1 Numerical assessment of the effects of end-restraints and a pre-
2 existing fissure on the interpretation of triaxial tests on stiff clays

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ABSTRACT

Conventional laboratory triaxial tests apply axi-symmetric boundary conditions to a cylindrical sample which has an axi-symmetric geometry. For a homogeneous sample this implies that the deformed shape of the sample should maintain an axi-symmetric geometry during the test. Consequently, the sample should deform in a barrelling mode and if slip planes develop they should define a cup and cone like failure surface. However, in many triaxial tests such behaviour is not observed, especially as failure is approached when a planar slip surface develops. Such a deformation mode is not axi-symmetric. One reason for this behaviour is that a fissure pre-exists in the sample. Employing hydro-mechanically coupled 3D finite element analyses, this paper investigates the influence of a single fissure in a triaxial sample of stiff clay on its behaviour throughout the test, focusing on the fissure position, orientation, strength and stiffness, in conjunction with the sample's end-restraints (rough or smooth). The effects are quantified in terms of the samples' overall stiffness and strength, indicating that the presence of a fissure can affect the very small strain stiffness, and that it has a significant effect on the strength of the sample, demonstrating that the conventional methods used to interpret laboratory tests may give unconservative results. The results also show a significant effect of the conditions at the top and bottom surfaces of the sample, where in particular the lateral restraint and rough ends introduce "bending" in the sample.

Key words: clays; laboratory tests; numerical modelling; shear strength; deformation

38 NOTATION

a, b, r, s	degradation parameters
B	non-linear permeability model parameter
c'	soil cohesion
c'_{fis}	fissure cohesion
$G_{0,r}$	maximum shear modulus at a reference mean effective stress
G_{sec}	secant shear modulus
H	sample height
$K_{0,r}$	maximum bulk modulus at a reference mean effective stress
K_0	coefficient of earth pressure at rest
k_s	shear stiffness of the fissure
k_n	normal stiffness of the fissure
k	soil permeability
k_0	permeability at zero mean effective stress
m_G, m_K	parameters defining the dependence of elastic stiffness on mean effective stress
p'	mean effective stress
p'_{ref}	reference mean effective stress
p_f	pore water pressure
q	deviatoric stress
q_i	initial deviatoric stress
q_{peak}	peak deviatoric stress
$R_{G,min}, R_{K,min}$	minimum normalised values of tangent shear modulus and tangent bulk modulus
α	fissure inclination
σ	total stress
σ_r	total horizontal stress
σ_v	total vertical stress
σ'_r	effective horizontal stress
σ'_v	effective vertical stress
σ'_n	effective normal stress
τ'_s	shear stress parallel to the fissure inclination
τ'_t	shear stress perpendicular to the fissure inclination
ε_a	axial strain
θ	angle defining position in the circumferential direction
ϕ'	angle of shear resistance of the soil
ϕ'_{fis}	angle of shear resistance of the fissure
ϕ'^{bc}_{fis}	back-calculated angle of shear resistance of the fissure
ϕ'^{inp}_{fis}	input angle of shear resistance of the fissure

INTRODUCTION

Fissures generally exist in overconsolidated stiff clays, as a consequence of geological processes such as stress release, tectonism, and weathering (Skempton et al., 1969; Chandler & Apted, 1988; Chandler et al., 1998). Since the first quantitative work of Terzaghi (1936), the influence of fissures on the behaviour of stiff clays has been extensively investigated both in the field and in the laboratory. The results of shear box tests on fissures in London clay samples reported by Skempton & Petley (1967) showed a weaker strength when the samples were sheared along the fissures compared to that of the intact clay. Similar behaviour was also observed in several undrained and drained triaxial tests performed on London clay where, besides a decrease of the overall shear strength, the samples tended to fail along the pre-existing fissures (Ward et al., 1965; Webb, 1966; Skempton et al., 1969; Marsland, 1971; Sandroni, 1977; Gasparre et al. 2007). A number of experimental investigations have also been carried out on the behaviour of other stiff fissured clays, such as those from southern Italy (Comegna & Picarelli, 2008; Vitone & Cotecchia, 2011; Nardelli et al.; 2016). These clays are intensely fissured, with samples containing many closely spaced fissures. Consequently, the behaviour of laboratory triaxial tests is not dominated by the presence of a single fissure and therefore they differ from that considered in this paper.

When triaxial testing is performed on London clay, fissures are generally not visible during the set-up process. To help identify the presence of fissures and analyse their influence on failure, Mohr's circle analysis has been widely adopted in the literature (Skempton et al, 1969; Burland, 1990; Gasparre et al. 2007). In this approach, when a triaxial specimen fails on a discrete shear plane, the stresses acting on that plane are estimated from the measured inclination of the shear plane by using a Mohr's circle. If the inclination of the shear plane is much lower or higher than that of the planes of the maximum stress obliquity obtained for the intact sample or/and the calculated strength on the shear plane is much smaller compared to that of the intact sample, failure is thought to be dominated by a pre-existing fissure. However, the influence of fissure characteristics, e.g. fissure position and orientation, on soil behaviour has not been generalised in the literature, since, as noted by Nishimura (2006), it is practically impossible to obtain soil samples with representative fissures.

Numerical tools have also been applied to help understand and reproduce the behaviour of fissured porous media. Most of the existing numerical models have been developed based on the concept of double porosity proposed by Barenblatt et al. (1960) for non-deformable rock formations and by Aifantis (1979) for deformable materials. Kahlili et al. (1999) further developed the theory and proposed a numerical model to simulate the consolidation of fissured clays, where the material is idealised as an assembly of porous blocks separated by a network of highly permeable and randomly distributed fissures. As the interaction between the clay blocks

and the fissures can only be implicitly accounted for inside one element, this type of numerical model cannot be directly adopted to model the influence of fissures in triaxial tests.

Despite these attempts to include the influence of the fissure on the behaviour of the sample, most of the results from samples that fail on a slip surface are still analysed assuming an axi-symmetric system of stresses. This paper investigates the influence of the existence of a single fissure and of end-restraints, in an otherwise homogeneous triaxial soil sample, on its stress-strain behaviour during typical compression tests using coupled hydro-mechanical 3D finite element (FE) analyses. The influence of the fissure position and orientation and of the type of sample's end-restraints on both stiffness and strength are interpreted and quantified through a set of numerical analyses.

NUMERICAL MODEL

The Imperial College Finite Element Program, ICFEP (Potts & Zdravkovic, 1999) was employed to perform all analyses presented in this paper. This employs a modified Newton- Raphson non-linear solver with an error controlled sub-stepping stress point algorithm. The objective of the numerical approach was to closely simulate the experimental procedure and the interpretation of triaxial experiments as conducted in the laboratory.

