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Modelling the time-dependent behaviour of London Clay

Modélisation du comportement dépendant du temps de London Clay

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ABSTRACT: This paper presents constitutive modelling of intact London clay subject to complex stress paths. An elastic viscoplastic constitutive model with isotach viscosity has been used to reproduce both stress – strain and time-dependent behaviour of the clay. Coupled consolidation finite element analyses were performed to simulate a loading path that included: (i) isotropic compression; (ii) drained anisotropic shearing in compression and (iii) intermediate stages of constant stress. The model is calibrated against a number of high quality triaxial and oedometer experiments. Simulations demonstrated that the current model implementation is able to satisfactory capture the overall creep behaviour of London Clay subject to complex stress paths using a consistent and calibrated set of parameters. The current work also demonstrates the need to implement nonlinear small strain behaviour to better capture the response of a stiff overconsolidated clay.

RÉSUMÉ : Cet article présente la modélisation constitutive de l'argile londonienne intacte soumise à des trajectoires de contraintes complexes. Un modèle constitutif viscoplastique élastique avec viscosité isotache a été utilisé pour reproduire à la fois le comportement de l'argile en fonction de la contrainte-déformation et du temps. Des analyses par éléments finis de consolidation couplée ont été effectuées pour simuler un chemin de chargement comprenant: (i) la compression isotrope; (ii) cisaillement anisotrope drainé en compression et (iii) étapes intermédiaires de contrainte constante. Le modèle est étalonné par rapport à un certain nombre d'expériences triaxiales et oedométriques de haute qualité. Les simulations ont démontré que la mise en œuvre du modèle actuel est capable de capturer de manière satisfaisante le comportement de fluage global de London Clay soumis à des trajectoires de contraintes complexes en utilisant un ensemble cohérent et calibré de paramètres. Les travaux actuels démontrent également la nécessité de mettre en œuvre un comportement de petite déformation non linéaire pour mieux capturer la réponse d'une argile surconsolidée rigide.

KEYWORDS: time-dependent, London Clay, modelling, creep, equivalent time

1 INTRODUCTION

Time-dependent behaviour of soils is critical in geotechnical design concerned with the long-term serviceability and stability of geotechnical structures. Being able to predict the magnitudes of settlement under existing structures (e.g. Rowe and Hinchberger, 1998, Karstunen and Yin, 2010, Bodas Freitas et al., 2015), or the enhanced capacity of the ground for re-development and re-use of existing sites (e.g. Lehane and Jardine, 2003, Zdravković et al., 2019), are the principal challenges of such design. A significant effort has been made internationally to study the time-dependent behaviour of soils. A wide range of researchers have investigated these time-related phenomena by means of the conventional oedometer apparatus due in part to its simplicity. As a result, a number of phenomenological models have been suggested to simulate the time-dependent deformations of soils under one-dimensional conditions. Works by Buisman (1936) and Taylor and Merchant (1940) led to the idea of "viscous flow" or "secondary consolidation" in soils. This was later extended by Bjerrum (1967) to incorporate time as an explicit variable in the framework of isotach viscosity, still under one-dimension.

Subsequent laboratory studies have investigated the soil's time and rate dependent behaviour under more general stress conditions. Most commonly, these studies have focused on characterising the response of the yield surface with applied strain rate (e.g. Tavenas et al., 1978, Graham et al., 1983, Leroueil et al., 1985). Results from these investigations suggest that the entire yield surface is rate dependent and increases in size with applied strain rate.

Various constitutive models have since been proposed to describe the time-dependent behaviour of soils which combine both the isotach framework and rate-dependency of the yield surface. The majority of these are of the elastic-viscoplastic (EVP) type, as viscous effects are assumed to only occur in the plastic strain, with many being based on the overstress theory proposed by Perzyna (1963). Such formulations require a definition of the "overstress function", $\Phi(F)$, in order to determine the magnitude of the viscous plastic strain. These models extend the behaviour observed in one dimension by assuming that the rate of increase of the plastic component of the strain tensor is a function of the "excess" stress (loading surface) above the static yield surface. Recent overstress type models have been shown to successfully capture the time-dependent behaviour of clays under more general triaxial conditions (e.g. Sivasithamparam et al., 2015, Rezania et al., 2016).

Two distinct frameworks currently exist which decompose the viscoplastic strain rate using the concept of instant and delayed compression: (i) Bjerrum's (1967) framework for the 1D compression of clay where instant deformation corresponds to the deformation that would take place simultaneously with the application of the effective stress increment, assuming that no hydro-dynamic lag occurs, and (ii) Yin and Graham (1999) where the instant deformation corresponds to the elastic time-independent deformation and that more generally the compression of a soil element should be related to an *equivalent* loading *time* rather than an absolute time or duration of loading.

