, which sits inside a second kinematic yield surface Y_2, as defined by Jardine (1992), Jardine *et al.* (2004), or Kuwano and Jardine (2002; 2007). The pre-Y_1 behaviour was described within the framework of cross anisotropic elasticity. Profiles of the independent stiffness terms, E_v', E_h', G_{hv}, and the corresponding cross-coupling Poisson's ratios, were derived that could be compared, as described later, with independent geophysical field measurements and data from HCA experiments. Multiple non-destructive bender element tests were also performed to track how small strain stiffness parameters varied as the effective stress state changed during consolidation and swelling stages, as shown in Fig. 9. It is interesting to note that the stiffness parameters assessed under in-situ stress conditions grew systematically, and nearly linearly, with the in-situ mean effective stress p_0'. However, stiffness appeared to be relatively insensitive to current effective stresses when samples were swelled or compressed to other values of p_0'. This behaviour implies that the small strain stiffness behaviour is at least partially fixed by the particular micro-fabric developed over geological time at each depth, proving relatively resistant to stress changes of the magnitudes imposed.

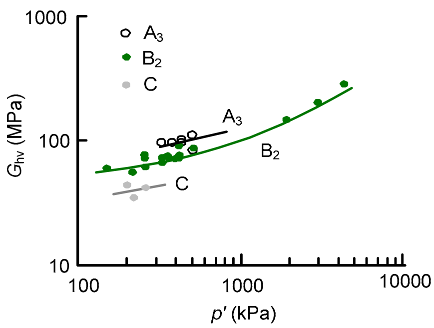

Fig. 9 Relationships between bender element shear stiffness G_{hv} and current p'

G_{hv}-p' trends are far steeper for tests under in-situ stresses; these show a nearly linear dependence with $G_{hv}\approx 360p'$ (Gasparre *et al.*, 2007b)

Triaxial probing test cycles demonstrated the limits to both Y_1 and Y_2 kinematic yield surfaces. Fig. 10 illustrates the linear and essentially recoverable response seen when very small stress increments are applied. The limits to this behaviour are outlined in Fig. 11, where the Y_1 kinematic yield surfaces observed in multiple tests are expressed by a single locus that retains, at all depths, a similar normalised size (in relation to the in-situ p_0') and shape around the in-situ stress point.

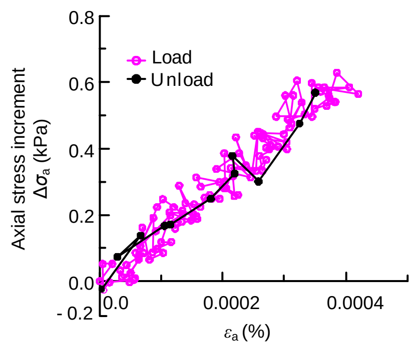

Fig. 10 Example of a drained small-strain probing stress cycle on a Terminal 5 London clay sample

Nearly linear and fully recoverable behaviour at strains <0.001% are shown (Gasparre *et al.*, 2007b)

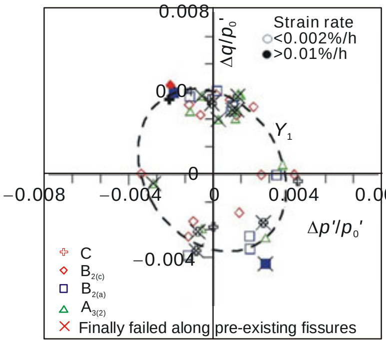

Fig. 11 Normalised Y_1 yield surface obtained from probing stress cycle tests performed on samples at all depths (Gasparre *et al*., 2007b)

The second kinematic yield surface Y_2, defines the points where strain increment vectors may deviate from their initial elastic pattern, the ratio of plastic to elastic strain increase sharply and the behaviour becomes markedly more time-dependent. When plotted in the same normalised coordinates as Fig. 11, the Y_2 kinematic yield surface appears around five times larger than Y_1 at most depths, and exhibits again a shape that is normalisable by p_0' (Fig. 12).

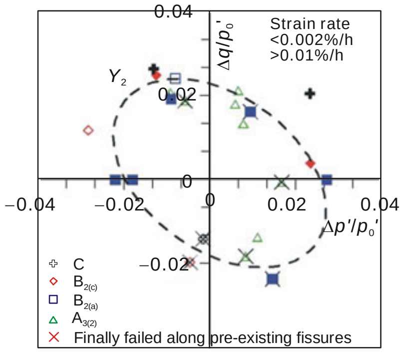

Fig. 12 Normalised Y_2 yield surface obtained from probing stress cycle tests performed on samples at all depths (Gasparre *et al*., 2007b)

Experiments taken to failure demonstrated the strongly non-linear response developed over the larger strain range, which is of primary engineering importance. A typical example from an undrained experiment is shown in Fig. 13, which also highlights the points at which Y_1 and Y_2 yielding is interpreted to have taken place. Local strain sensors are essential if the detailed stiffness behaviour of the mudrock is to be examined experimentally. The non-linear features observed with such gauges have important effects on ground movements and soil-structure interaction in practical engineering applications (Jardine *et al*., 2005).