(a)

(b)

Fig. 1. Typical finite element mesh: (a) coarse and (b) fine

The soil samples were 100 mm in diameter and 200 mm in height. As stated by Vaughan et al. (1993), this sample size is more likely to contain a representative pre-existing fissure compared to other commonly used smaller sample sizes, and, therefore, has been widely adopted in triaxial tests on clay samples which might contain a pre-existing fissure (Gasparre et al., 2007; Hosseini Kamal et al., 2014; Brosse et al., 2017). It is also noted that it is easier to attach local instruments to large samples. A typical FE mesh used for the analyses presented in this paper is shown in Fig. 1(a). It consists of 1280 20-noded hexahedral elements representing the intact clay and 64 16-noded zero-thickness interface elements (Cui et al., 2019), placed along the plane AB, which represent the fissure. Variation of the orientation and vertical position of the plane AB enabled the simulation of a fissure with a different orientation and position. When analysing a sample with no fissure the interface elements were omitted and the mesh consisted of horizontal layers of hexahedral elements (i.e. no inclined boundaries). To investigate mesh dependency, additional analyses were performed using a mesh with inclined boundaries, as in Fig. 1(a), but without the interface elements, and also with a finer mesh, Fig. 1(b), which consisted of 8064 20-noded hexahedral elements and 224 16-noded interface elements. These different mesh options did not lead to any significant differences in the results, indicating that there was little mesh dependency. Both the solid and interface elements had displacement degrees of freedom at all element nodes and pore water pressure degrees of freedom only at their corner nodes.

SOIL BEHAVIOUR

As the objective of the analyses was to focus on the effect of a fissure and of end-restraints, and not on non-uniformities arising from clay's brittle behaviour, the material behaviour of both the intact clay and the fissure were assumed ductile. This also avoided the problem of mesh dependency (i.e. objectivity) that arises when analysing brittle material behaviour (Summersgill

et al., 2017). Both the intact clay and the fissure strength were characterised by the generalised 3D Mohr-Coulomb failure criterion. Values of $c' = 8$ kPa and $\phi' = 25^\circ$ were adopted for the clay, based on the analysis of temporary slopes at Heathrow Terminal 5, by Kovacevic et al. (2007), while the angle of shear resistance, ϕ' , for the fissure was varied, maintaining $c' = 0$. Zero dilatancy was assumed for both materials so that a distinct peak shear stress occurs under undrained loading, preventing the stress path progressing along the Mohr-Coulomb failure envelope (Potts & Zdravkovic, 2001).

The non-linear stiffness of the intact clay was modelled using the Imperial College Generalised Small-Strain Stiffness (IC.G3S) model (Taborda et al., 2016), which allows an independent calibration of both shear and effective bulk stiffness. This calibration was based on the data for London Clay adopted in Kovacevic et al. (2007). The parameters are presented in Table 1. For the fissure the shear and normal stiffness were set as $k_s = k_n = 10^7$ kN/m³ for the majority of the analyses. As it is difficult to experimentally obtain data from which these stiffness parameters can be determined, the values were selected to simulate stiff behaviour but are not too high as to cause numerical oscillations. While the above constitutive models are relatively simple, they enable the numerical results to be compared with those obtained from the equations used to interpret real triaxial tests shown later in the paper. To investigate the influence of the constitutive model on the predicted behaviour of the intact clay, additional analyses were performed using a more sophisticated modified two-surface kinematic-hardening model of Grammatikopoulou et al. (2006) and adopting model parameters for London clay calibrated by Avgerinos et al. (2016). Only essential results from these analyses are presented in the Appendix, but they demonstrate the same trends in the samples' stress-strain behaviour with respect to different end-restraints and the existence of a fissure. It is therefore thought that the conclusions of this study are not dependent on the constitutive model.

Table 1. Parameters used in the IC.G3S model for shear and bulk small strain behaviour of London clay

Shear	$G_{0,r}$: kPa	p'_{ref} : kPa	m_G	$R_{G,min}$	a : m/m	b
	61307.4	100	1.0	0.0544	7.98E-5	1.1051
Bulk	$K_{0,r}$: kPa	p'_{ref} : kPa	m_K	$R_{K,min}$	r : m/m	s
	30079.5	100	1.0	0.1337	1.23E-4	2.1990

The hydraulic part of the intact clay behaviour is characterised by the non-linear relationship between the permeability k and the mean effective stress p' :

$$k = k_0 e^{-Bp'} \quad (1)$$

where k_0 is the permeability at zero mean effective stress (m/s), and B is a model parameter (m^2/kN). In this study the values of $k_0 = 2.0 \times 10^{-9}$ m/s and $B = 0.007$ m^2/kN , as derived in Kovacevic et al. (2007) from field measurements, were adopted. A permeability of 8.1×10^{-9} m^2/s was adopted for the fissure. Since the units of permeability differ to those of the intact clay, the value adopted was chosen to approximately match the average permeability of the intact clay over the effective stress range in the analyses. It should be noted that changing the fissure permeability by plus or minus an order of magnitude had a negligible effect on the results of the analyses.

BOUNDARY CONDITIONS

To reduce the number of potential variables in the analyses, the drains and platens at the end of the sample and the rubber membrane around the circumference of the sample, which exist in laboratory equipment, were not modelled explicitly in the analyses. Instead, appropriate boundary conditions were applied to the surface of the sample.

To model the cell pressure acting on the sample a normal boundary stress was applied to the curved lateral surface of the specimen and, for the initial stages of a test, to the top of the sample. To model smooth (lubricated) ends at the top and bottom of the sample the displacements in the x - and y - directions (see Fig. 1(a) for coordinate axes) of the node located at the centre of the top of the sample were restrained from moving. In addition, to prevent rotation of the sample the x displacement of the node at position C on the top of the sample, Fig. 1(a), was also restrained from moving. All the other nodes on the top surface and all the nodes on the bottom surface of the sample were free to move unrestrained in these lateral directions. To model rough ends at the top and bottom of the sample, all nodes on these surfaces were restrained from moving laterally (i.e. in both the x and y directions). For both smooth and rough ends the vertical displacements (z direction) of all nodes on the bottom surface of the sample were restrained from moving, while all the nodes on the top surface of the specimen were given the same prescribed displacement during the shearing stages. At the initial stages of the test, when a stress is applied to the top of the sample to represent the change in cell pressure, the degrees of freedom associated with the vertical displacements of all the nodes located on the top of the sample were tied so that they all deformed vertically by the same amount (see Potts & Zdravković (1999) for details of this boundary condition). This is necessary when simulating the presence of a “rigid” top cap and porous stone.