In applying either of these concepts to a constitutive framework, the individual strain components specific to the formulation are dependent on inherent assumptions in the model. Recent investigations into the nature of the overstress function have revealed that a locus of constant $\Phi(F)$ must be adopted in order to correctly predict critical state conditions at large strains. The absolute magnitudes of viscoplastic strain component are a product of the overstress function and are commonly obtained by calibration against laboratory data from oedometer tests or triaxial tests under isotropic conditions (e.g. Kutter and Sathialingam, 1992, Yin and Graham, 1999).

In this paper the three-dimensional elastic viscoplastic constitutive model developed by Bodas Freitas et al. (2011) is calibrated against experimental data from intact London clay. A brief description of the characteristics of the London Clay from

Hyde Park is presented. The experimental programme and results obtained from oedometer tests are also presented in order to determine the parameters necessary to calibrate the numerical parameters. Simulations of three triaxial specimens subject to complex stress paths using parameters obtained from oedometer specimens are presented. Finally, a discussion of the simulations is presented to highlight the capabilities of an advanced elastic viscoplastic constitutive model to capture the observed behaviour of London clay.

2 DESCRIPTION OF LABORARY TESTS

A test programme investigating the viscous effects of London Clay was performed in conjunction with the Crossrail project (Standing et al., 2015). A series of advanced triaxial and oedometer tests were performed on high quality undisturbed specimens of London Clay to characterise the time-dependent nature of the soil. All samples were selected from similar depths, 21.10 – 28.50 m, and of the same geological formation, Division B2.

Three oedometer tests were performed on two unique specimens (2 of the specimens were lathed from a single larger core sample). Twelve loading stages were applied to each of the specimens with stress starting from 127 kPa to a maximum of 8400 kPa vertical effective stress. The $e - \log \sigma_v'$ relationship of the three specimens, as well as oedometer tests from Heathrow terminal 5 (T5) project (Gasparre, 2005) from a similar geological unit, are shown in Figure 1. Intrinsic compression curves, obtained from reconstituted specimens from nearby depths are also plotted. Difference between the intact and intrinsic compression curves indicate a degree of structure present in the natural material as noted by Gasparre et al. (2007). Consistency was generally observed between the slopes of the one-dimensional curves for the different specimens. Compressibility of the specimens from Hyde Park and Heathrow T5 suggest that the behaviour is similar for samples from the same lithological unit.

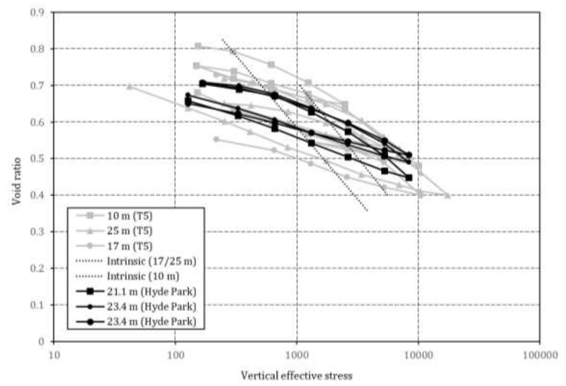

Figure 1. One-dimensional compression curves on intact specimens of London Clay

The void ratio--time relationship at each loading stage for a sample from 23.4 m (Hyde Park) is shown in Figure 2. Secondary compression index, C_α, was measured to vary with vertical applied load. The C_α value increased from a minimum of 0.005 at a vertical stress of 336 kPa to a maximum of 0.02 at 8382 kPa. Time to end-of-primary consolidation using Casagrande's logarithmic time method was relatively constant at all stages of loading for the intact soil specimens. The time to end-of-primary (t_{100}) is approximately 0.1-0.25 days for all stages. Results from the one-dimensional oedometer tests were used to determine the stiffness indices, λ, κ, time dependent compression index, C_α/ψ, and to estimate the overconsolidation ratio (OCR) value. The time-dependent parameter, ψ, was calculated using the C_α determined at 660 kPa. It was found that the time-dependent behaviour observed in the oedometer at 336 kPa was not representative of the true behaviour due to interaction between consolidation and sample swelling. The secondary compression indices at stages between 660 – 5283 kPa were similar in magnitude. The time-dependent behaviour shown in Figure 2 is consistent with that found for the other two specimens tested.

A total of three 70 mm dia. samples were tested in special triaxial apparatus designed to investigate long-term creep effects. Following set-up and an undrained saturation stage, all samples were reconsolidated isotropically back to the approximate in-situ mean effective stress. As all samples were from very similar depths, a common initial isotropic stress was selected. From this origin, three different stress paths were followed until drained failure of the sample.

Along each stress path, target stresses were achieved using computer control. Shearing was performed at sufficiently low stress rates (1 kPa / hr) so as to not generate excess pore pressure. At predetermined stress states along the specified stress path, stages of constant stress were defined; stages of constant stress were held for a minimum of 1000 mins. More details related to the triaxial tests can be found in Le et al. (2019).

