A structured Cam Clay model

M.D. Liu and J.P. Carter

Abstract: A theoretical study of the behaviour of structured soil is presented. A new model, referred to as the Structured Cam Clay model, is formulated by introducing the influence of soil structure into the Modified Cam Clay model. The proposed model is hierarchical, i.e., it is identical to the Modified Cam Clay soil model if a soil has no structure or if its structure is removed by loading. Three new parameters describing the effects of soil structure are introduced, and the results of a parametric study are also presented. The proposed model has been used to predict the behaviour of structured soils in both compression and shearing tests. By making comparisons of predictions with experimental data and by conducting the parametric study it is demonstrated that the new model provides satisfactory qualitative and quantitative modelling of many important features of the behaviour of structured soils.

Key words: calcareous soils, clays, fabric, structure, constitutive relations, plasticity.

Résumé : On présente une étude théorique sur le comportement du sol structuré. On formule un nouveau modèle auquel on réfère comme le modèle Cam Clay structuré en introduisant l'influence de la structure du sol dans le modèle Cam Clay modifié. Le modèle proposé est hiérarchique, i.e., il est identique au modèle de sol Cam Clay modifié si un sol n'a pas de structure ou si sa structure est éliminé par chargement. Trois nouveaux paramètres décrivant les effets de la structure du sol sont introduits et les résultats d'une étude paramétrique sont également présentés. Le modèle proposé a été utilisé pour prédire le comportement des sols structurés dans les essais de compression et de cisaillement. En faisant des comparaisons de prédictions avec des données expérimentales et en conduisant un étude paramétrique, il est démontré que le nouveau modèle fournit une modélisation qualitative et quantitative de plusieurs caractéristiques importantes du comportement des sols structurés.

Mots clés : sols calcareux, argiles, fabrique, structure, relations de comportement, plasticité.

[Traduit par la Rédaction]

Introduction

Soils in situ usually possess natural structure, which enables them to behave differently from the same material in a reconstituted state (e.g., Burland 1990; Leroueil and Vaughan 1990; Cuccovillo and Coop 1999). Recently, there have been important developments in formulating constitutive models incorporating the influence of soil structure, such as those proposed by Gens and Nova (1993), Whittle (1993), Wheeler (1997), Rouainia and Muir Wood (2000), and Kavvadas and Amorosi (2000). In this paper a new constitutive model for structured clays is proposed. The main objective of this new formulation is to provide a constitutive model suitable for the solution of boundary value problems encountered in geotechnical engineering practice. It is intended therefore, that the new model should be relatively simple and should have few parameters, each of which has a clear physical meaning and can be conveniently identified. The model should also be relatively easy to understand and apply.

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M.D. Liu[1] and J.P. Carter. Department of Civil Engineering, The University of Sydney, NSW 2006, Australia.

[1]Corresponding author (e-mail: m.liu@civil.usyd.edu.au).

The Modified Cam Clay model (Roscoe and Burland 1968) is widely referenced and has been widely used in solving boundary value problems in geotechnical engineering practice (e.g., Gens and Potts 1988; Yu 1998; Potts and Zdravkovic 1999), although it was developed originally for reconstituted clays. Because of some familiarity with the model by the geotechnical profession and because it captures well the essential behaviour of reconstituted soil, the Modified Cam Clay model was chosen as the basis for the current research. A new model has been formulated by introducing the influence of soil structure into the Modified Cam Clay model. A parametric study demonstrating the capabilities and limitations of this new model is presented. The model has also been used to predict the behaviour of a variety of structured natural soils in both compression and shearing tests, allowing for a reasonably comprehensive evaluation. It is demonstrated that the proposed new model is suitable for describing the behaviour of a variety of natural clays and cemented soils.

Generalization of Modified Cam Clay

The Modified Cam Clay model was proposed by Roscoe and Burland (1968) and a description and systematic study of the model can be found in the text by Muir Wood (1990). Formulations of this model suitable for use in finite element analysis can also be found in various texts (e.g., Britto and Gunn 1987; Potts and Zdravkovic 1999). In this paper, the Modified Cam Clay model is employed as a basis for

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formulating a hierarchical model for structured clays, which is referred to as the "Structured Cam Clay" model.