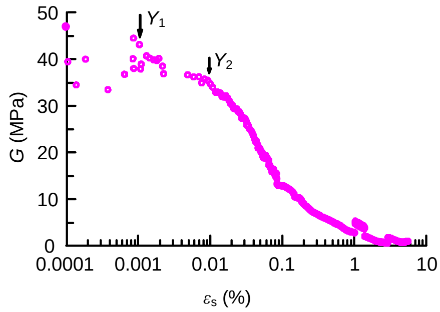

Fig. 13 Typical stiffness strain relationship from undrained triaxial test taken to failure on Terminal 5 London clay sample

G: shear stiffness; $\varepsilon_s=2(\varepsilon_1-\varepsilon_3)/3$, where ε_1, ε_3 are the principal strain components; and kinematic yield surface Y_1 and Y_2 yield points are also shown (Gasparre *et al*., 2007b)

3.2 Hollow cylinder experiments

Two HCA sets were employed on the T5 study, both applying the general operating principles illustrated in Fig. 14. However, they adopted slightly different sample dimensions (height, h; outer diameter, OD; inner diameter, ID), and had different sets of instruments. The smaller set illustrated in Fig. 15 was applied to both block sample testing and the profiling study on rotary cores. The apparatus, which was originally constructed in conjunction with Soil

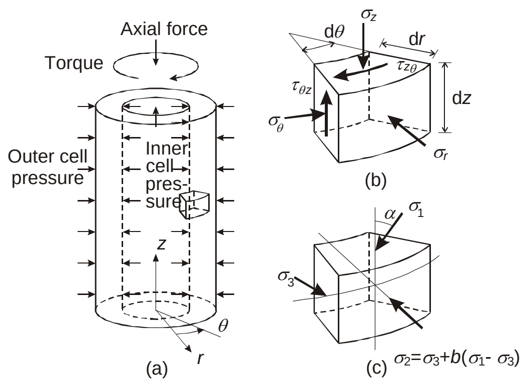

Fig. 14 Principles of hollow cylinder apparatus (HCA)

(a) The whole sample; (b) and (c) Elements taken from the sample wall. z, r, and θ are three variables in the cylindrical coordinate system, respectively; σ_1, σ_2, and σ_3 are three principal stress components; σ_z, σ_θ, and σ_r are the vertical, circumferential, and radial stresses, respectively; α is the angle between σ_1 and the vertical direction; $\tau_{z\theta}$ is the shear stress (Nishimura *et al*., 2007)

Dynamics Inc. of Kentucky, USA, was upgraded at Imperial College with new control systems for this study, as described by Nishimura (2006). The HCA set included a Hardin type oscillator, and could conduct non-destructive resonant column tests.

The second set illustrated in Fig. 16, followed the design of Jardine (1995) and tested larger specimens with OD=100 mm. The local strain instrumentation systems and control programmes applied in the T5 testing are described by Ann-Minh (2006).

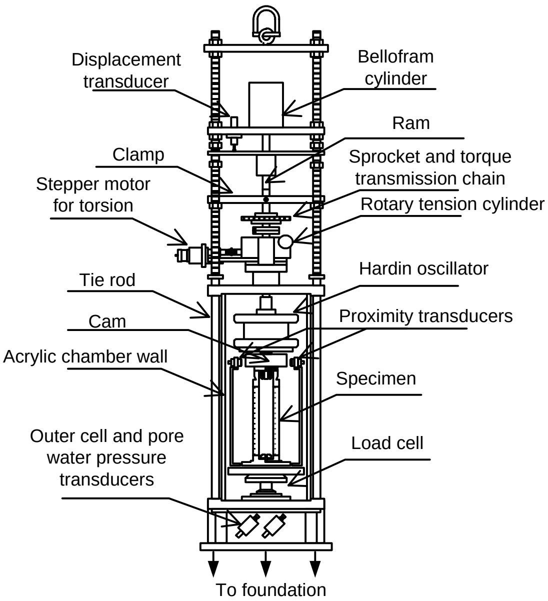

Fig. 15 General arrangement of Imperial College resonant column hollow cylinder apparatus (HCA) used to test London clay samples (OD=70 mm, ID=38 mm, *h*=170 mm) (Nishimura *et al*., 2007)