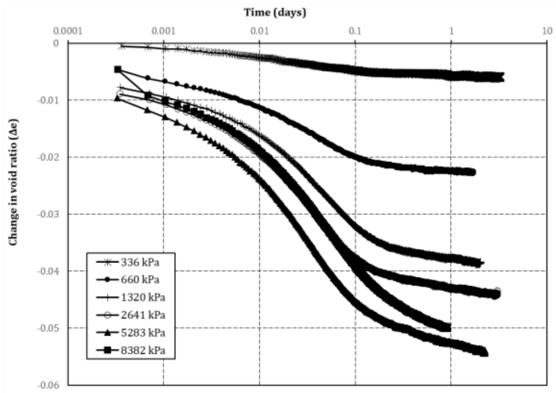

Figure 2. One-dimensional compression curves on intact specimens of London Clay

3 NUMERICAL SIMULATION OF LABORARY TESTS

3.1 *Constitutive model*

The constitutive model used to simulate a selection of triaxial tests on London clay (Le et al., 2019) is based on an overstress-based elastic viscoplastic model, developed in the equivalent time, ET, framework. Details of the model can be found in Bodas Freitas et al. (2011). The model was implemented in the finite element software ICFEP (Potts and Zdravković, 1999). Hereafter the constitutive model is referred to as the ET model and a brief summary of its formulation is presented.

The model assumes that the total deformation of a soil element, associated with the application of an effective stress increment, $\{\Delta\sigma'\}$, over a time increment, $\{\Delta t\}$, is decomposed into elastic and viscoplastic components:

$$\{\Delta\varepsilon^T\} = \{\Delta\varepsilon^{el}\} + \{\Delta\varepsilon^{vp}\} \quad (1)$$

where the elastic strain increment, $\{\Delta\varepsilon^{el}\}$, is instaneous and time independent and the viscoplastic strain increment, $\{\Delta\varepsilon^{vp}\}$, is time dependent and irreversible. The elastic strain increment can be determined by inverting the elastic constitutive matrix consisting of a stress dependent bulk modulus K (defined by Eq. 2, in which V is the specific volume, $\frac{\kappa}{V}$ is a material parameter and p' is mean effective stress) and a second elastic parameter that can be either Poisson's ratio, ν, or the elastic shear modulus, G.

$$K = \frac{V \cdot p'}{k} \tag{2}$$

It is noted here that the ET model is currently deficient in representing the nonlinear elastic small strain behaviour of soils.

Based on the overstress theory proposed by Perzyna (1963), the viscoplastic strain increment is evaluated as

$$\{\Delta\varepsilon^{vp}\} = \langle\Phi\rangle \left\{\frac{\partial g}{\partial \sigma'}\right\} \Delta t \tag{3}$$

where $\langle\Phi\rangle$ is a Macaulay bracket such that $\langle\Phi\rangle = \Phi$ if the stress state lies outside the yield surface, $\langle\Phi\rangle = 0$ if the stress state lies on or inside the yield surface; Φ is the viscoplastic scalar multiplier and g is the plastic potential, which can be different from the current loading surface. The value of Φ: (*i*) is determined using the concept of equivalent time (Yin and Graham, 1999); (*ii*) incorporates the hyperbolic creep function to describe the variation of volumetric viscoplastic strain with time under an isotropic effective stress state (Yin, 1999); and (*iii*) assumes that Φ is constant for all stress states located on a given loading surface (Bodas Freitas et al., 2012). From the latter assumption, the problem of determining Φ at a general state $(p', J, \varepsilon_{vol})$, where J is an invariant of deviatoric stress and ε_{vol} is volumetric strain, is reduced to calculating Φ at the equivalent isotropic state $(p' = p'_m, J = 0, \varepsilon_{vol,m})$; p'_m is the mean effective stress at $J = 0$ on the current loading surface and the geometric significance of the quantity $\varepsilon_{vol,m}$ is illustrated in Figure 3. The scalar multiplier Φ is calculated using Eq. 4.

$$\Phi = \frac{\psi_0}{V \cdot t_0} \cdot \left(1 + \frac{\varepsilon^{ref}_{vol,m} - \varepsilon_{vol,m}}{\varepsilon^{vp}_{vol,m,Limit}}\right)^2 \cdot \exp\left[\frac{V}{\psi_0} \frac{\varepsilon^{ref}_{vol,m} - \varepsilon_{vol,m}}{\left(1 + \frac{\varepsilon^{ref}_{vol,m} - \varepsilon_{vol,m}}{\varepsilon^{vp}_{vol,m,Limit}}\right)}\right] \cdot \frac{1}{\left|\frac{\partial g}{\partial p'}\right|_{\substack{p'=p'_m \\ J=0}}} \tag{4}$$

where t_0, $\frac{\psi_0}{V}$ and $\varepsilon^{vp}_{vol,m,Limit}$ are input model parameters. $\varepsilon_{vol,m}$ is the current volumetric strain at $p' = p'_m$; $\varepsilon^{ref}_{vol,m}$ is the volumetric strain on the reference time line at $p' = p'_m$; and $|\partial g/\partial p'|_{p'=p'_m, J=0}$ is the partial derivative of the plastic potential function, g, in relation to the mean effective stress, p', evaluated at the equivalent isotropic stress state. The absolute value function is introduced to ensure that Φ is always a positive quantity.