As the analyses were hydro-mechanically coupled it was necessary to specify hydraulic boundary conditions. At all stages of the analyses the curved lateral surface of the specimen was assumed to be impermeable (i.e. no flow boundary). During application of a backpressure, changes in pore water pressure were applied to the nodes on the top and bottom surfaces of the sample. At all

other stages of the test the pore water pressure degrees of freedom on the top and bottom of the sample were tied, resulting in a uniform pore water pressure across the top and bottom of the sample, the magnitude of which resulted from the applied loading and undrained nature of the analysis. These boundary conditions simulated the presence of porous stones located at the extremities of the sample and, when applied along with the no flow boundary condition imposed on the curved surface of the sample, resulted in no pore water entering or leaving the sample.

INITIAL STRESS CONDITIONS

The triaxial testing procedure simulated in the analyses is based on the undrained triaxial tests presented in Gasparre et al (2007), who tested samples of London Clay. The initial stress state was chosen to be representative of the samples retrieved from Heathrow Terminal 5 at 20m depth. To account for the effect of gravity on both the pore water pressure and total vertical stress, bulk unit weights of 9.81 kN/m³ and 20.0 kN/m³ were taken for the water and saturated clay, respectively.

(a)

(b)

Fig. 2. Initial stress distribution in the sample: (a) effective stresses and (b) pore water pressure

At the beginning of the analyses the finite elements representing the intact clay were assigned the pore water pressures (suctions), vertical and horizontal effective stresses as indicated in Fig. 2. These give a zero total horizontal stress throughout the sample and a total vertical stress of zero at the top of the sample increasing with depth with a gradient of 20.0 kN/m³. For the fissure the pore water pressure and the effective stress distributions were specified to be consistent with those in the intact clay. This involved specifying the effective stress normal to the fissure plane, σ'_n , and two shear stress components parallel to the fissure plane - one parallel to the fissure inclination, τ'_s , and the other perpendicular to the fissure inclination, τ'_t . The expressions that ensure the compatibility between the intact clay and the fissure stresses depend on the inclination angle of the fissure, α , and are given as (da Silva, 2005):

$$\begin{aligned}\sigma'_n &= \frac{\sigma'_v + \sigma'_r}{2} + \frac{\sigma'_v - \sigma'_r}{2} \cos(2\alpha) \\ \tau'_s &= \frac{\sigma'_v - \sigma'_r}{2} \sin(2\alpha) \\ \tau'_t &= 0\end{aligned}\tag{2}$$

Fig. 3. Boundary conditions applied at different stages in the modelling of triaxial testing: (a) cell water filling; (b) sample saturation; (c) anisotropic consolidation to in-situ K_0 and (d) undrained shearing

First stage: Filling triaxial cell with water

The first increment of the analysis simulated the filling of the triaxial cell with water. This involved applying a cell pressure (total stress) to the cylindrical surface of the sample as indicated in Fig. 3(a) and a zero change in the backpressure at the sample ends. The time increment adopted at this stage was large enough to ensure drained conditions. For the analyses with smooth ends this resulted in the radial effective stresses becoming equal to 200 kPa constant with depth. However, due to gravity and the difference between the bulk unit weights of the water and the soil, the vertical effective stress increases with depth from a value of 200 kPa at the top of the sample with a gradient of 10.19 kN/m³. For the analyses with rough ends the radial effective stresses were 200 kPa over most of the sample height except near to the base of the sample where they reduced to 198 kPa. Also, near to the base of the sample the distribution of the vertical stress departed slightly from a linear distribution. These deviations are not surprising and are a result of the larger increase in total radial stress applied at the base of the sample and of the rough end.

Second stage: Restoring compressive pore water pressure

In the laboratory tests this stage is carried out to ensure saturation of the sample and to restore the in-situ compressive pore water pressure. In the analyses the sample is assumed to be saturated, however, this stage of the test was also considered so that compressive pore water pressures were obtained. It was simulated by simultaneously increasing the cell pressure and back pressure by 398.04 kPa, see Fig. 3(b). This was performed over increments 2 to 60 with each having a time step large enough to ensure drained conditions (i.e. 2 hours). For both the analyses with smooth and rough ends the effective stresses did not change during this process.

Third stage: Anisotropic consolidation

Fig. 4. Stress path during anisotropic consolidation (third stage)

In the laboratory this stage of the test involves reconsolidating the sample to its estimated in-situ stress state. Based on the results presented by Gasparre (2005), this stress state was $p'=200$ kPa and $q=-75$ kPa (equivalent to a $K_0 = 1.5$) and was performed following the effective stress path labelled 1-2 in Fig. 4. In the FE analyses this was simulated by imposing a zero change in backpressure and changes in the total stresses on the top and on the cylindrical sides of the sample of -50 kPa and +25 kPa respectively (see Fig. 3(c)). These changes were applied proportionally over 60 increments (increments 61 to 120) each with a sufficiently large time step (2 hours) to ensure that no excess pore water pressures developed. At the end of this stage of the analyses the pore water pressures in the sample were 200 kPa at its top, increasing with depth with a gradient of 9.81 kPa. The lateral and vertical effective stresses at the centre of the sample for analyses with no fissure and either smooth or rough ends are shown in Fig. 5. Similar results were obtained for samples with fissures. For the analyses with smooth ends the radial effective stress is 225 kPa and constant with depth, while the vertical effective stress is 150 kPa at the top of the sample, increasing with depth with a gradient of 10.19 kPa. As the sample compresses during this drained recompression, the presence of rough ends modifies the effective stress

257 distributions near the extremities of the sample, as shown in Fig. 5. The effect of rough ends is
 258 also visible in Fig. 6, which shows the distribution of the mean effective stress, p' , at the end of
 259 this stage.

Fig. 5. Distribution of effective stresses along the central line of the sample at the end of anisotropic consolidation (third stage)

(a)

(b)

Fig. 6. Contours of mean effective stress in the central y-z plane of the intact sample at the end of anisotropic consolidation (third stage): (a) smooth and (b) rough ends

Fourth stage: Undrained shearing

This stage of the analyses involved undrained shearing. The pore water pressure degrees of freedom on the top and bottom surfaces of the sample were tied as described earlier, no change in cell pressure was imposed and vertical displacements were prescribed at all nodes on the top surface of the sample (see Fig. 3(d)). The uniform displacement change on the top of the sample and the associated time were prescribed incrementally and imposed an overall axial strain rate of 5% per day. A total axial strain of approximately 30% was reached at the end of this stage. The small strain stiffness was reset at the start of this stage of the analyses, to account for the effect of the change in the subsequent stress path direction.

