Modelling of time-dependent behaviour of soft soils using simple elasto-viscoplastic model

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Abstract: The purpose of this study was to present the development of an elasto-viscoplastic constitutive model to describe the time-dependent behaviour of soft soils. The elasto-viscoplastic model was established within the framework of Perzyna's overstress theory and the Modified Cam Clay model. The stress-strain relationship was solved by using an implicit backward Euler method of stress with updated algorithm, and implemented in a finite element program. Different types of tests were simulated using the EVP-MCC model to simulate the time-dependent behaviour of soft soils under different loading conditions, such as the strain rate effects on preconsolidation pressure as well as on undrained shear strength; the primary, secondary consolidation behaviour and stress effects on secondary compression coefficient $C_{\alpha e}$; the creep and stress relaxation features under different stress levels. It was shown that the model could satisfactorily describe the time-dependent behaviour of normally consolidated or slightly overconsolidated clayey soils along different loading paths. Time-dependent properties behaved in multiple stages triaxial tests, and field and laboratory pressuremeter tests had also been successfully simualted by the proposed EVP-MCC model.

Key words: elasto-viscoplastic constitutive model; time-dependency; soft soils; strain rate; creep; relaxation

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一个能够模拟软土时效特性的简单弹黏塑性模型

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摘 要：基于 Perzyna 超应力理论与修正剑桥模型，建立了一个能够模拟软土时效特性的简单的弹黏塑性本构模型，提出了参数的实验室确定方法。以室内试验为基础，模拟了不同试验条件下软土的时效特性：应变速率对先期固结压力和不排水抗剪强度的影响；一维固结与次固结特征及竖向应力对次固结系数的影响；不同应力水平下的不排水蠕变特征；不同应变水平下的应力松弛特征。通过实验数据与数值模拟的比较，对模型进行了验证，发现上述本构模型能够较好地描述不同加载路径下的正常固结与微超固结土的时效特征。同时，通过对同一试样的多阶段加卸载三轴实验、现场压力仪实验及实验室压力仪实验的模拟，发现此模型可以较好地拟合实验过程中复杂应力路径下软土的时效特征。

关键词：弹黏塑性模型；时效特性；软土；应变速率；蠕变；应力松弛

0 Introduction

The numerous constitutive models integrating the viscous behaviour of fine soils which have been proposed up to now can all be classified into three categories as empirical models, rheological models and general stress–strain–time models. The empirical models are mainly obtained by matching the equations with experimental results from constant strain rate, creep and stress relaxation tests[1-7]. These models are strictly

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limited to specific boundary and loading conditions. Rheological models usually describe uniaxial conditions and they are expressed in a differential form or as closed-form solutions. They are often used to obtain a conceptual understanding of time effects in soil[8-9]. According to Singh and Mitchell[3], the rheological models could be generalised from one to three dimensions, but practical calibration and application seem to be difficult to achieve. The general stress–strain–time models are three-dimensional models and they are usually expressed in incremental form. Therefore, they are suitable to numerical implementation in a finite element program for example. Furthermore, these models are not limited to given boundary conditions and can be used to simulate all possible stress paths.

In the last three decades, the majority of researchers proposed viscoplastic models based on the framework of Perzyna's overstress theory. Some of them used the distance between the static yield surface and the dynamic loading surface as their scaling function[10-15], while others introduced the secondary compression coefficient $C_{\alpha e}$ of one-dimensional tests into their scaling function [16-20]. However, the coefficient $C_{\alpha e}$ varies during the compression for different stress levels[21]. Fig. 1 shows the evolution of $C_{\alpha e}$ versus the applied stress from conventional oedometer tests performed on several natural clays: Flumet clay[12], Shanghai clay and Yuhuan clay by authors. It is found that the value of $C_{\alpha e}$ varies for a large domain of applied stress before reaching a stable value for these natural clays.

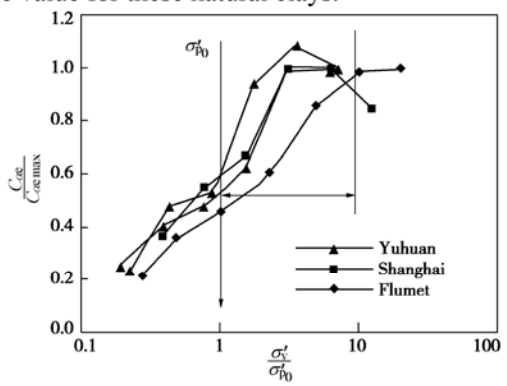


Fig. 1 Evolution of the secondary compression coefficient versus applied vertical stress from conventional oedometer tests

The purpose of this study is to develop a time-dependent constitutive model to provide a simple but realistic approach in modelling time-dependent behaviour of soils, so that it can be easily calibrated and used in geotechnical projects.