The loading and plastic potential surfaces are described independently by a flexible function that can reproduce a wide range of shapes, requiring three model parameters, α, μ and M. In the present study the loading and plastic potential surfaces, in $p' - q$ space (where q is triaxial deviatoric stress), are set to take the shape of to the Modified Cam Clay (MCC) ellipse (Roscoe and Burland, 1968), by adopting $\alpha = 0.4$ and $\mu = 0.9$. The influence of the intermediate principal stress is accounted for using the Matsuoka-Nakai failure criterion in the deviatoric plane.

At the start of the analysis procedure, the initial volumetric strain, ε_{vol}, is initialised as

$$\varepsilon_{vol} = \frac{\lambda}{V}\ln(p'_{mc}) - \frac{\kappa}{V}\ln(p'_{mc}/p'_i) \tag{5}$$

where p'_i is the initial mean effective stress and p'_{mc} the size of the loading surface corresponding to the largest normally consolidated stress state that the soil element has experienced.

During the analysis procedures ε_{vol} is updated by

$$\Delta\varepsilon_{vol} = \Delta\varepsilon_x + \Delta\varepsilon_y + \Delta\varepsilon_z \tag{6}$$

In addition, $\varepsilon_{vol,m}$ and $\varepsilon^{ref}_{vol,m}$ are calculated as

$$\Delta\varepsilon_{vol,m} = \varepsilon_{vol} + \frac{\kappa}{V}\ln(p'_m/p') \tag{7}$$

$$\varepsilon^{ref}_{vol,m} = \frac{\lambda}{V}\ln(p'_m) \tag{8}$$

The size of the yield surface is controlled by the parameter $\varepsilon^{vp}_{vol,m,Limit}$ that is assumed to be constant (see Figure 3). Assuming contractive volumetric strains are positive, the onset of viscoplastic deformation can be expressed as

$$1 + \frac{\varepsilon^{ref}_{vol,m} - \varepsilon_{vol,m}}{\varepsilon^{vp}_{vol,m,Limit}} > 0 \tag{9}$$

It is worth noting that the equivalent time equations, described above, completely define the soil behaviour: the elastic behaviour, the ratio of elastic to viscoplastic strain increments, and failure, as defined by the plastic potential.

3.2 Model parameters

The calibrated model was then used to simulate three triaxial experiments performed on high quality specimens of London Clay (from Unit B2) from Hyde Park (Standing, 2018, Le et al., 2019). The specimens were sheared under drained conditions, subjected to specific stress paths with intermediate stages of drained creep, as listed in Table 1. The experimental programme investigated the general time-dependent development of both volumetric and shear strains in a heavily overconsolidated material under a narrow, yet relatively unexplored range of stress states.

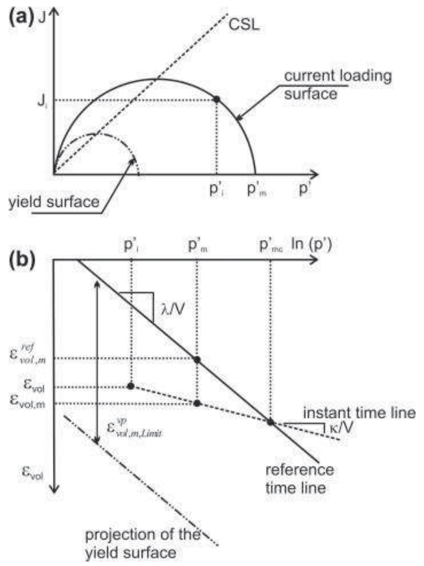

Figure 3. Model framework in general stress space

The derived model parameters used in this study are given in Table 1 (Giannopoulos, 2011). The first seven parameters are equivalent to the parameters required by the MCC model. The further three parameters, t_0, ψ_0/V and $\varepsilon^{ref}_{vol,m}$, describe the time and rate dependent response of the model. Model predictions were obtained by performing a coupled consolidation finite element analysis under axi-symmetric conditions. The mesh consists of eight noded isoparametric elements with four pore pressure degrees of freedom at the corner nodes, and two displacement degrees of freedom at both corner and mid-side nodes. The analysis was selected as it allowed for the combined effects of loading, consolidation and creep to be taken into account. The hydraulic boundary conditions were defined to reproduce free draining end platens.

As the coefficient of secondary compression, C_α, is commonly the only variable used in determining the time-dependent behaviour of soils, some assumptions are required to relate C_α to t_0, ψ_0/V and $\varepsilon^{ref}_{vol,m}$. In all analyses, the parameter $\varepsilon^{ref}_{vol,m}$ is set to equal $e_0/(1+e_0)$ as the model yields close to linear logarithmic creep law for the time intervals considered in this study.