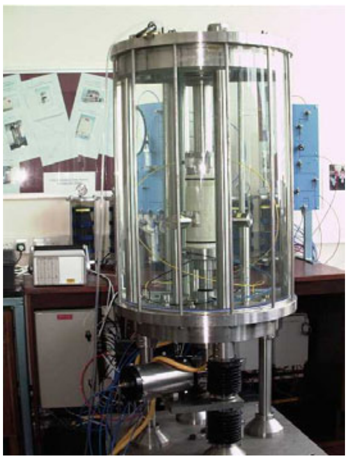

Fig. 16 Photograph of Imperial College Mark II HCA used to test London clay samples (OD=100 mm, ID=60 mm, *h*=200 mm) (Nishimura *et al*., 2007)

This equipment was used in an intensive study of the anisotropy in shear strength and stiffness of block samples taken at 5.2 m depth below original ground level, while a parallel study was performed with the smaller HCA on samples from 10.5 m below original ground level. Both sets of equipment were able to apply 4D stress paths, with computerised control being possible through either stress or strain controlled algorithms and actuators.

Great care was taken in sample preparation, following the procedure developed by Porovic (1995) in previous studies at Imperial College of mudrocks beneath a nuclear power plant (Hight *et al*., 1997) and pile behaviour in low over consolidation ratio (OCR) Pentre clay-silt (Chow, 1997). Albert *et al*. (2003) adopted the same technique for HCA tests on Bothkennar soft clay. The key steps are:

1. Select and expose the test specimen by either cutting a rectangular prism of clay from a preserved block sample by bandsaw, or removing the protection from a 100 mm OD preserved core sample.

2. Trim a cylinder to the OD (70 or 100 mm) and height (170 to 200 mm) required for the particular HCA with a soil lathe. Then encase in a two part metal split mould fitting that specimen size and trim parallel flat ends.

3. Place the encased sample into a metal working lathe, as shown in Fig. 17, and advance a series of drill bits, with increasing diameter, to form a central hollow cylindrical space over the full sample length. Then adopt a progression of boring tools, until a smooth wall has been achieved with the required ID of either 38 or 60 mm depending on which HCA is to be employed.

4. Remove from lathe, trim ends to final flat parallel condition, and then install in HCA.

Fig. 17 Intact mudrock sample preparation for hollow cylinder apparatus (HCA) testing
Internal hollow cylindrical cavities are bored out employing lathe and drill sizes are expanded (Ann-Minh, 2006)

Figs. 18 and 19 outline the steps taken in re-consolidating samples and performing undrained shearing under conditions where the direction of the major principal stress axis with respect to the vertical α and the intermediate stress parameter b defined in Fig. 14 could be varied, as desired. Additional

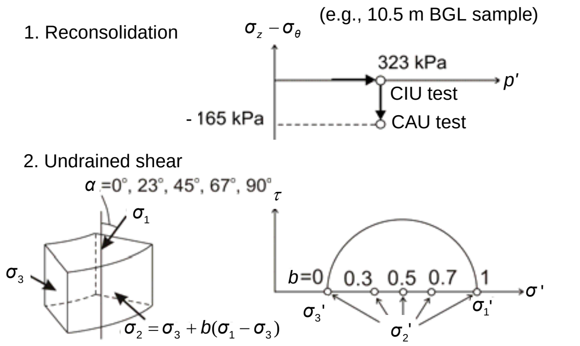

Fig. 18 Testing schemes for hollow cylinder apparatus (HCA) specimens (Nishimura *et al*., 2007)
CIU test: isotropic consolidated undrain test; CAU test: anisotropic consolidated undrain test; BGL: below ground level