POST-PROCESSING

The results of the shearing stages of the analyses are presented with a selection of plots, quantified by the interpretation of analysis results in a similar manner to that of a real laboratory test. The deviatoric stress, $q = (\sigma_v - \sigma_r)$, is obtained by adding the change in deviatoric stress resulting from the undrained shearing stage to the initial deviatoric stress q_i (i.e. -75 kPa). The change in deviatoric stress is obtained by dividing the sum of the vertical reaction forces at all nodes on the top surface of the sample (resulting from the imposed displacements during undrained shearing) by the original cross-sectional area of the sample (i.e. 0.00785 m²). The use of the original cross-sectional area is consistent with the conventional “small” displacement nature of the FE analyses in which all geometric integrations are with respect to the original mesh

dimensions. Additional analyses employing a “large” displacement FE formulation were also performed, in which the change in mesh geometry is accounted for, and the results were interpreted based on the area correction adopted in the laboratory tests (La Rochelle et al., 1988). The pore water pressure is that of the node located at position D in Fig. 1(a). This node is located on the cylindrical surface at the mid height of the sample at $\theta=90^\circ$ (see Fig. 1(a) for the definition of θ) and represents the pore water pressure measured by a probe usually employed in a laboratory test. The axial strain, ε_a , is determined as the average of three local strains, representing those measured by the LVDT local strain transducers. Each of the local strains was determined by taking the difference in vertical displacements, which occurred during undrained shearing, between two nodes on the cylindrical surface of the sample located at the same θ , with one node being at a third of the sample height below the top of the sample and the other at a third of the sample height above the bottom of the sample, divided by a third of the sample height (which represents the initial gauge length). The three local strains were located around the sample at $\theta=0^\circ$, 120° and 240° . The secant shear modulus was determined using $G_{\text{sec}} = (q - q_i)/(3\varepsilon_a)$.

EFFECT OF A PRE-EXISTING FISSURE

To demonstrate the influence of a fissure, analyses of the shearing stage of an undrained triaxial compression test on a sample containing a fissure, located at the mid-height of the sample with an inclination of $\alpha = 30^\circ$, are compared with similar analyses on a sample without a fissure. Strength parameters of $c'_{fis} = 0$ and $\phi'_{fis} = 14^\circ$ were adopted for the fissure so that failure could occur along it. Analyses with both smooth and rough ends were performed, providing a total of four reference cases.

Shear stiffness

The variation of the secant shear modulus, G_{sec} , with axial strain, ε_a , during shearing is shown in Fig. 7. Also shown on this figure is the “ideal” curve which is obtained by integrating the constitutive equations for the intact soil over the strain path associated with undrained shearing. The results from the analyses without a fissure produce similar stiffness degradation curves that agree well with the “ideal” curve, indicating that the analyses are accurate and that the different end-restraints have little influence (see Group 1 in Fig. 7). The results from the analyses performed with a fissure where a $k_s = k_n = 10^7$ kN/m³ also produce nearly identical degradation curves (see Group 2 in Fig. 7), however, at small strains, the predicted secant shear modulus is considerably smaller than the “ideal”. Also shown on this figure are results from two analyses using fine meshes, one with a fissure and one without, both with rough ends. They show

negligible differences to the results from the equivalent analyses using coarser meshes, confirming that these analyses are accurate and no refinement of the meshes is required.

Fig. 7. Variation of secant shear modulus with axial strain

Results from additional analyses in which the shear and normal stiffness components of the fissure were both increased, first to $k_s = k_n = 10^{10}$ kN/m³ and then to $k_s = k_n = 10^{12}$ kN/m³, are also presented in Fig. 7 (i.e. Group 3). Apart from the minor differences between the analyses using the fine mesh with $k_s = k_n = 10^{10}$ kN/m³ and the other analysis in this group at very small strains, the results of this group are indistinguishable from each other, indicating that any further increase in fissure stiffness is unlikely to produce significant changes. While intuitively one might expect that for infinite values of k_s and k_n an analysis should give identical results to that of the "ideal" curve, this is not indicated by the trend of the present results. It is unclear whether this is due to not being able to significantly increase the values of k_s and k_n above 10^{12} kN/m³ in the present analysis without invoking numerical oscillations (Day & Potts, 1994), or due to some other unknown numerical issue with zero thickness interface elements.

As it is difficult to measure the stiffness of a fissure in practice it is difficult to determine a realistic value. Some experimental results are given in Nardelli et al. (2016), in which an inter-particle loading apparatus was employed to study the contact behaviour of clay scales. Interpreting these results gives a fissure stiffness of $k_s = k_n = 2 \times 10^5$ kN/m³. The result from an analysis with this value for k_s and k_n is shown in Fig.7 (see Group 4) and it indicates a significantly smaller sample's shear modulus compared to that of the intact sample. This result does not agree with other

experimental data which does not indicate such significant differences in shear modulus between intact and fissured samples (Gasparre, 2005).

Overall, the results presented here indicate that, at small strains, the fissure could have a significant influence on the small strain stiffness determined from a laboratory triaxial test. Since the main focus of the paper is to evaluate the stress-strain behaviour of the samples, the reference values for the stiffness of $k_s = k_n = 10^7$ kN/m³ were maintained for all analyses.

Stress-strain behaviour

Fig. 8(a) shows the stress-strain curves for the four reference analyses and the “ideal” curve. They are all similar until failure is approached. Comparing the two analyses without a fissure, that with smooth ends gives a maximum shear stress $q_{\text{peak}}=212$ kPa (the same as the “ideal” curve), which is approximately 7% higher than that with rough ends ($q_{\text{peak}}=197$ kPa). The presence of the fissure results in a smaller peak shear stress, with a reduction of approximately 40% and 8% for the analyses with smooth and rough ends, respectively. In contrast to the no fissure cases, the analysis with rough ends produces a higher strength than the analysis with smooth ends. The same behaviour can be observed (see Fig.A-1 in the Appendix) when a more sophisticated modified two-surface kinematic hardening model (Grammatikopoulou et al., 2006), with parameters calibrated by Avgerinos et al. (2016), was adopted, indicating that the predicted influence of a pre-existing fissure is independent of the selected constitutive model. The stress-strain curve of the intact sample with rough ends obtained employing a “large” displacement formulation is also plotted in Fig. 8(a), in which the area correction assuming a “bulging” deformed shape, as presented by La Rochelle et al. (1988), was applied. Similar results were obtained using a “right cylinder” assumption. No significant difference was observed in the predicted peak deviatoric stress for the analyses with the “small” and “large” displacement formulation. However, a post peak strain-softening behaviour was interpreted in the latter analysis, due to the approximation introduced by the area correction.

(a)

(b)

Fig. 8. Comparison in predicted behaviour of fissured and intact clay samples with smooth and rough ends: (a) stress-strain curve and (b) pore pressure variation

Fig. 8(b) shows the evolution of pore water pressure with axial strain. The shapes of the predicted curves are very similar to the corresponding stress strain curves shown in Fig. 8(a). However, both the analyses with rough ends produce higher pore water pressures than their equivalent analyses with smooth ends.

From the shapes of the curves presented in Fig. 8 it is interesting to note that the analysis with a fissure and rough ends indicates significant strain hardening behaviour, which starts just before

reaching the peak shear stress predicted for the fissured sample with smooth ends and only reaches its peak after an additional axial strain of approximately 2.5%. For all other analyses an abrupt perfectly plastic type behaviour occurs.