Given that soil compressibility is often characterised based on 24 hour incrementally loaded oedometer tests, the parameter t_0 – real time associated with the reference compression line – is set equal to 1.0 day simply for convenience. Experimentally, the parameter ψ_0/V can be determined from the condition that the volumetric viscoplastic strain rate predicted by a linear logarithmic law with the adopted C_α value and of that predicted by the ET model at $t = t_0$ are the same. The value of ψ_0/V can therefore be related to C_α by:

$$\frac{\psi_0}{V} = \frac{C_\alpha}{V \cdot \ln 10} \tag{10}$$

Contrary to a linear logarithmic creep law, the slope of the compression curve following the dissipation of excess pore water pressure decreases with time from a value of ψ_0/V at $t_e = 0$ to zero at t_e = infinity, where the maximum volumetric compression due to time-dependent deformations is $\varepsilon^{vp}_{vol,m,Limit}$. It has been shown by Bodas Freitas et al. (2011) that when ψ_0/V is set equal to zero, then $\Phi = 0$ and no viscoplastic strains are predicted. In such cases, the soil state follows closely the reference compression line independent of the applied strain rate, and that, for practical purposes, the model can be taken as time independent.

3.3 *Analysis results*

Figures 4-6 compare the laboratory test data with the numerical predictions of the three drained triaxial compression tests on samples LC1, LC2 and LC4, respectively. Figure 7 maps the effective stress paths, peak friction angle envelope, critical state line and indicates the individual stages of creep for all three samples. The drained triaxial compression tests were simulated by applying the loading-time sequence performed in the experiment. The appropriate incremental changes of normal stress were applied at the top (axial stress) and at the vertical circumferential (radial stress) boundaries of the sample, ensuring values of the stress path gradients (J/p') were maintained between creep stages.

The predictions generally reproduce reasonably well both the strain-time and stress strain behaviour of the London Clay. All three samples were readily modelled using the same parameters except for the shear modulus, G, which was slightly different for the simulation of LC4. This was the result of the current inability of the model to account for a nonlinear small strain stiffness of soils and for the inherently anisotropic stiffness of London Clay.

As observed in Figure 4 and Figure 5, the model is capable of simulating the variation of viscoplastic strain rate with stress ratio for tests LC1 and LC2. It is particularly noteworthy that the strain developed during creep for all stages can be very well modelled in general stress space. Results from Figures 4a. and 5a. suggest that while the ET model accurately captures the evolution of strain at constant effective stress at lower stress ratios, the model tends to overpredict viscous strain once significant plastic strain has occurred. The strain experienced during creep at higher stress can be limited by reducing the time-dependent parameter ψ_0/V, however, this has the effect of reducing the overall strain developed during drained shearing as the viscoplastic strain component represents both the plastic and viscous deformations.

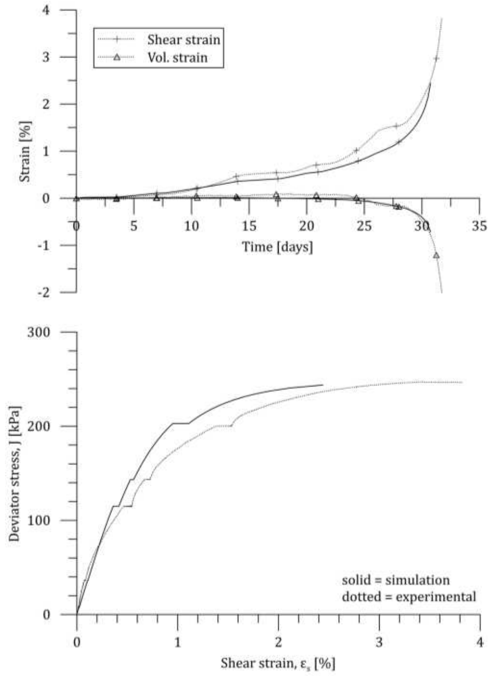

Figure 4. Drained triaxial compression test on London Clay LC1

The development of both shear and volumetric strain component with stress change and time is well simulated by the numerical model. For London Clay, it was found that a significant reduction in the plastic potential parameter (M_g), compared to the yield surface (M_p), is necessary to simulate the observed dilation of London Clay with increasing stress ratio. Specifically, as component of shear strain was significantly more than the volumetric strain, a lower plastic potential gradient was necessary to simulate the incremental viscoplastic strain observed from the experiment. It is suggested therefore that the lower plastic potential value mainly reflects the highly overconsolidated nature of the clay and that a kinematic yield surface may be necessary in order to capture the development of viscoplastic strain increments under drained loading conditions.

The model parameters reflect the sample's initial stress state, applied stress path and geological history (overconsolidated nature). It was also found that by assuming a constant Φ function on a given loading surface, the model was able to capture volumetric expansion during creep. This is essential as models which assume constant volumetric strain on a given loading surface would fail to capture dilation on the dry side of a yield surface.