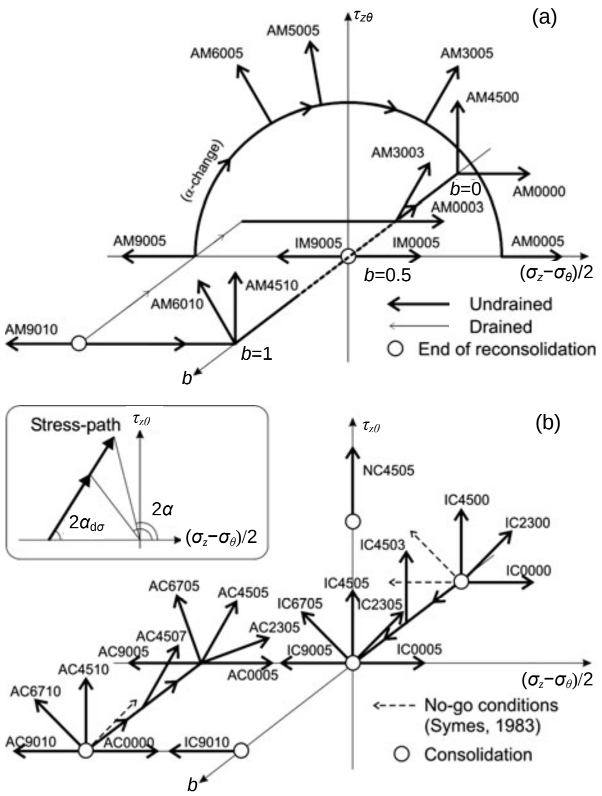

Fig. 19 Hollow cylinder apparatus (HCA) testing
(a) Scheme A: keeping α and b constant at single values for each shearing test; (b) Scheme B: keeping stress change direction $\alpha_{d\sigma}$ and b constant in each shearing test (Nishimura *et al*., 2007)

types of probing tests were conducted by Ann-Minh (2006) to investigate the response to small-strain uniaxial probing experiments, which yielded direct measurements of static cross anisotropic stiffness parameters, as well as monotonic tests to failure in which only one stress variable was varied, following re-consolidation to either an isotropic or K_0 initial stress state.

Alternative procedures were explored for the undrained shearing tests to failure that were performed to evaluate the shear strength anisotropies developed at 5.2 and 10.5 m depth below ground level (BGL). Scheme A, as outlined in Fig. 19a, was applied to the shallower samples. This scheme kept the direction α and parameter b constant during undrained shearing to failure. The application of Scheme B (Fig. 19b) to the deeper samples also involved keeping b fixed, but in these tests the orientation of the axis of major principal stress change was kept constant as the samples were sheared to failure without drainage.

Systematic sets of experiments were conducted in which a grid of b and α values were applied to nominally identical samples. These revealed that peak shear strength, as expressed by either maximum Mohr circle radius S_u or stress ratio q/p', is markedly dependent on both b and α. The general trend is illustrated in Fig. 20 by presenting failure data from Nishimura (2006)'s tests and on samples from 10.5 m depth. The close correspondence between these tests and those by Ann-Minh (2006) on shallower samples is demonstrated in Fig. 21, where data are presented from series at both depths where b was held constant at 0.5, representing nominal 'plane strain' failure conditions. It is evident that shear strength displays a minimum when α is around 55°. Shear strength is at a maximum under axial compression. The data show a mixed pattern when α>60° with most specimens developing higher strengths than in the minimum 'near simple shear' condition.

A limited set of parallel HCA tests on reconstituted samples demonstrated that at least part of the natural mudrock's anisotropy was related to the structure imposed on it by its geological history (Nishimura, 2006; Nishimura *et al*., 2007). The root causes for the mudrock's overall are interpreted as being:

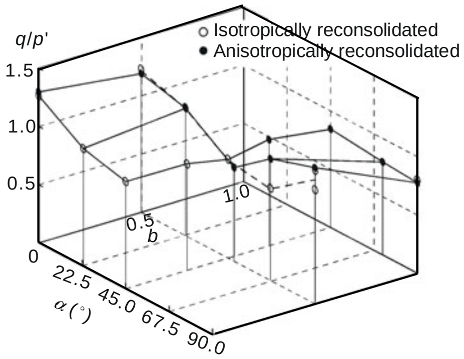

Fig. 20 Shear strength anisotropy from comprehensive test series run on block samples from 10.5 m depth
Values of α and b are both zero in triaxial compression, and are 90° and 1.0 in triaxial extension (Nishimura *et al*., 2007)

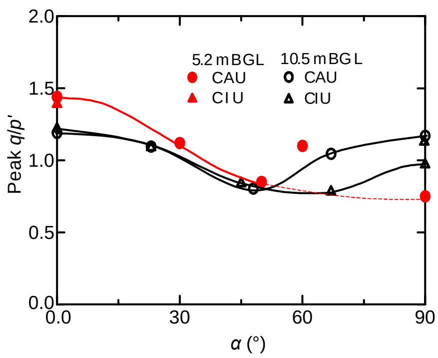

Fig. 21 Shear strength anisotropy for samples tested with *b*=0.5 (at near plane strain) from comprehensive test programmes run at two depths in different apparatus (Nishimura *et al*., 2007)
The dotted line is shown when subject to uncertainty

1. The highly aligned micro-fabric and system of inter-particle contacts, which affected the stiffness characteristics and determined the pre-failure undrained effective stress path inclinations;

2. The macro-fabric, as represented by the patterns of fissures and bedding features, which could be engaged preferentially as failure was approached with different σ_1 axis orientations, α.