It is assumed that the behaviour of soil in a reconstituted state can be described adequately by the Modified Cam Clay model. The term "soil structure" is used here to mean the arrangement and bonding of the soil constituents, and for simplicity it encompasses all features of a soil that are different from those of the corresponding reconstituted soil. Following the suggestion of Burland (1990), the properties of a reconstituted soil are called the intrinsic properties, and are denoted by the symbol * attached to the relevant mathematical symbols. Hence, under all stress conditions, the influence of soil structure can be measured by comparing its behaviour with the intrinsic behaviour.

The formation and development of soil structure often produces anisotropy in the mechanical response of soil to changes in stress. Destructuring usually leads to the reduction of anisotropy. To concentrate on introducing the physical concepts of the framework and to avoid unnecessary complexity of mathematical detail, only the isotropic effects of soil structure are included in the proposed theoretical framework. The extension required to include the influence of soil anisotropy shall be a future research topic, however, it is noted that others have previously considered this feature of soil behaviour, e.g., Dafalias (1987), Whittle and Kavvadas (1994), Wheeler (1997), and Rouainia and Muir Wood (2000). It should also be noted that coaxiality between the principal axes of the plastic strain increment and those of stress is assumed in the proposed framework.

The stress and strain quantities used in the present formulation are defined as follows. Variables σ'_{ij} and ε_{ij} are the Cartesian components of effective stress and of strain, respectively. The simplified forms for stress and strain conditions in conventional triaxial tests are also listed, where σ'_1 (or ε_1) and σ'_3 (or ε_3) are the axial effective stress (strain), and the radial effective stress (strain), respectively.

The mean effective stress p', deviatoric stress q, and stress ratio η are given by

$$[1] \qquad p' = \frac{\sigma'_{11} + \sigma'_{22} + \sigma'_{33}}{3} \quad = \frac{\sigma'_1 + 2\sigma'_3}{3} \qquad \text{for conventional triaxial tests}$$

$$[2] \qquad q = \frac{\sqrt{(\sigma'_{11} - \sigma'_{22})^2 + (\sigma'_{22} - \sigma'_{33})^2 + (\sigma'_{33} - \sigma'_{11})^2 + 6(\sigma'^{2}_{12} + \sigma'^{2}_{23} + \sigma'^{2}_{31})}}{\sqrt{2}}$$

$$= (\sigma'_1 - \sigma'_3) \qquad \text{for conventional triaxial tests}$$

$$[3] \qquad \eta = \frac{q}{p'}$$

The corresponding (work-conjugate) volumetric strain increment, $d\varepsilon_v$, and deviatoric strain increment, $d\varepsilon_d$, are defined by

$$[4] \qquad d\varepsilon_v = d\varepsilon_{11} + d\varepsilon_{22} + d\varepsilon_{33} = d\varepsilon_1 + 2d\varepsilon_3 \qquad \text{for conventional triaxial tests}$$

and

$$[5] \qquad d\varepsilon_d = \frac{\sqrt{2}\sqrt{(d\varepsilon_{11} - d\varepsilon_{22})^2 + (d\varepsilon_{22} - d\varepsilon_{33})^2 + (d\varepsilon_{33} - d\varepsilon_{11})^2 + 6(d\varepsilon_{12}^{\,2} + d\varepsilon_{23}^{\,2} + d\varepsilon_{31}^{\,2})}}{3}$$

$$= \frac{2(d\varepsilon_1 - d\varepsilon_3)}{3} \qquad \text{for conventional triaxial tests}$$

Influence of soil structure on virgin isotropic compression

The work by Liu and Carter (1999, 2000) is employed here as a starting point for including the effects of soil structure in the model. The material idealization of the isotropic compression behaviour of structured clay is illustrated in Fig. 1. In this figure e represents the voids ratio for a structured clay, e^* is the voids ratio for the corresponding reconstituted soil at the same stress state during virgin yielding, $p'_{y,i}$ is the mean effective stress at which virgin yielding of the structured soil begins, and Δe, the additional voids ratio, is the difference in voids ratio between a structured soil and the corresponding reconstituted soil at the same stress state. Hence, the virgin isotropic compression behaviour of a structured soil can be expressed as