It is also worth noting that both experimental data and numerical results for London Clay are consistent with the data reported by Tavenas et al. (1978) for a natural lightly overconsolidated clay. That is, the time-dependent shear strain component was found to contribute increasingly to the total strain measured during stages of constant load. For both lightly and heavily overconsolidated clays, experimental data suggests that the incremental viscous strain vector (orientation of the plastic potential) rotates towards the direction of the stress path until drained failure where shear strains completely dominates both experimentally and numerically. Significant rotation of the viscous strain increment found after the stress state exceeded $q/p' > 0.6 \cdot M_{cs}$.

Table 1. Model parameters for London Clay Unit B2

$M_p\,(-)$	$M_g(-)$	$\alpha_p/\alpha_g\,(-)$	$\mu_p/\mu_g(-)$	$\lambda/V(-)$	$k/V(-)$	G (kPa)	$\psi/V(-)$	$\varepsilon^{vp}_{vol,m,Limit}(-)$	$e_0(-)$	t_0(day)	k(m/day)	OCR
0.983	0.15	0.4	0.9	0.02	0.005	30000	0.00305	0.402	0.69	1.0	2.59E-4	3.65

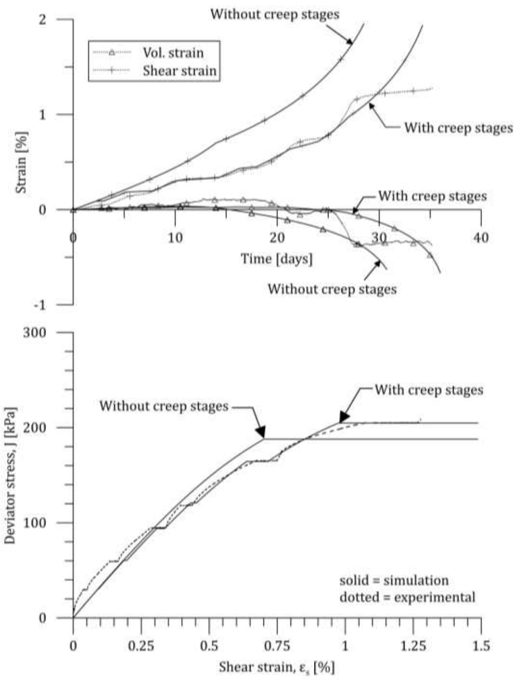

Figure 5. Drained triaxial compression test on London Clay LC2

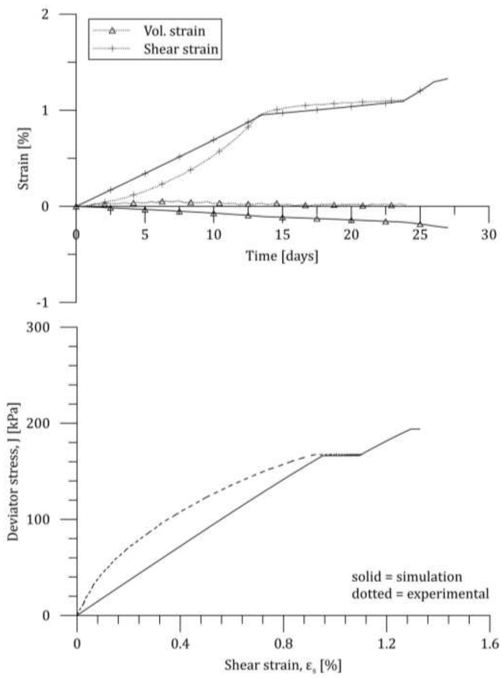

Figure 6. Drained triaxial compression test on London Clay LC4

While simulations were able to satisfactorily capture the time-dependent behaviour at low strain levels, deviation between numerical and experimental results were found at higher strains as the sample neared failure. To better understand the contribution of the time-dependent components, an additional simulation was carried out for LC2 with the removal of all creep stages. Results from the additional simulation are plotted in Figure 5. Three effects are noteworthy when comparing the two simulated test: (i) failure is reached at a lower deviatoric stress level for the simulation without creep stages compared to the simulation with, (ii) strain levels at the conclusion of the final loading stage are different between the simulation with and without creep stages, and (iii) failure initiates at a lower strain level for the simulation without creep. All three of these effects are a result of the limited contribution of the viscous component during the early stages of shearing (due to the reduced time associated with strain development). While these observations are not surprising, time is shown to significantly influence the measured strain development throughout the shearing process. To better capture the experimental behaviour, it would be necessary to account for the stiffness degradation of the material with strain in addition to modelling time-dependent behaviour.

3.4 Strain during creep stages

As highlighted by previous researchers (e.g. Tavenas et al., 1978, Yin, 1999), the strain-rate—time relationship is often more complex than the standard linear logarithmic function. More specifically, experimental data suggests that: (*i*) linear logarithmic functions often inaccurately capture viscous deformations as the "origin of time" (Leroueil et al., 1985) must be defined, and (*ii*) that while the general stress-strain-time equation proposed by Mitchell et al. (1968) closely match a range of soils subject to creep prior to creep rupture under both drained and undrained conditions subject to staged loading, functions which depict linear decay of strain rate fail to simulate the interaction of primary consolidation (dissipation of excess pore pressure) and viscous deformation for tests which apply slow drained stages with intermediate stages of creep.