Bearing in mind the second factor, the systems of pre-existing fissures were mapped carefully before testing, and their roles in each failure were tracked during testing, as reported by Nishimura *et al*. (2007).

Nishimura (2006) also conducted multiple simple shear tests on rotary core samples with HCA, covering a full range of depths after conducting torsional shear resonant column tests on samples reconsolidated to the anticipated in-situ K_0 stresses. HCA was constrained and controlled to ensure no vertical straining while it underwent monotonic torsional simple shear to failure. Keeping volume and average ID constant at the same time ensured that the radial and circumferential strains were also, on average, zero. Unlike conventional simple shear apparatus, the measurements made during HCA simple shear tests define the stress conditions fully at all test stages, allowing the paths to be tracked in terms of q, p', b and α; full 3D Mohr circles can also be drawn for any stage, and true S_u values were identified at failure. The simple shear failures developed with b values between 0.5 and 0.7 and α values around 50° to 65°.

The simple shear S_u-depth trends found by Nishimura (2006) are summarised in Fig. 22, along with data from triaxial compression and extension tests conducted on K_0 consolidated samples by Gasparre (2005) and Nishimura (2006). The interpreted trends show a clear hierarchy that is fully compatible with the intensive studies performed at 5.2 and 10.5 m that are illustrated in Figs. 20 and 21. Triaxial compression gave the highest peak values, which plot well above the upper bound of the simple shear and extension tests. A clear lower bound trend can also be seen, which appears to correspond broadly to the post rupture shear strength outlined earlier in Fig. 6, but with some variation between the stratigraphic units, as identified in Fig. 7. The degree of strength anisotropy is evidently marked, and

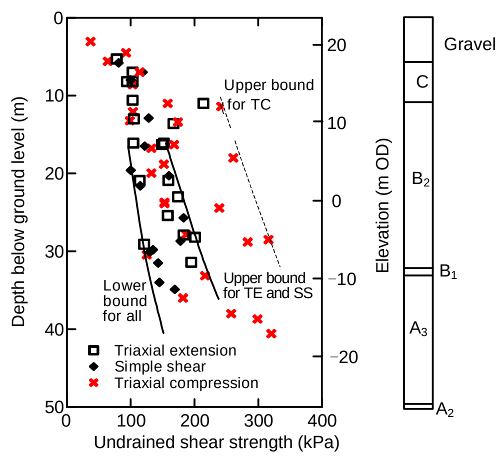

Fig. 22 Undrained shear strength (S_u) profiles for Heathrow Terminal 5 from K_0 reconsolidated samples of intact London clay tested in triaxial compression (TC) and extension (TE) and in hollow cylinder simple shear (SS) tests (Nishimura *et al*., 2009)

increases with depth. Considering possible implications for practice, shear strengths can be expected to be at a minimum over the basal shear zone of a potential instability mechanism such as that outlined in Fig. 21, where near simple shear conditions may apply. Laboratory compression strength tests are likely to overestimate the strengths available outside the active compression zone indicated in Fig. 21.

As mentioned above, Ann-Minh (2006) conducted static uniaxial HCA tests on samples from 5.2 m depth that gave sufficient information on their behaviour at very small strains to quantify all of the cross anisotropic elastic parameters. Kuwano and Jardine (1998) showed how combining static drained probing with dynamic bender element determinations allow cross anisotropic parameter sets to be determined. Gasparre (2005) applied this approach to the full range of depths. Nishimura (2006)'s resonant column (RC) tests, which were conducted on all of HCA specimens, gave further information on the G_{vh} mode of stiffness. These data are shown together, plotted against depth in Fig. 23. It is clear that horizontally applied stress probes invoke the stiffest response and that the ratio of E_h'/E_v' increases with depth. Further details are given by Gasparre *et al*. (2007b) and Hight *et al*. (2007) who also reported equivalent profiles from field geophysical testing. Note that the field testing and laboratory sample coring sites were set some distance apart, and have slightly different profiles as a result of works conducted 70 years ago at the site, the laboratory and field testing trends are generally compatible. The laboratory and field stiffness profiles correlate closely over the upper part of the profile and show slightly more divergent trends at depth. Bearing in mind the possible effects of frequency, and of micro-fabric leading to scale effects, it is reassuring that similar trends and conclusions could be drawn from measurements made by quite different techniques. While the mudrock's strong anisotropy at small to moderate strains has not been addressed in many practical engineering analyses, it can have a very significant impact on, for example, the shapes of the settlement troughs developed above tunnels formed in the London clay (Addenbrooke *et al*., 1997).