In this paper, the authors present firstly the development of an elasto-viscoplastic model with its numerical derivation for stress-strain relationship and its implementation in a finite element program CESAR_LCPC. The model parameters identification is then proposed.

The experimental verification of the proposed model for soft soils is finally undertaken, based on simulating different types of laboratory and in situ tests, along stress and strain paths such as constant strain rate loading, creep and stress relaxation.

1 Constitutive model

1.1 Constitutive model

The Modified Cam Clay model[22] has been widely used for estimating the time-independent behaviour of soft clay. Because this model is formulated very simply and suited for finite element analysis, it is adopted in the present study as a basis for formulating a viscoplastic model.

According to the assumption of Perzyna's overstress theory[23-24], the viscous effects are negligible in the elastic region. In other words, the elastic strains are time independent, whereas the inelastic strains are time dependent. We define a static yield criterion $f_{\rm s}$ which represents a reference yield surface (elastic limit with null viscoplastic flow rule) for the material. Its initial shape depends on the consolidation pressure $p_{\rm c}^{\rm s}$. The expansion of the static yield surface, which represents the hardening of the material, is expressed by the variation of the consolidation pressure due to the inelastic volumetric strain $\varepsilon_{\rm v}^{\rm vp}$.

$$\mathrm{d}p_{\rm c}^{\rm s} = p_{\rm c}^{\rm s} \cdot \frac{1+e_0}{\lambda-\kappa} \cdot \mathrm{d}\varepsilon_{\rm v}^{\rm vp} = \frac{p_{\rm c}^{\rm s}}{\beta^*} \cdot \mathrm{d}\varepsilon_{\rm v}^{\rm vp} \quad , \qquad (1)$$

where $p_{\rm c}^{\rm s}$ means the static consolidation pressure, $\varepsilon_{\rm v}^{\rm vp}$ is the inelastic volumetric strain, β^* is the compressibility index.

A dynamic loading yield criterion $f_{\rm d}$ is defined to represent the current state of stress and is expressed as:

$$f_{\rm d} = \frac{q^2}{M^2} + p' \cdot (p' - p_{\rm c}^{\rm d}) = 0 \quad , \qquad (2)$$

where $p_{\rm c}^{\rm d}$ is the dynamic consolidation pressure, M is the slope of the critical state line, p' is the effective mean stress, q is the deviatoric stress.

Based on the values of $p_{\rm c}^{\rm s}$ and $p_{\rm c}^{\rm d}$, the scaling function $\mu \cdot \phi(F)$ is taken as an exponential form shown as Equation (3). It could control the amplitude of the viscoplastic strain rate and enlarge the domain of application of this model[12].

$$\mu \cdot \phi(F) = \mu \cdot \left(\exp\left[N \cdot \left(\frac{p_{\mathrm{c}}^{\mathrm{d}}}{p_{\mathrm{c}}^{\mathrm{s}}} - 1 \right) \right] - 1 \right) \quad , \qquad (3)$$

where, μ and N are the viscosity parameters of the model. For constant strain rate test, N controls the strain rate parameter $\rho_{\alpha 0}$, and μ controls the amplitude of stress as showed in details by Yin[25]. The flow rule for the viscoplastic strain rate, in a simple case of infinitesimal strain field, follows the form proposed by Perzyna[24].

$$\dot{\varepsilon}_{ij}^{\mathrm{vp}} = \mu \langle \phi(F) \rangle \frac{\partial f_{\mathrm{d}}}{\partial \sigma_{ij}'} \quad , \qquad (4)$$

where, the function of MacCauley is

$$\langle F \rangle = \begin{cases} 0 & (F \le 0) \ , \\ F & (F > 0) \ . \end{cases}$$

The principles of the elasto-viscoplastic model (called EVP-MCC model) are illustrated by the effective stress path of an undrained triaxial test in $p' - q$ space (see Fig. 2). The stress state "*A*" represents an initially normally K_0 consolidation state. Along the loading stress path "*A-B-C*" for constant strain rate test, viscoplastic volumetric strains occur during loading and cause the static yield surface to expand in the stress space. As point *C* approaches *C*', corresponding to the critical state, the soil is subjected to a constant amount of overstress which provokes an increase of the deviatoric strain at constant strain rate, without any volumetric strain.