Fig. 9. Deformed sample shape at the end of undrained shearing for: (a) intact sample with smooth ends; (b) intact sample with rough ends; (c) fissured sample with smooth ends; (d) fissured sample with rough ends and (e) fissured sample with rough ends but with tied displacements at the bottom end

To understand this difference in behaviour it is instructive to examine the deformed shapes of the samples at failure, which are shown in Fig. 9(a)–9(d). For the analysis with no fissure and smooth ends an approximate uniform lateral expansion accompanies the vertical compression (Fig. 9(a)). For the equivalent analysis with rough ends a “bulging” type of deformation occurs, with non-uniform lateral displacements occurring along the sample height and the largest displacements concentrated in the bottom half of the sample (Fig. 9(b)). The reason for this concentration is thought to be due to the combined effect of the rough bottom boundary, the gravity forces and the fact that the mean effective stress in the sample just before shearing is lower in this vicinity (Fig. 6). This deformed shape is similar to the typical “barrel” shape that is often observed in real triaxial tests that do not fail on a shear plane (Alshibli & Akbas, 2007). For both analyses with no fissure the deformed shape is axi-symmetric.

Fig. 9(c) shows the deformed shape for the sample with a fissure and with smooth ends. This clearly shows the sample failing along the fissure. For the case with a fissure and rough ends the sample also shows failure along the fissure, but it additionally exhibits a considerable amount of “bulging” and “bending” which can be seen in the solid elements in the vicinity of the fissure (see Fig. 9(d)). This “bending” arises due to the absolute horizontal restraint imposed on both the top and bottom surfaces of the sample. It is interesting to note that the same behaviour (i.e. “bending”) was also shown by the triaxial test data from Webb (1966) who investigated the behaviour of London clay from Ashford Common. A peak shear stress was observed by Webb (1966) at an axial strain of around 6-7% for a sample which failed on a pre-existing fissure with an inclination of 38°, while a higher peak shear stress was reached at a much lower axial strain of approximately 4% for an intact sample without a fissure. Therefore, while the present result is realistic and simulates the laboratory test, it differs from the case of smooth ends where the top and bottom of

the sample can move laterally relative to each other. In practice, perfectly smooth (lubricated) ends would be difficult to achieve and, in addition, the membrane would resist this relative movement to some degree. To quantify the effect of this horizontal restraint in the analysis with rough ends, the analysis was repeated with the lateral displacement boundary condition on the bottom surface of the sample changed from full fixity of the displacements in both the x- and y- directions to tying of all the displacements in these directions together. This resulted in a rough bottom end restraint, but with the ability to move relative to the top surface of the sample. In essence, this modelled a rough bottom platen that is on rollers so that it has no resistance to lateral movement. The deformed shape of the sample at failure predicted by this analysis is shown in Fig. 9(e), demonstrating that the “bulging” and “bending” of the sample has been considerably reduced. The stress-strain relationship from this analysis is added to Fig. 8(a) and while the peak strength is only slightly greater than that of the equivalent analysis which had the lateral movement of the bottom of the sample fixed, the strain hardening behaviour observed in the latter has now disappeared. The above aspects of behaviour are also reflected in the pore water pressure-axial strain plot (see Fig. 8(b)).

INFLUENCE OF THE FISSURE CHARACTERISTICS

Fissure strength

(a)

Fig. 10. Stress-strain curves for a fissured sample with various fissure strength for: (a) smooth and (b) rough ends

To investigate the influence of the fissure strength further analyses have been conducted while keeping the same position (i.e. mid-height) and inclination (i.e. $\alpha=30^\circ$) of the fissure as in the previous section. In these analyses the angle of shearing resistance on the fissure was varied from $\phi'_{fis}=10^\circ$ to 25° , with the upper limit being equal to that of the intact clay. For each fissure strength, analyses with smooth and rough ends were performed.

The stress-strain curves interpreted from these analyses are shown in Fig. 10(a) and Fig. 10(b), for smooth and rough ends, respectively. Also shown are the curves for a sample with no fissure (labelled “intact clay”). For fissure strengths of $\phi'_{fis} = 19^\circ$ and greater, the shear stress-strain curves and peak shear stress matched those obtained from the equivalent analyses with no fissure, indicating that the presence of the fissure had no effect on the samples’ behaviour for those values of the fissure’s strength. When $\phi'_{fis} < 19^\circ$ a decrease in the peak shear stress is observed, indicating that failure is triggered by the fissure and a shear plane is formed. For analyses with a fissure strength $\phi'_{fis} < 11^\circ$ the samples failed along the fissure during the anisotropic consolidation stage (third stage) of the test, before undrained shearing could be applied.

It should be noted that for the analyses with smooth ends, the same pre-peak behaviour is observed (see Fig. 10(a)) for the different fissure strengths, but the curves depart before reaching the peak if the sample fails along the fissure. For the analyses with rough ends (Fig. 10(b)), the

decrease in the peak shear stress, from that obtained with a sample without a fissure, is smaller than the equivalent analysis with smooth ends. For example, for a fissure strength $\phi'_{fis} = 14^\circ$ the reduction in peak shear stress is 8% and 38% for the analyses with rough and smooth ends, respectively. Fig. 10(b) also shows that the effect of “bending” on the shear stress – strain curve becomes more pronounced as the fissure strength reduces.

Fig. 11. Variation of peak shear strength with fissure strength

The peak shear stress values shown in Fig. 10 are plotted against the fissure strength in Fig. 11. When the fissure strength is low the analyses with rough ends produce higher peak shear stresses than the equivalent analyses with smooth ends. The difference between the peak shear stresses for rough and smooth ends reduces as the fissure strength increases and for high fissure strengths the trend reverses and a higher peak shear stress is predicted by the analyses with smooth ends.

Fissure position

(a)

(b)

Fig. 12. Stress-strain curves for a fissured sample with various fissure positions for: (a) smooth and (b) rough ends

To assess the influence of the fissure position on the shear stress-strain behaviour, further analyses were performed with the fissure centre located at distances of $H/5$, $H/4$, $3H/4$ and $4H/5$ above the bottom of the sample, H being the sample height. The fissure inclination ($\alpha = 30^\circ$) and its reference strength parameters ($c'_{fis} = 0$ kPa and $\phi'_{fis} = 14^\circ$) remained unchanged. In all cases considered the fissure passed completely through the sample and did not intersect its extremities. Analyses with both smooth and rough ends were conducted.

The results from these analyses are presented in Fig. 12. Nearly identical stress-strain curves are obtained for all the analyses with smooth ends. However, differences exist in cases with rough ends. As the fissure position moves away from the sample mid height towards either its top or bottom, $4H/5$ and $H/5$, the stress-strain curves exhibit less strain hardening, indicating less “bending” in the sample. However, all analyses give the same peak shear stress, although reach it at different values of axial strain.