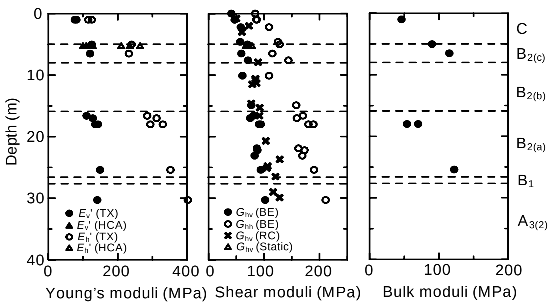

Fig. 23 Profiles of elastic (pre-Y_1) stiffness parameters from laboratory tests on Terminal 5 samples, showing strong cross anisotropy (Gasparre *et al*., 2009b)

4 Summary and conclusions

This paper has advocated the advantages of an integrated approach to the characterisation of geomaterial properties and behaviour, combining geology, in-situ testing, fabric studies, routine index experiments and advanced laboratory testing. It has also illustrated the potential of advanced laboratory testing in exploring features of behaviour, such as kinematic yielding, anisotropy in stiffness or shear strength that would otherwise be impossible to quantify. The case history selected to demonstrate these features was the London clay study performed for the new Heathrow Terminal 5 site.

Ten of the key points that have emerged are:

1. Sound geology is an essential starting point, along with an appropriate and consistent framework for test design, interpretation and application.

2. High quality rotary coring or block sampling is essential in mudrock deposits. Other techniques are required to deliver suitable samples when investigating different classes of geomaterials, especially soft clays and sands.

3. Advanced triaxial stress path testing, involving local strain sensors, bender element transducers and mid-height pore pressure probes allows many significant features of soil behaviour to be investigated, including potential anisotropy at very small strains, kinematic yielding behaviour, the effects of discontinuities on peak strength, post peak brittleness and the effects of geological structure.

4. The effects of geological history can be investigated and quantified through parallel experiments involving reconstituted clay samples.

5. Pre-existing discontinuities play a pivotal role in the shear strength of the mudrocks. Samples

must have sufficient volumes to capture these features representatively.

6. Hollow cylinder testing offers additional information on shear strength anisotropy, and stiffness anisotropy over both the elastic and non-linear ranges. The HCAs employed must possess suitable sample geometries, local instrumentation and control systems.

7. Great care is required in HCA experiments with sample preparation, sample reconsolidation, the choice of stress paths to be followed in test control.

8. The study described included HCA experiments involving constant b and α values, those with constant stress increment directions, tests in which only a single stress variable was altered, and kinematically controlled HCA simple shear tests.

9. The intensive triaxial and HCA programmes proved that the London clay has strong anisotropy in stiffness, which was confirmed by field geophysical testing, and shear strength. The mudrock is also highly non-linear over the strain range of interest, has markedly anisotropic shear strength, and exhibits a pronounced degree of brittleness.

10. All of the above features can impact significantly on the practical design and analysis of civil engineering works including shallow and deep foundations, tunnels and excavations, and the stability of slopes.

The approach described above for the London clay study is now being applied in a new set of studies aimed at providing equivalent information for the geologically older Gault, Oxford, Kimmeridge and Lias clays, whose outcrops in Southern and Central England are identified in Fig. 1.

Acknowledgements

Their supports are acknowledged gratefully, as are the crucial contributions of the five former colleagues at Imperial College who were co-authors on the source papers from which this paper has been developed: Dr. Nguyen ANN-MINH, Prof. Matthew COOP, Dr. Apollonia GASPARRE, Dr. David HIGHT, and Dr. Satoshi NISHIMURA. Other colleagues from Imperial College, Geotechnical Consulting Group (GCG) and other organisations who contributed valuably to the Terminal 5 project, and to earlier relevant studies include: Mr. Steven ACKERLEY, Mr. Cedric ALLENOU, Dr. Nebojsa KOVACEVIC, Prof. Reiko KUWANO, Mr. William MANNION, Prof. David POTTS, Dr. John POWELL, Dr. Jamie STANDING, Dr. Akihiro TAKAHASHI, and Dr. Lidija ZDRAVKOVIC.

References

Addenbrooke, T.I., Potts, D.M., Puzrin, A.M., 1997. The influence of pre-failure stiffness on the numerical analysis of tunnel construction. *Géotechnique*, **47**(3):693-712. [doi:10.1680/geot.1997.47.3.693]

Albert, C., Zdravkovic, L., Jardine, R.J., 2003. Behaviour of Bothkennar Clay under Rotation of Principal Stresses. *In*: Vermeer, Schweiger, Karstunen, Cudney (Eds.), International Workshop on Geotechnics of Soft Soils-Theory and Practice. Essen Verlug Gluckauf, p.441-447.