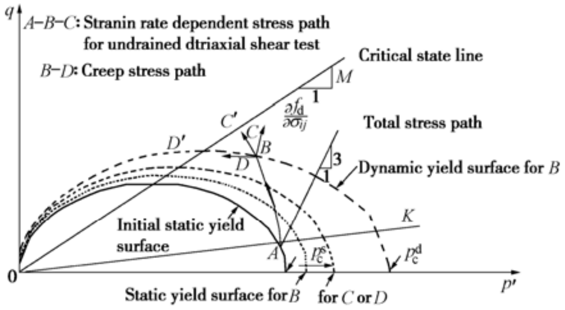


Fig. 2 Schematic behaviour of the elasto-viscoplastic Modified Cam Clay model during CAU triaxial compression and creep tests

As for the creep stress path "*B-D*", the static yield surface expands with the viscoplastic volumetric strain, function of the amount of overstress. If the static yield surface can reach the actual stress point, the equilibrium is obtained and the strain will be stabilized with time. If not, the effective stress will continue to evolve until it reaches the critical state at point *D*' where it will stop because no viscoplastic volumetric strain will develop, but deviatoric strain will continue to increase.

Taking into account the elastic stress-strain relations, the constitutive equations of the viscoplastic model for normally consolidated clays are derived as follows:

$$\dot{\varepsilon}_{ij} = \frac{\dot{s}_{ij}'}{2G} + \frac{\dot{p}'}{3K}\delta_{ij} + \mu \langle \phi(F) \rangle \left(\frac{3s_{ij}'}{M^2} + \left(2p' - p_{\mathrm{c}}^{\mathrm{d}} \right) \frac{\delta_{ij}}{3} \right) \quad , (5)$$

where σ_{ij}' is the effective stress tensor, $\dot{\varepsilon}_{ij}^{\mathrm{vp}}$ is the inelastic strain rate tensor, s_{ij}' is the deviatoric stress tensor.

An implicit backward Euler method of stress update algorithm is used to integrate the viscoplastic stress-strain equations to a FEM code of CESAR-LCPC.

1.2 Methodology of parameter identification

The proposed model involves the parameters of Cam Clay model $\{E, \nu, \beta^*, M, p_{\mathrm{c0}}\}$, and two additional parameters of viscosity $\{N, \mu\}$.

The details about the sensitivity of the parameters of Cam Clay model will not be presented here since studies on them are well investigated and documented[26]. We can point out as:

(1) E, M, p_{c0}' have a significant effect on the stress-strain relationship;

(2) ν has a very small effect on the stress-strain relationship;

(3) β^* has a very small effect on the stress-strain relationship for triaxial and pressuremeter tests but a significant effect for oedometer tests;

(4) only E can change the initial slope of the strain-stress curve.

Yin[26] found that the viscoplastic parameters of N and μ have an important effect on the computed stress, especially for $5 < N < 20$ and $\mu < 1 \times 10^{-6}$. The parameters will be conducted with at least three different strain rates.

Based on the parameters study[25], a general procedure to determine the soil parameters from laboratory tests is established and employed as follows:

(1) ν was taken equal to 0.3, which is a common value for clay.

(2) E could be determined by the initial slope of the strain-stress curves.

(3) β^* could be determined from oedometer tests performed on the studied soil: $\beta^* = (\lambda - \kappa)/(1 + e_0)$.

(4) The slope M of the critical state line could be

Table 1 Values of EVP model and hydraulic parameters

Sites	w /%	I_P /%	E /kPa	ν	β^*	M	p'_{c0} /kPa	N	μ ($s^{-1}\cdot kPa$)	k ($m\cdot s^{-1}$)
Batiscan	80	21	5000	0.30	0.76	1.00	70	10	1×10^{-9}	1.0×10^{-8}
Burswood	—	83	4000	0.30	0.07	1.45	38	12	2×10^{-7}	3.3×10^{-9}
Flumet	—	—	4000	0.30	0.07	1.50	70	10	1×10^{-8}	1.0×10^{-9}
Fukakusa	—	22	25000	0.30	0.05	1.00	350	12	1×10^{-10}	—
Haney	—	18	35000	0.25	0.09	0.85	515	10	1×10^{-7}	—
Hongkong	52	32	20000	0.30	0.06	1.10	270	12	2×10^{-8}	—
Osaka	—	—	35000	0.30	0.14	1.20	500	12	5×10^{-9}	—
St-Herblain	121	42	2000	0.30	0.13	1.25	30	10	1×10^{-9}	1.0×10^{-10}

determined from triaxial tests.