Fig. 13. Fissure positions with respect to local strain ‘measurement’: (a) $H/2$; (b) $H/4$ and $3H/4$; and (c) $H/5$ and $4H/5$

Fig. 13 shows the relative positions of the fissure with respect to the position of the local strain measurements (1: $\theta=0^\circ$, 2: $\theta=120^\circ$ and 3: $\theta=240^\circ$). Looking in more detail at the local strain results and considering the analyses performed with smooth ends, all the local strain positions gave identical results throughout the analysis with the fissure positioned at $H/2$. For the analyses with the fissure positioned at $H/4$ or $3H/4$, two local strain positions (i.e. 1 and 3 for $H/4$ and 1 and 2 for $3H/4$) stopped straining when the peak shear stress was reached (i.e. point A in Fig. 12(a)), while the other position (i.e. 2 for $H/4$ and 3 for $3H/4$) continued straining and gave identical results, resulting in a lower average strain compared to that with the fissure positioned at $H/2$. For the analyses with the fissure positioned at either $H/5$ or $4H/5$ all three local strain positions stopped recording strains when the peak shear stress was reached. Consequently, as the average of the strains was taken to produce the curves in Fig. 12(a) then, strictly speaking, the results are only near identical up to peak shear stress after which they differ, but as the peak shear stress is constant this is not evident in the figure.

Fig. 14. Variation of local axial strains with increment number for a fissured sample with rough ends and positions at: (a) $H/2$ and (b) $4H/5$

For the analyses performed with rough ends the behaviour is more complex. For a better understanding, the local strain measurements from the analyses with the fissure positioned at $H/2$ and $4H/5$ are shown in Fig. 14(a) and Fig. 14(b), respectively, against the increment number. Up until increment 200 (labelled as positions B and C on Fig. 14(a) and Fig. 14(b), respectively) all three strain positions for each analysis give similar results. However, after this increment the measurements begin to diverge. The equivalent positions B (for $H/2$) and C (for $4H/5$) are also

marked on Fig. 12(b) and it is possible to observe that in both analyses the divergence in local strain measurements only occurs once the peak shear stress is reached. This was also the case for other fissure positions. These results imply that pre-peak results from the conventional local strain measurements are unlikely to be significantly affected by their absolute position in the θ direction.

Fissure inclination

To quantify the effect of fissure inclination analyses were performed with the fissure centre positioned at mid height of the sample, but with the inclination varied from $\alpha=0^\circ$ (i.e. horizontal fissure) to 52.5° . Fissure inclinations greater than 52.5° were not considered as they would have resulted in extremely distorted finite elements adjacent to the top and bottom surfaces of the sample. None of the inclinations considered resulted in fissures intersecting the top or bottom surfaces of the sample. The fissure strength parameters (i.e. $c'_{fis} = 0$ kPa and $\phi'_{fis} = 14^\circ$) remained unchanged in all analyses.

(a)

Fig. 15. Stress-strain curves for a fissured sample with various fissure inclinations for: (a) smooth and (b) rough ends

Fig. 16. Variation of peak shear stress with fissure inclination

The stress-strain curves derived from these analyses are presented in Fig. 15(a) and Fig. 15(b) for analyses with smooth and rough ends, respectively. All analyses with fissure inclinations smaller than $\alpha=22.5^\circ$, with either smooth or rough ends, produced curves that were indistinguishable from those derived from the equivalent (in terms of end restraint) analyses

without a fissure, indicating that failure was occurring in the intact soil rather than along the fissure. As the fissure inclination increased above $\alpha=22.5^\circ$ the peak shear stress reduced and failure occurred along the fissure. The change in peak shear stress with fissure inclination is shown in Fig. 16. The reduction in peak shear stress is greater for the analyses with smooth ends. For example, the analyses with a fissure inclination of $\alpha=52.5^\circ$ had peak shear stresses 50% and 28% smaller than the equivalent analyses with no fissure, for analyses with smooth and rough ends respectively. These findings agree with the experimental data presented by Sandroni (1977), who claimed that the specimens with a fissure inclined at an angle of $40-60^\circ$ exhibited a much lower strength compared to those that failed along a flatter fissure (e.g. the one with an inclination angle of 25°). The “bending” effect that occurs when rough ends exist also becomes more significant as the fissure inclination increases (see Fig. 15(b)). As observed in the previous section, when considering the fissure position the results of the three local strains were similar until peak shear stress was reached, after which they diverged.

DISCUSSION

Intact samples

When interpreting triaxial test results it is usually assumed that the vertical and radial effective stresses are the principal effective stresses and are uniform within the sample, with their magnitudes derived from the total stresses applied to the boundaries of the sample and the pore pressure measured by a transducer. Consequently, for the analyses presented in this paper, at the beginning of the undrained shear stage it would be appropriate to take $\sigma'_r=225$ kPa, $\sigma'_v=151.02$ kPa and $p_f=199.02$ kPa (giving $p'=200.34$ kPa), which are the values at the mid-height of the sample in the triaxial test with smooth ends (see Fig. 6). If these values are used as the initial stresses and the constitutive model used in the FE analyses is integrated over the strain path imposed during the undrained shearing stage of the triaxial test, the “ideal” triaxial test behaviour can be obtained. The results of such an analysis are shown in Fig. 8(a) and Fig. 8(b) in the form of stress–strain and pore water pressure–strain plots. They are indistinguishable from those obtained from the analysis performed without a fissure and with smooth sample ends. However, the match is not so good for the analysis with rough ends, as this analysis shows a lower peak shear stress and a spurious strain hardening behaviour. As shown previously, the latter effect can be eliminated by enabling the relative lateral movement between the top and bottom of the sample, but the lower peak stress remains. These results suggest that laboratory tests in which the top and bottom of the sample can move laterally relative to each other are preferable. In this respect it is interesting to note that Sandroni (1977) achieved a similar behaviour in his laboratory tests at Imperial College by having a top platen that contained ball bearings onto which the internal load cell rested.