Ann-Minh, N., 2006. An investigation of the Anisotropic Stress-Strain-Strength Characteristics of an Eocene Clay. PhD Thesis, Imperial College London, London, UK.

Bishop, A.W., Wesley, L.D., 1975. A hydraulic triaxial apparatus for controlled stress path testing. *Géotechnique*, **25**(4):657-670. [doi:10.1680/geot.1975.25.4.657]

Burland, J.B., 1990. On the compressibility and shear strength of natural clays. *Géotechnique*, **40**(3):329-378. [doi:10.1680/geot.1990.40.3.329]

Chow, F.C., 1997. Investigations into Displacement Pile Behaviour for Offshore Foundations. PhD Thesis, Imperial College London, London, UK.

Cuccovillo, T., Coop, M.R., 1997. The measurements of local strains in triaxial testing using LVDTs. *Géotechnique*, **47**(1):167-171. [doi:10.1680/geot.1997.47.1.167]

De Freitas, M.H., Mannion, W.G., 2007. A biostratigraphy for the London clay in London. *Géotechnique*, **57**(1):91-99. [doi:10.1680/geot.2007.57.1.91]

Gasparre, A., 2005. Advanced laboratory characterisation of London clay. PhD Thesis, Imperial College London, London, UK.

Gasparre, A., Nishimura, S., Coop, M.R., Jardine, R.J., 2007a. The influence of structure on the behaviour of London clay. *Géotechnique*, **57**(1):19-31. [doi:10.1680/geot.2007.57.1.19]

Gasparre, A., Nishimura, S., Anh-Minh, N., Coop, M.R. Jardine, R.J., 2007b. The stiffness of natural London clay. *Géotechnique*, **57**(1):33-47. [doi:10.1680/geot.2007.57.1.33]

Harris, D.I., 2002. The Big Ben Clock Tower and the Palace of Westminster. Building Response to Tunnelling. *In*: Burland, J.B., Standing, J.R., Jardine, F.M. (Eds.), Case Studies from Construction of the Jubilee Line Extension. Thomas Telford, London, **2**:453-508.

Hight, D.W., 1982. A simple piezometer probe for the routine measurement of pore pressures in triaxial tests on saturated soils. *Géotechnique*, **32**(4):396-402. [doi:10.1680/geot.1982.32.4.396]

Hight, D.W., Jardine, R.J., 1993. Small Strain Stiffness and Strength Characteristics of Hard London Tertiary Clays. International Symposium on Hard Soils-Soft Rocks, Athens, Greece, p.533-552.

Hight, D.W., Pickles, A.R., De Moor, E.K., Higgins, K.G., Jardine, R.J., Potts, D.M., 1992. Predicted and Measured Tunnel Distortions Associated with Construction of Waterloo International Terminal. Wroth Memorial Symposium on Predictive Soil Mechanics, Oxford. Thomas Telford, London, p.317-338.

Hight, D.W., Bennell, J.D., Chana, B., Davis, P.D., Jardine, R.J., Porovic, E., 1997. Wave velocity and stiffness measurements of the Crag and Lower London Tertiaries at Sizewell. *Géotechnique*, **47**(3):451-474. [doi:10.1680/geot.1997.47.3.451]

Hight, D.W., McMillan, F., Powell, J.J.M., Jardine, R.J., Allenou, C.P., 2003. Some Characteristics of London Clay. *In*: Tan *et al*. (Eds.), Characterisation and Engineering Properties of Natural soils. Swets & Zeitlinger, Lisse, the Netherlands, p.851-908.

Hight, D.W., Gasparre, A., Nishimura, S., Ann-Minh, N., Jardine, R.J., Coop, M.R., 2007. Characteristics of the London clay from the Terminal 5 site at Heathrow Airport. *Géotechnique*, **57**(1):3-18. [doi:10.1680/geot.2007.57.1.3]

Jardine, R.J., 1992. Observations on the kinematic nature of soil stiffness at small strains. *Soils and Foundations*, **32**(2):111-124.

Jardine, R.J., 1995. A Design for a New Versatile Hollow Cylinder Apparatus. Internal Report, Imperial College London, UK.