(5) p'_{c0} could be determined by the fitting of several curves, knowing that its value was located between the initial consolidation pressure and the preconsolidation pressure measured by oedometer tests. However, its determination from oedometer tests is not straightforward, since its value depends on the loading rate, as it will be shown later.

(6) N and μ could be determined from tests with more than two different strain rates, or tests with a relaxation or a creep stage.

(7) k could be determined by fitting the pore pressure dissipation curves for tests with drained or partly drained conditions.

2 Modelling the time-dependent behavior of soft soils along different loading paths

2.1 General description

To analyze the model capability of reproducing the time-dependent behaviour of soils, different types of tests found in the literatures are simulated, and the results were compared with the experimental data. For each type of test, the authors summarized the water content and the plastic index of the studied soil in Table 1. The permeability value was taken into account for tests with drained or partially drained conditions.

2.2 Effect of strain rate on preconsolidation stress

Leroueil et al[5] presented constant strain rate oedometer tests on Batiscan clay with strain rate varying from test to test between 1.7×10^{-8} s^{-1} and 4×10^{-5} s^{-1}. The specimens were 19.0 mm high and 50.8 mm in diameter. The drainage was allowed only at the top of the specimens. The initial state was taken at σ'_{v0} = 65 kPa, a stress equal to the in situ vertical effective stress at the corresponding depth of the specimens. The values of the parameters determined from constant strain rate and creep tests are given in Table 1.

The predictions and experimental results, shown in Fig. 3, are in good agreement for settlements less than 17%. In particular, the EVP-MCC model can take into account the effect of strain rate on the preconsolidation pressure, which is more accurate than that described by the linear relationship (Eq.(6)) between the preconsolidation pressure and the vertical strain rate assuming a constant ratio of $C_{\alpha e}/C_c$ proposed by Leroueil et al[27], i.e., the EVP-MCC model improves the description of the effect of strain rate on the preconsolidation pressure compared with these models based on the formulation Eq.

$$\frac{\left(\sigma'_p\right)_{v1}}{\left(\sigma'_p\right)_{v0}}=\left(\frac{\dot{\varepsilon}_{v1}}{\dot{\varepsilon}_{v0}}\right)^{C_{\alpha e}/C_c} , \tag{6}$$

where the index v_0 , v_1 represent the two values corresponding to two different strain rates; $C_{\alpha e}$ is the coefficient of secondary compression; C_c is the compression index. However, for the settlements more than 17%, there is a large difference between the measured and predicted results, which is perhaps due to the particles debonding of clays under large applied stress that is not taken into account in EVP-MCC model, and the small strain analysis underestimated the stress for this case.

2.3 Primary and secondary consolidation

One-dimensional creep tests were also carried out on Batiscan clay from the same depth by Leroueil et al[5] with the same device and under the same initial conditions. The applied vertical stress varied from 78 to 139 kPa. The material parameters are given in Table 1.

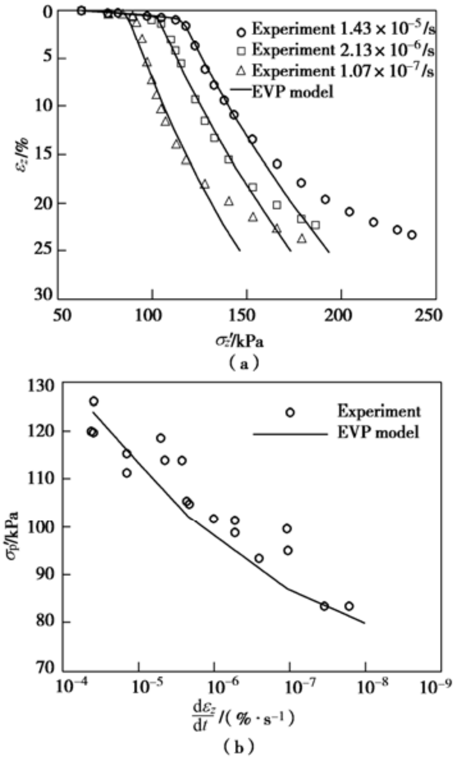


Fig. 3 Comparison between predicted and experimental results for one-dimensional constant strain rate tests on Batiscan clay

A good agreement was achieved between the experimental results and simulations, as presented in Fig. 4. The values of $C_{\alpha e}$ obtained by the EVP-MCC model are slightly different from the experimental results.