(a)

(b)

Fig. 17. Contours of mean effective stress, p' , at the end of undrained shearing for the intact sample with rough ends: (a) 5%/day and (b) 20%/day

As the constitutive behaviour assumed in the present analysis is isotropic non-linear elastic perfectly plastic, with a zero dilation, there will be no change in mean effective stress prior to or at failure during the undrained shearing. For the samples without a fissure the peak shear stress (at failure) is therefore given by (see Potts & Zdravković (1999)):

$$q_{\text{peak}} = \left(\frac{c'}{\tan \phi'} + p' \right) \frac{6 \sin \phi'}{3 - \sin \phi'} \quad (3)$$

The peak shear stress given by the analysis without a fissure and with smooth ends was around 212 kPa, which is very close to the theoretical value of $q_{\text{peak}} = 214$ kPa for $p' = 200.34$ kPa, $c' = 8$ kPa and $\phi' = 25^\circ$. However, the value obtained from the similar analysis with rough ends was only 197 kPa. One of the reasons for this lower value is because at the start of the undrained loading the effective stresses were not uniform in the sample (see Fig. 5) and this resulted in p' values lower than the ideal value of 200.34 kPa in the bottom part of the sample where failure occurred (see Fig. 6). Substituting the value of $q_{\text{peak}} = 197$ kPa into Eq. (3) gives $p' = 183$ kPa which is very comparable to Fig. 17(a), where the distribution of p' at failure in this analysis is shown. This reduction in p' is possible as it is only the boundaries of the sample that prevent water to enter or leave the sample. Within the sample there can be flow of water which will depend on the hydraulic gradients established by the non-uniform deformation of the sample and its permeability. This could then result in volume changes within the sample. Fig. 18(a) shows the changes in volumetric strains which occurred in the analysis with rough ends during undrained shearing. It is interesting to note that both positive and negative volumetric strains occur and that if they are integrated over the sample there is no overall volume change.

(a)

(b)

Fig. 18. Volumetric strains occurring during undrained shearing of the intact sample with rough ends: (a) 5%/day and (b) 20%/day

Fig. 19. Distribution of pore water pressure down the centre-line of the intact sample at failure. The distributions at failure of pore water pressure down the centre-line of the sample are shown in Fig. 19 for the analyses without a fissure. Both the analyses with smooth and rough ends have distributions which vary hydrostatically with depth, indicating that they are in hydraulic equilibrium albeit with a higher magnitude in the case of rough ends. In laboratory triaxial tests the difference between the pore water pressure at the mid height of the sample and at either the

top or bottom of the sample is often used to confirm that an acceptable rate of shearing is being applied to minimise non-uniformities in the effective stress distribution in the sample. The present analyses indicate that, while such an approach confirms hydraulic equilibrium, rough ends result in a non-uniform distribution of total, and therefore effective, stresses within the sample (see Fig. 17).

Results from further analyses in which the rate of undrained shearing was changed to 1% and 20% per day are also shown. For the analyses with smooth ends the pore water pressure distributions plot on top of each other in Fig. 19 and other results from these analyses are independent of the rate of undrained shearing (not shown here for brevity). This is not surprising as the samples behave in a near uniform manner. In the analyses with rough ends the sample does not behave uniformly and a different distribution of pore water pressure is generated which could violate hydraulic equilibrium if the soil is not sufficiently permeable and/or the shearing rate is too high. The lack of hydraulic equilibrium is evident in Fig. 19 in the analysis with the fastest shearing rate, which, as a result, increases the non-uniformity in the distribution of both stress and strain at failure (see Fig. 17(b) and Fig. 18(b)). However, at failure all three analyses predict maximum shear stress and mean effective stress values within 1% of each other, indicating that it is the rough ends that have the major influence on the behaviour and not any lack of hydraulic equilibrium.

Fig. 20. Effective stress status at failure for intact samples with different initial mean effective stresses

As the intact soil has both a cohesion and angle of shearing resistance, it is not possible to back-calculate these values from a single analysis. Consequently, further analyses without a fissure

were performed, following the same procedure as outlined before, but changing the initial and boundary conditions such that samples had “ideal” mean effective stress values of 100.34 kPa and 400.34 kPa at the mid height of the sample prior to undrained shearing. The shear stress prior to undrained loading was also changed so that all samples had the same q/p' ratio equivalent to $K_0 = 1.5$. Analyses with both smooth and rough ends were performed and the resulting stress states at failure, along with those from the original analyses, are plotted in Fig. 20. Also shown on this figure is the Mohr-Coulomb failure line corresponding to the input strength parameters for clay $c' = 8$ kPa and $\phi' = 25^\circ$. All analyses produce failure stress states that are close to this failure line, implying that, despite the difference in end restrains, they produce the correct drained strength parameters. However, as noted above, undrained strengths are under-predicted for all analyses with rough ends.

Samples containing fissures

In the analyses of a homogeneous sample with smooth ends the radial effective stress σ'_r is constant with depth and the vertical effective stress σ'_v has only a slight increase with depth due to the sample's effective weight at any stage of the undrained shearing. If this slight variation in σ'_v is ignored then Eq. (2) indicates uniform stresses over any inclined plane within the sample. For a soil sample containing a fissure, this implies that the stresses acting on it, normal (σ'_n) and tangential (σ'_t), would be uniform over its surface. Consequently, if the failure criterion, $\sigma'_t = \sigma'_n \tan \phi'$, is satisfied at any point of the plane, it would imply an abrupt failure along the fissure surface. This was observed in all the analyses with smooth ends that failed on the fissure. With rough ends the stress conditions in the sample are not uniform, especially due to the sample's bending discussed previously.

Rearranging Eq. (2) and introducing the pore water pressure and total radial and vertical stresses corresponding to the end of the undrained shearing stage of the analyses, i.e. $p_{f,ult}$, $\sigma_{r,ult}$ and $\sigma_{v,ult}$ (these quantities are essentially what are measured in a triaxial test), and noting that a fissure is assumed to have no cohesion in this study, the following expression for back-calculating the angle of shearing resistance on a fissure, if it were present, is obtained:

$$\phi'_{fis}{}^{bc} = \arctan \left[\frac{(\sigma_{v,ult} - \sigma_{r,ult}) \sin (2\alpha)}{(\sigma_{v,ult} + \sigma_{r,ult} - 2p_{f,ult}) + (\sigma_{v,ult} - \sigma_{r,ult}) \cos (2\alpha)} \right] \quad (4)$$

This is the equation that would be applied in practice to obtain the fissure strength on a sample that clearly failed on a planar surface.

Fig. 21. Comparison of back-calculated and input fissure strengths for samples with a fissure inclination of $\alpha=30^\circ$ and which failed along the fissure

Fig. 22. Back-calculated fissure strength from analyses performed with different fissure inclinations but the same input strength of $\phi'_{fis}^{inp} = 14^\circ$ and which failed along the fissure

Substituting into Eq. (4) the total stresses and pore water pressure determined at the end of the analyses performed on samples with a fissure located at mid height, with an inclination $\alpha=30^\circ$ and with different values of input fissure strength, ϕ'_{fis}^{inp} , which caused failure on the fissure, it is

possible to obtain the back-calculated values of fissure strength, $\phi'_{fis}{}^{bc}$, presented in Fig. 21. For the analyses with smooth ends the $\phi'_{fis}{}^{bc}$ values agree well with the strength values input in the analyses, $\phi'_{fis}{}^{inp}$. However, for the analyses with rough ends the back-calculated strength is higher than the input strength.