Jardine, R.J., Potts, D.M., 1988. Hutton tension leg platform foundations: an approach to the prediction of driven pile behaviour. *Géotechnique*, **38**(2):231-252. [doi:10.1680/geot.1988.38.2.231]

Jardine, R.J., St. John, H.D., Hight, D.W., Potts, D.M., 1991. Some Practical Applications of a Non-Linear Ground Model. Proceedings of the 10th ECSMFE, Florence, Italy, p.223-228.

Jardine, R.J., Gens, A., Hight, D.W., Coop, M.R., 2004. Developments in Understanding Soil Behaviour. Conference on Advances in Geotechnical Engineering, Thomas Telford, London, p.103-207.

Jardine, R.J., Standing, J.R., Kovacevic, N., 2005. Lessons Learned from Full Scale Observations and the Practical Application of Advanced Testing and Modeling. Proceedings of the 3rd International Symposium on Deformation Characteristics of Geomaterials, Lyon, France, **2**:201-245.

King, C., 1981. The Stratigraphy of the London Clay and Associated Deposits. Tertiary Research Special Paper 6, Rotterdam, Backhuys.

Kovacevic, N., Hight, D.W., Potts, D.M., 2004. Temporary Slopes Stability in London Clay: Back Analyses of Two Case Histories. Advances in Geotechnical Engineering, Proceedings of the Skempton Memorial Conference on London, **3**:1-14.

Kovacevic, N., Hight, D.W., Potts, D.M., 2007. Predicting the stand-up time of temporary London clay slopes at Terminal 5, Heathrow Airport. *Géotechnique*, **57**(1):63-74. [doi:10.1680/geot.2007.57.1.63]

Kuwano, R., Jardine, R.J., 1998. Stiffness Measurements in a Stress Path Cell. Prefailure Behaviour of Geomaterials. Thomas Telford, London, p.391-395.

Kuwano, R., Jardine, R.J., 2002. On the applicability of cross anisotropic elasticity to granular materials at very small strains. *Géotechnique*, **52**(10):727-750. [doi:10.1680/geot.2002.52.10.727]

Kuwano, R., Jardine, R.J., 2007. A triaxial investigation of kinematic yielding in sand. *Géotechnique*, **57**(7):563-580. [doi:10.1680/geot.2007.57.7.563]

Nishimura, S., 2006. Laboratory Study of the Anisotropy of Natural London Clay. PhD Thesis, Imperial College London, UK.

Nishimura, S., Minh, N.A., Jardine, R.J., 2007. Shear strength anisotropy of natural London clay. *Géotechnique*, **57**(1):49-62. [doi:10.1680/geot.2007.57.1.49]

Porovic, E., 1995. Investigations of Soil Behaviour Using a Resonant Column Torsional Shear Hollow Cylinder Apparatus. PhD Thesis, University of London, UK.

Potts, D.M., 2003. Numerical Analysis: a virtual dream or practical reality? *Géotechnique*, **53**(6):535-573. [doi:10.1680/geot.2003.53.6.535]

Potts, D.M., Zdravkovic, L., 1999. Finite Element Analysis in Geotechnical Engineering: Theory. Thomas Telford, London, p.440.

Potts, D.M., Zdravkovic, L., 2001. Finite Element Analysis in Geotechnical Engineering: Application. Thomas Telford, London, p.427.

Potts, D.M., Kovacevic, N., Vaughan, P.R., 1997. Delayed collapse of cut slopes in stiff clay. *Géotechnique*, **47**(5):953-982. [doi:10.1680/geot.1997.47.5.953]

Ridley, A.M., Burland, J.B., 1993. A new instrument for the measurement of soil moisture suction. *Géotechnique*,, **43**(2):321-324. [doi:10.1680/geot.1993.43.2.321]

Shibuya, S., Hight, D.W., Jardine, R.J., 2003a. Four dimensional local boundary surfaces of an isotropically consolidated loose sand. *Soils and Foundations*, **43**(2):89-103.

Shibuya, S., Hight, D.W., Jardine, R.J., 2003b. Local boundary surfaces of a loose sand dependent on consolidation path. *Soils and Foundations*, **43**(3):85-93.

Smith, P.R., Jardine, R.J., Hight, D.W., 1992. On the yielding of Bothkennar clay. *Géotechnique*, **42**(2):257-274. [doi:10.1680/geot.1992.42.2.257]

St. John, H.D, Potts, D.M., Jardine, R.J., Higgins, K.G., 1992. Prediction and Performance of Ground Response Due to Construction of a Deep Basement at 60 Victoria Embankment. Wroth Memorial Symposium on Predictive Soil Mechanics, Oxford. Thomas Telford, London, p.581-608.

Symes, M.J.P.R., 1983. Rotation of Principal Stresses in Sand. PhD Thesis, University of London, UK.