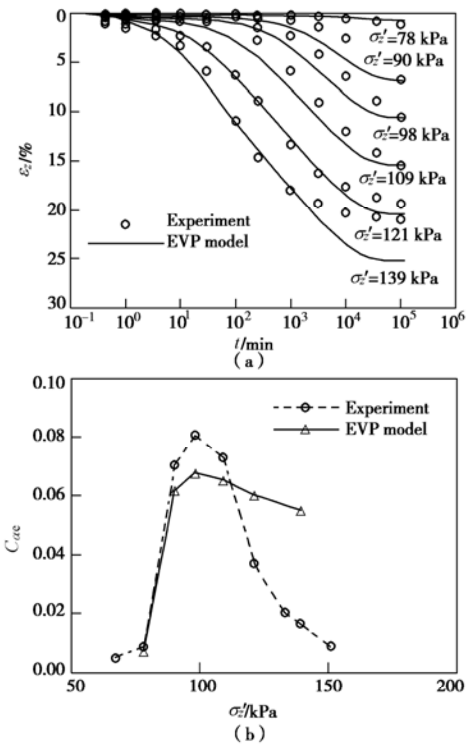


Fig. 4 Comparison between predicted and experimental results for one-dimensional loading tests on Batiscan clay

The strong decrease of $C_{\alpha e}$ in experimental results is not generally observed and can be explained by a considerable reduction in the compression index C_c, as observed in Fig. 3, which is usually observed for sensitive clays. In conclusion, the EVP-MCC model can predict the primary and secondary consolidation. In addition, the proposed model with two viscosity parameters can well describe the effect of stress on $C_{\alpha e}$, but not for the viscoplastic models based on a single viscosity parameter $C_{\alpha e}$.

2.4 Effect of strain rate on undrained shear strength in triaxial tests

Adachi et al[10] conducted undrained constant strain rate triaxial tests on alluvial Osaka clay. The samples were initially consolidated under the isotropic stress of 580 kPa. The triaxial tests were carried out under constant strain rate varying from 0.00078%/min to 1.0%/min. The parameters determined from compression tests are summarized in Table 1. The comparison between experimental and numerical results, presented in Fig. 5, shows a good agreement in the evolution of the deviatoric stress as well as the effective stress path.

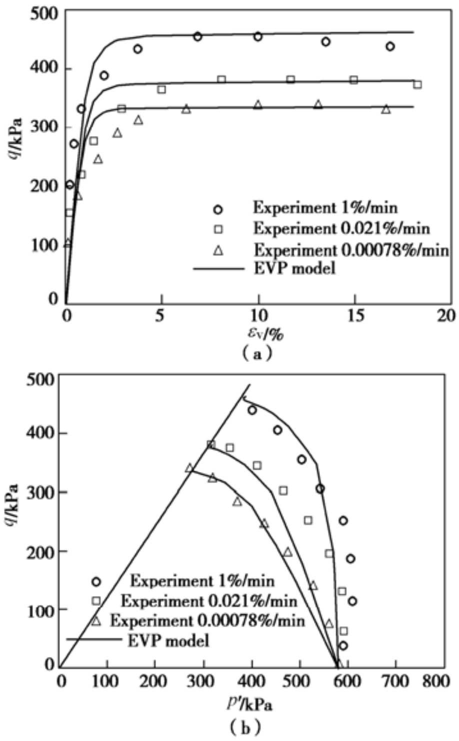


Fig. 5 Comparison between predicted and experimental results for strain rate triaxial tests on Osaka clay

Constant strain rate tests on remoulded Haney clay under strain rates varying from 0.0001%/min to 10%/min performed by Vaid and Companella[28] were also simulated by using the parameters summarized in Table