Performing a similar exercise for the analyses on samples containing a mid-height fissure with different inclinations and with a constant input strength of $\phi'_{fis}{}^{inp} = 14^\circ$, results in the $\phi'_{fis}{}^{bc}$ values presented in Fig. 22. For the analyses with smooth ends the back-calculated strengths for all fissure inclinations considered are in agreement with the input strength value. However, for the analyses with rough ends the back-calculated strength is again much higher than the input strength. Also shown in Fig. 21. and Fig.22 are the results from the analysis with rough ends, in which the bottom and top of the sample could move relative to each other, showing that the back-calculated fissure strength also exceeds the strength input into the analysis.

The above results imply that using Eq. (4) to calculate the fissure strength from the results of a real triaxial test with rough ends is likely to lead to an overestimation of the shear strength. The discrepancies increase as the fissure strength decreases with respect to the strength of the intact clay and/or with an increase in the fissure inclination.

Fig. 23. Summary plot showing the influence of the fissure characteristics. Additional analyses were performed varying the input strength and inclination of a fissure located at mid height of the sample. This has enabled the summary plot given in Fig. 23 to be

constructed, which defines three zones. If the combination of input fissure strength and inclination falls into:

Zone 1 - the sample behaviour is unaffected by the presence of the fissure and represents that of an intact sample.

Zone 2 - the sample behaviour is affected by the presence of the fissure and failure occurs along the fissure.

Zone 3 - the sample fails along the fissure during the anisotropic consolidation stage of the test before it can be subjected to undrained shearing.

The derived delineation of the effect of the fissure characteristics in Figure 23 is verified with available experimental data from triaxial tests on London clay. The first set of data (Gasparre, 2005), marked with cross symbols and with identification codes 16.6iUC, 26.3UC, 27UC, and 23.7UC, represents samples that failed on a pre-existing fissure, for which the fissure strength was interpreted as $c'_{fis} = 0$ kPa and $\phi'_{fis} = 17^\circ$, together with their measured fissure inclinations. The second set of data, shown as a shaded box in Fig.23, represents the range of fissure strengths and inclinations reported by Sandroni (1977), also for samples that failed on a pre-existing fissure. Both sets of experimental data plot within Zone 2, which defines failure along a pre-existing fissure. Gasparre (2005) further reports on samples which prematurely failed along a pre-existing fissure during the anisotropic consolidation stage (e.g. sample 11.9DE), which confirms the existence of Zone 3 in Fig.23.

CONCLUSIONS

One reason why axi-symmetric triaxial samples subjected to axi-symmetric boundary conditions fail on a planar slip surface, which does not maintain an axi-symmetric geometry, is that a fissure exists in the sample. This paper has investigated the influence of a single fissure in an otherwise homogeneous triaxial sample of London clay, through the comprehensive simulation of typical undrained compression tests using coupled hydro-mechanical 3D FE analyses. In addition to the influence of the fissure position, inclination and strength, the effect of the sample's end-restraints on both the interpreted stiffness and strength of the soil are quantified. Analyses without a fissure (intact sample) have also been performed for comparison. The analyses results were interpreted as if they were real triaxial tests, in the sense that stresses were evaluated from the applied loads (cell pressure and vertical load) and from the pore water pressure at a point on the lateral boundary at mid-height of the sample, while the axial strains were interpreted from vertical displacements between the two nodes representing the transducer's gauge length on the sample's vertical boundary. The following main conclusions arise from these analyses:

1. For intact samples the small and medium strain stiffness behaviour and the drained strength parameters back-calculated from the analysis are unaffected by the end restraint and agree well with those input in the analyses. However, while the undrained strength back-calculated from the analysis with smooth ends agrees with the ideal behaviour arising from the constitutive model, the undrained strength from the analysis with rough ends is some 7% lower. This was shown to be a direct consequence of the rough ends.
2. For samples containing a fissure, the analyses with smooth ends showed that the behaviour of the sample was similar to that of the intact sample until the fissure strength was mobilised, followed then by an abrupt failure. For the analyses with rough ends the effect of the fissure was evident before failure occurred, with the stress-strain and pore pressure response being similar to that of the intact sample up until approximately two thirds of the peak shear stress, at which point the behaviour became more ductile until ultimate failure occurred on the fissure. This behaviour was shown to be a consequence of the inability of the bottom and top of the sample to move laterally with respect to each other, causing bending in the sample.
3. For the samples which failed on the fissure, it is possible to back calculate the fissure strength using a simple equation based on the assumption of a uniform stress state, as typically used to interpret real triaxial test results. For the analyses with smooth ends the back-calculated strengths agreed well with the input fissure strength. However, for the analyses with rough ends the back-calculated strengths exceed those input into the analyses by as much as 37%.
4. A practical interaction chart was developed from the numerical results, for assessing whether a sample will be unaffected by the fissure, fail along the fissure in shearing, or fail prior to shearing. This assessment is based on the fissure's drained strength and inclination.
5. Good agreement was found between the numerical predictions and existing experimental data, thus validating the developed numerical model and its ability to realistically simulate and interpret laboratory experiments

While this study makes a distinct contribution to the interpretation of triaxial tests on stiff clays, further investigations are needed to evaluate the effect of a pre-existing fissure and end-restraints on the strain-softening aspect of the stiff clay behaviour, in terms of both the softening rate and the ultimate strength post-softening.

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832 **APPENDIX A**

833 To verify that the stress-strain behaviour observed in the four reference cases of a triaxial test
834 investigated in this study was not the consequence of applying a relatively simple non-linear
835 elastic-perfectly plastic Mohr-Coulomb model, these analyses were repeated with an advanced
836 modified two-surface kinematic-hardening model of Grammatikopoulou et al. (2006). This is a
837 critical state model which enables both the non-linearity and plasticity of the real soil behaviour
838 to be reproduced inside the state bounding surface. The adopted model parameters for London
839 clay were calibrated by Avgerinos et al. (2016) for ICFEP analyses of Crossrail tunnelling in Hyde
840 Park, London. The behaviour of the fissure, located in the middle of the sample and inclined at
841 $\alpha = 30^\circ$ to the horizontal, was modelled in the same manner as that in the main study, adopting

842 the Mohr-Coulomb model with strength parameters $c'_{fis} = 0$ and $\phi'_{fis} = 14^\circ$, and normal and
843 shear stiffness $k_n = k_s = 10^7$ kN/m³.

844 The stress-strain behaviour predicted by this model for the four reference cases is shown in Fig.
845 A-1. The results indicate the same trend in the effect of end restraints and the presence of a fissure
846 on the mobilised shear stress, as that shown in Fig. 8(a) for the non-linear Mohr-Coulomb results.
847 It is therefore thought that the general conclusions of the study presented in this paper are not
848 dependent on the choice of the soil constitutive model.

849

850 Fig.A-1. Comparison in predicted stress-strain curve of fissured and intact clay samples with a
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852

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