Figure 8 plots the experimental and numerical predictions of both volumetric and shear strains with time during stages of constant stress for Test LC1. Experimental and numerical agreement are similar for Tests LC2 and LC4. For all cases, the ET model predicts reasonably well the experimental data. The ET model is capable of simulating the measured variation in rate of both shear and volumetric strain with time, as well as the relative increase in both strain invariants with stress ratio. It can therefore be said that the ET model has the ability to model increasing viscous strain-rate with stress level. Similar experimental findings have been presented by Kuwano and Jardine (2002) for artificial sands.

Numerical simulations suggest that the ET model recovers well both the magnitude and change in plastic potential during stages of constant stress. It can be seen that during stages of creep, viscous strain-rate is predicted to remain constant with time while experimental data demonstrate a gradual decrease, following a period of constant strain-rate. Both experimental data and numerical simulations suggest that as the stress state approached the failure plane, increased sample deformation led to a higher initial gradient of viscous strain-rate.

4 CONCLUSION

The investigations of time-dependent behaviour of soils have usually been focused on soft clays. The study presented here concentrated on the time-dependent behaviour of a stiff clay and investigated the ability of an advanced constitutive model to simulate a set of experiments conducted to explore the clay's response during periods of constant load. The applied constitutive model was formulated in the equivalent time framework with isotach viscosity. Based on the results of these numerical simulations it can be concluded that the model is capable of reproducing much of the complex volumetric and shear strain development with non-standard stress paths under drained conditions. The model is therefore adequate at simulating more complex stress paths experienced by soil units in practical applications where time-dependent behaviour is critical. While these simulations do highlight a deficiency of the current model formulation to represent adequately the nonlinear small strain behaviour of a stiff clay, the study demonstrated a satisfactory overall ability of the model to reproduce the creep behaviour of the clay.

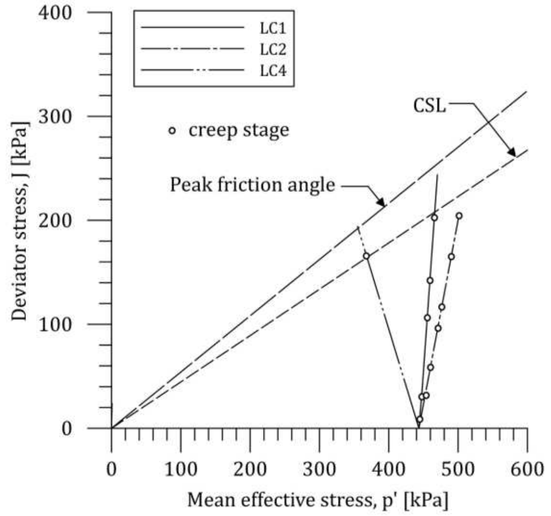

Figure 7. Drained triaxial compression paths on London Clay

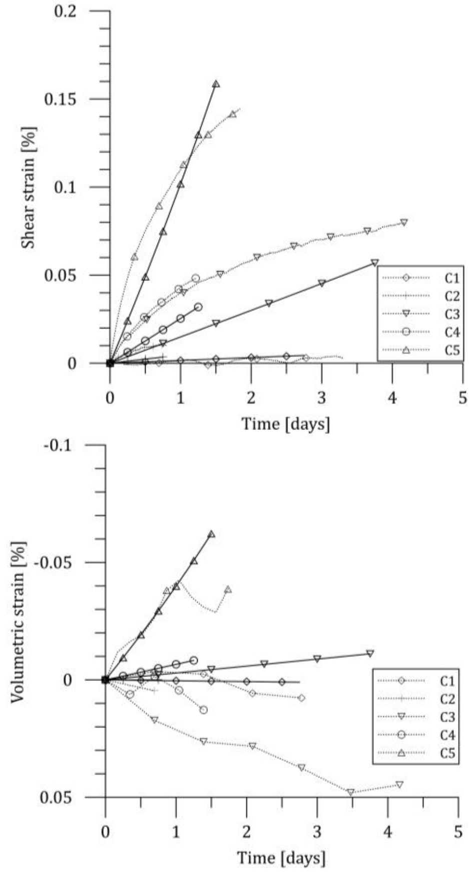

Figure 8. Measured and simulated shear and volumetric strain components during creep from Test LC1

5 REFERENCESS

Bjerrum, L. (1967). Engineering geology of Norwegian normally-consolidated marine clays as related to settlements of buildings. *Géotechnique,* 17 (2)**,** 83-118.