1. The results of the normalized maximal deviatoric stress $q^*_{\max}$ (the maximal deviatoric stress for each rate divided by the one corresponding to the greatest strain rate) as a function of the strain rate, presented in Fig. 6, show the effect of the strain rate on the undrained shear strength in two phases: for the tests with small strain rates, the shear strength converges to a constant stress value representing the long-term strength of material; while for tests with middle-level strain rates, the strain rate effect can be represented by the relationship proposed by Sheahan et al[29]:

$$\frac{q_{\mathrm{f,a}}}{q_{\mathrm{f,a0}}}=1+\rho_{\mathrm{a0}}\cdot\lg\left(\frac{\dot{\varepsilon}_{\mathrm{a}}}{\dot{\varepsilon}_{\mathrm{a0}}}\right)\quad, \tag{7}$$

where the indexes $\dot{\varepsilon}_{\mathrm{a}}$ and $\dot{\varepsilon}_{\mathrm{a0}}$ represent two different strain rates, q_{f} is the undrained shear strength and ρ_{a0} is a strain rate parameter.

As shown in Fig. 5 and Fig. 6, the EVP-MCC model can take into account the strain rate effect on the undrained shear strength in a large range of strain rate, as well as the decrease of this effect for small values of the strain rate (Haney clay samples).

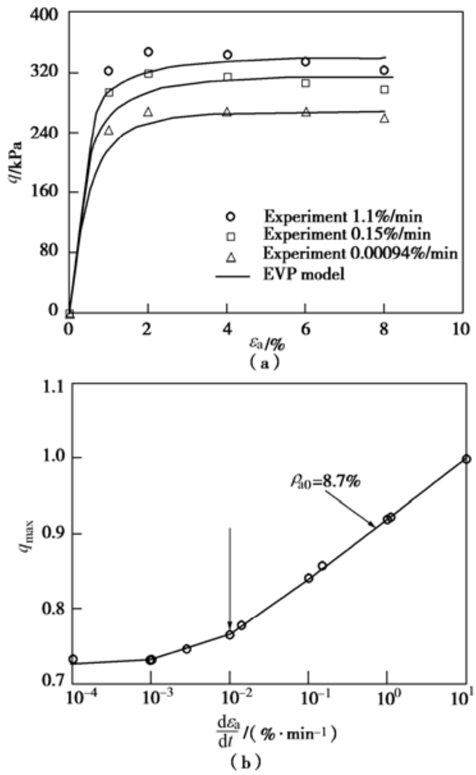


Fig. 6 Comparison between predicted and experimental results for constant strain rate triaxial tests with strain rate parameters on Haney clay

2.5 Creep at different stress levels

Undrained creep triaxial tests on remoulded Fukakusa clay have been performed by Adachi and Oka[10] after the initial consolidation stage under a isotropic stress of 392 kPa. A deviatoric stress varying from 78 to 235 kPa was applied for each sample. The material parameters determined by the previous study are given in Table 1.

Fig. 7 shows that the predictions are close to the measured values. For low applied stress the strain increases slowly and seems to converge toward a stabilized value, as for a higher applied stress the strain rate is higher and the strain keeps increasing with time. The increase in pore pressure versus time predicted by the model agrees generally with the measured one. The EVP-MCC model can model the time-dependent behaviour of soil during undrained creep.

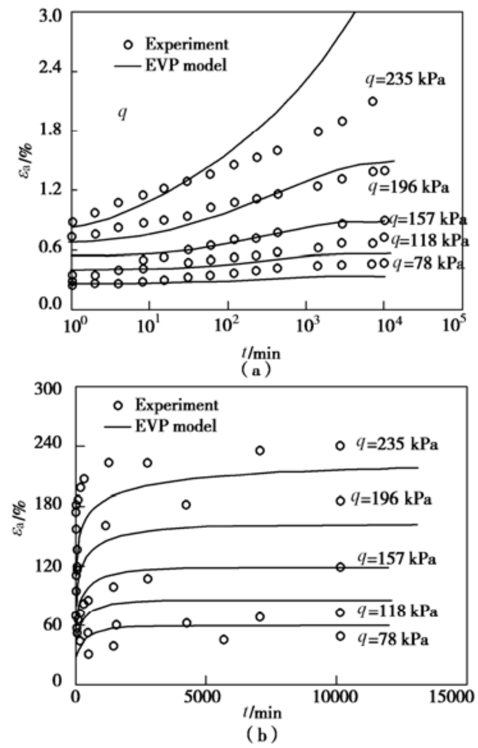


Fig. 7 Comparison between predicted and experimental results for creep triaxial tests on Fukakusa clay

2.6 Stress relaxation at different strain amplitudes

Results of stress-relaxation tests on Flumet clay in a triaxial apparatus are provided by Fodil et al[12]. An initial loading with an axial strain rate of 0.15%/h was applied. The stress-relaxation tests were then performed on the same sample for each test at about 1%, 3.5% and 6.7% of axial strain and lasted at least 24 h. The material parameters are summarized in Table 1. The viscoplastic model predicts reasonably well the results of the stress-relaxation tests in terms of deviatoric stress decrease versus time, as shown in Fig. 8.

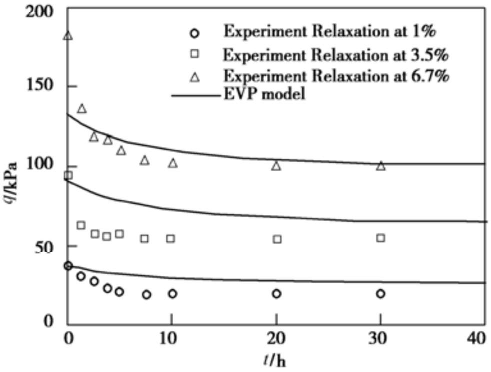


Fig. 8 Comparison between predicted and experimental results for relaxation tests on Flumet clay

2.7 Time-dependent behaviour during multiple stages triaxial tests

Undrained triaxial shear tests combined with stress relaxation were conducted on remoulded Hong Kong marine clay by Yin et al[19]. The samples were isotropically consolidated under a stress of 300 kPa for 36 hours. The parameters given by Yin et al[19]. and determined from simulations are summarized in Table 1.

A comparison between the experimental and the calculated results in terms of deviatoric stress versus axial strain, excess pore pressure versus axial strain, deviatoric stress versus strain rate and effective stress paths is shown in Fig. 9. A general agreement can be observed although the relaxation stages are not perfectly reproduced. The predictions are acceptable for such a complicated loading history.

2.8 Strain-rate-dependent behaviour during pressuremeter tests

Self boring pressuremeter (SBP) tests were performed in the Burswood Peninsula site at a depth of 5.25 m by Lee and Fahey[30]. The expansion phase was conducted at two different rates to around 10% cavity strain: 0.167%/s and 0.0185%/s. The parameters from oedometer and triaxial tests by Lee and Fahey with viscous parameters determined by curve fitting are summarized in Table 1.

The pressuremeter tests in laboratory, using a modified triaxial apparatus called pressio-triax, were performed on the Saint-Herblain clay at depth of 4.5～5.5 m by Rangeard[31] at a strain rate of 6×10^{-6} s^{-1} until a strain of 4.2%, followed by an unloading and reloading phase. The parameters determined by curve fitting are summarized in Table 1.

Fig. 10 gives a comparison between the predicted and measured results for SBP tests and pressio-triax tests in terms of total pressure at the cavity wall versus cavity strain. It shows that the EVP-MCC model can also simulate satisfactorily the time-dependent behavior of soils during pressuremeter tests.