Bodas Freitas, T. M., Potts, D. M. & Zdravkovic, L. (2011). A time dependent constitutive model for soils with isotach viscosity. *Computers and Geotechnics,* 38 (6)**,** 809-820.

Bodas Freitas, T. M., Potts, D. M. & Zdravkovic, L. (2012). Implications of the definition of the Φ function in elastic-viscoplastic models. *Géotechnique,* 62 (7)**,** 643-648.

Bodas Freitas, T. M., Potts, D. M. & Zdravkovic, L. (2015). Numerical study on the response of two footings at Bothkennar research site. *Géotechnique,* 65 (3)**,** 155-168.

Buisman, A. (1936). Results of long duration settlement tests. In *Proc. 1st ICSMFE,* vol. 1, 103-107: Cambridge.

Gasparre, A., Nishimura, S., Coop, M. & Jardine, R. (2007). The influence of structure on the behaviour of London Clay. *Géotechnique,* 57 (1)**,** 19-31.

Giannopoulos, K. (2011). *Numerical Analysis of the Reuse of Piled Raft Foundations.* PhD thesis, Imperial College London.

Graham, J., Crooks, J. & Bell, A. L. (1983). Time effects on the stress-strain behaviour of natural soft clays. *Géotechnique,* 33 (3)**,** 327-340.

Karstunen, M. & Yin, Z.-Y. (2010). Modelling time-dependent behaviour of Murro test embankment. *Géotechnique,* 60 (10)**,** 735-749.

Kutter, B. & Sathialingam, N. (1992). Elastic-viscoplastic modelling of the rate-dependent behaviour of clays. *Géotechnique,* 42 (3)**,** 427-441.

Kuwano, R. & Jardine, R. J. (2002). On measuring creep behaviour in granular materials through triaxial testing. *Canadian Geotechnical Journal,* 39 (5)**,** 1061-1074.

Le, T., Airey, D. & Standing, J. (2019). Creep behaviour of undisturbed London Clay in triaxial stress space. In *E3S Web of Conferences,* vol. 92, 05006: EDP Sciences.

Lehane, B. & Jardine, R. (2003). Effects of long-term pre-loading on the performance of a footing on clay. *Géotechnique,* 53 (8)**,** 689-695.

Leroueil, S., Kabbaj, M., Tavenas, F. & Bouchard, R. (1985). Stress–strain–strain rate relation for the compressibility of sensitive natural clays. *Géotechnique,* 35 (2)**,** 159-180.

Mitchell, J. K., Campanella, R. G. & Singh, A. (1968). Soil creep as a rate process. *Journal of the Soil Mechanics and Foundations Division,* 94 (1)**,** 231-253.

Perzyna, P. (1963). The constitutive equations for rate sensitive plastic materials. *Quarterly of applied mathematics,* 20 (4)**,** 321-332.

Potts, D. M. & Zdravković, L. (1999). *Finite element analysis in geotechnical engineering Vol. 1*: Thomas Telford London.

Rezania, M., Taiebat, M. & Poletti, E. (2016). A viscoplastic SANICLAY model for natural soft soils. *Computers and Geotechnics,* 73 128-141.

Roscoe, K. H. & Burland, J. (1968). On the generalized stress-strain behaviour of wet clay.

Rowe, R. K. & Hinchberger, S. D. (1998). The significance of rate effects in modelling the Sackville test embankment. *Canadian Geotechnical Journal,* 35 (3)**,** 500-516.

Sivasithamparam, N., Karstunen, M. & Bonnier, P. (2015). Modelling creep behaviour of anisotropic soft soils. *Computers and Geotechnics,* 69 46-57.

Standing, J. (2018). Identification and implications of the London Clay Formation divisions from an engineering perspective. *Proceedings of the Geologists' Association*.

Standing, J. R., Potts, D. M., Vollum, R., Burland, J. B., Tsiampousi, A., Afshan, S., Yu, J. B., Wan, M. S. P. & Avgerinos, V. (2015). Investigating the effect of tunnelling on existing tunnels. In *Underground Design and Construction Conference,* Hong Kong: IOM3 Hong Kong Branch.

Tavenas, F., Leroueil, S., Rochelle, P. L. & Roy, M. (1978). Creep behaviour of an undisturbed lightly overconsolidated clay. *Canadian Geotechnical Journal,* 15 (3)**,** 402-423.

Taylor, D. W. & Merchant, W. (1940). A theory of clay consolidation accounting for secondary compression. *Journal of Mathematics and Physics,* 19 (1-4)**,** 167-185.

Yin, J.-H. (1999). Non-linear creep of soils in oedometer tests. *Géotechnique,* 49 (5)**,** 699-707.

Yin, J.-H. & Graham, J. (1999). Elastic viscoplastic modelling of the time-dependent stress-strain behaviour of soils. *Canadian geotechnical journal,* 36 (4)**,** 736-745.

Zdravković, L., Potts, D. & Bodas Freitas, T. (2019). Extending the life of existing infrastructure. *Proc. XVII ECSMGE-2019.* Reykjavik.