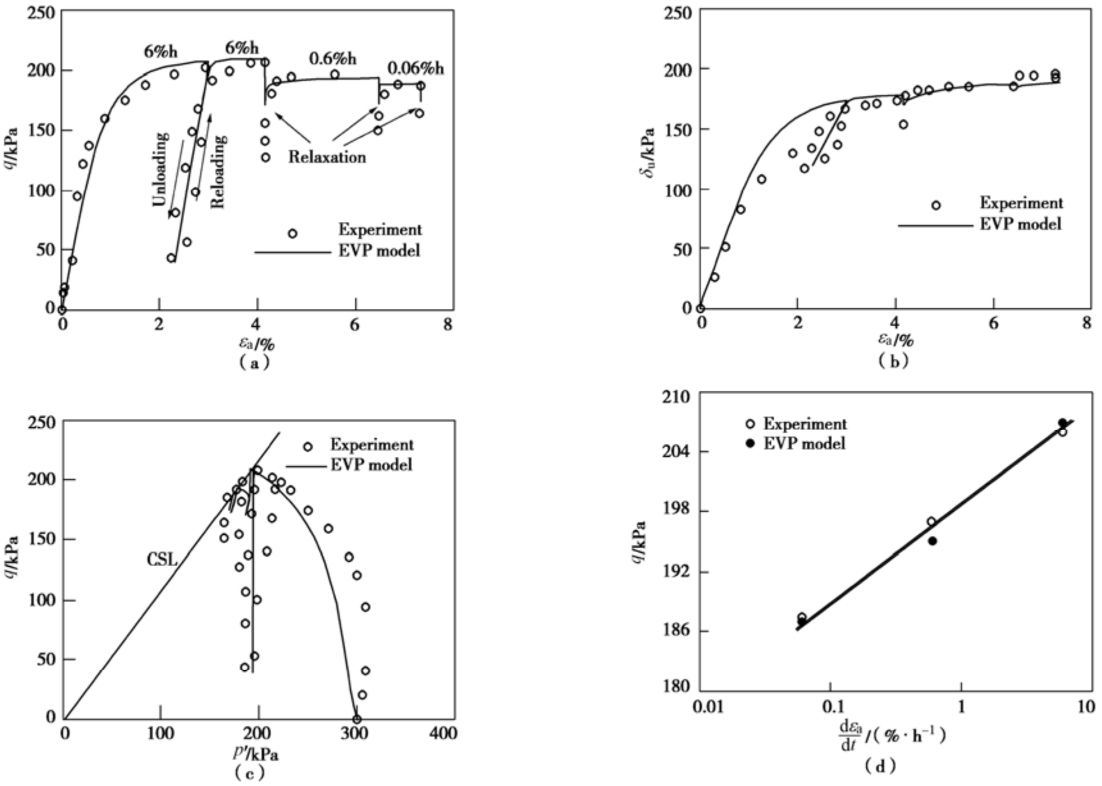
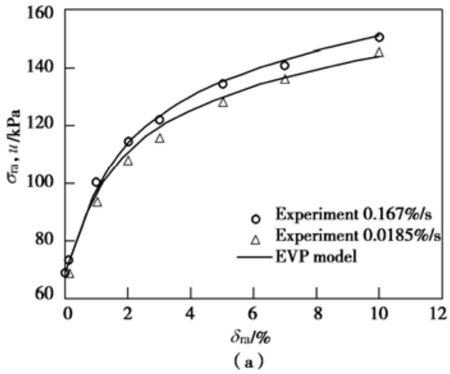


Fig. 9 Comparison between predicted and experimental results on remoulded Hong Kong marine clay

Fig. 10 shows comparison between predicted and experimental results for SBP tests on Burswood clay and for pressio-triax tests on Saint-Herblain clay.



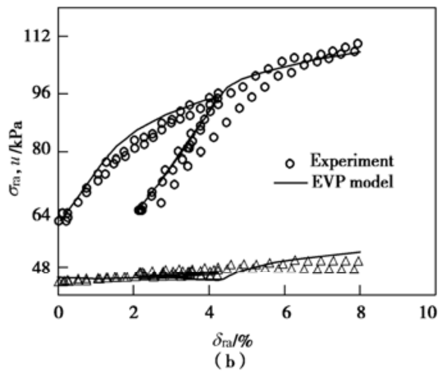


Fig. 10 Comparison between predicted and experimental results for SBP tests on Burswood clay and for pressio-triax tests on Saint-Herblain clay

3 Discussion

All these examples demonstrate that the main features of the stress-strain-time relationship along different loading paths could be reproduced by the EVP-MCC model for normally or slightly overconsolidated clayey soils.

The model could be taken as a basis to be further extended, for example, to incorporate soil particle debonding effect for destructuration feature of structured soil, to modify the plastic yield criterion for anisotropy feature of natural clays, to introduce frequency effect on hardening law and elastic modulus for the soil behaviour under cyclic loading, etc.

4 Conclusions

Different types of tests were simulated by using the EVP-MCC model, which is based on the Modified Cam Clay elastoplastic model and on Perzyna's overstress theory. The comparison between the experimental data and numerical results shows the ability of the model to reproduce different aspects of the time-dependent behaviour of soil under different monotonic loading conditions, such as, the strain rate effects on preconsolidation pressure; the primary, secondary consolidation behaviour and stress effects on $C_{\alpha e}$; the strain rate effects on undrained shear strength; the creep feature under different the stress levels; the stress relaxation feature under different strain levels; the time-dependent behaviour during multiple stages triaxial tests with complex stress path.

The time-dependent behaviour during pressuremeter tests in the field and in the laboratory has also been successfully reproduced by the proposed model.

Further validation for the predictive capability of this model is needed, such as the prediction of the behaviour of embankments or infrastructures built on soft clay.

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