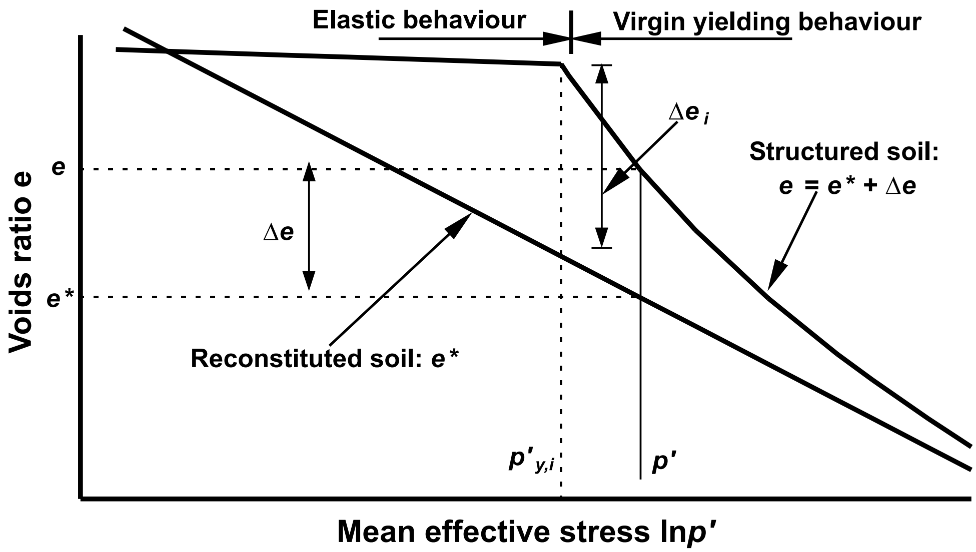
$$[6] \qquad e = e^* + \Delta e$$

The following equation was proposed by Liu and Carter (2000) to describe the volumetric behaviour of natural clays during virgin isotropic compression:

$$[7] \qquad e = e^* + \Delta e_i \left(\frac{p'_{y,i}}{p'} \right)^b$$

where Δe_i is the additional voids ratio at $p' = p'_{y,i}$, where virgin yielding of the structured soil begins (Fig. 1), and b is a parameter quantifying the rate of destructuring and is referred to here as the *destructuring index*. The value of b depends on soil type and structure and generally $b \geq 1$ for soft structured clays and $b < 1$ for stiff clays. For the 30 different

Fig. 1. Idealization of the isotropic compression behaviour of reconstituted and structured soils.



clays studied by Liu and Carter (1999, 2000), it was found that generally $0 \le b \le 30$. For clay samples of a given mineralogy and with similar geological stress history but different depths below the surface, it is found that b depends mainly on the liquidity index.

Yield surface for structured clay

In the proposed Structured Cam Clay model the behaviour of clay is divided into virgin yielding behaviour and elastic behaviour by its current yield surface, which is dependent on soil structure as well as stress history, i.e., the *structural yield surface* is defined by the current stress state, voids ratio, stress history, and soil structure. Similar to the original proposal by Roscoe and Burland (1968), the yield surface of a structured soil in $p' - q$ space is assumed to be elliptical and to pass through the origin of the stress coordinates (Fig. 2). The aspect ratio for the structural yield surface is M^*, the critical state stress ratio of the reconstituted soil. One axis of the ellipse coincides with the p' axis. The value of the p' coordinate where the ellipse again intersects the axis, p'_s, represents the size of the structural yield surface. The yield surface is thus given by the yield function f, where

$$[8] \qquad f = \left(\frac{q}{0.5M^*\, p'_s}\right)^2 + \left(\frac{p' - 0.5p'_s}{0.5p'_s}\right)^2 - 1 = 0$$

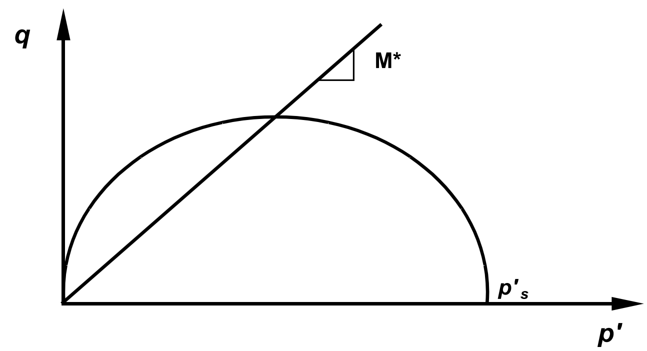
Volumetric deformation for virgin yielding along general stress paths

As illustrated in Fig. 1, virgin yielding and the elastic behaviour of reconstituted clay, i.e., a clay where structure is completely removed, are both linear in $e - \ln p'$ space, with gradients λ^* and κ^*, respectively. The isotropic virgin compression line for the reconstituted soil, i.e., ICL*, is given by

$$[9] \qquad e^* = e^*_{IC} - \lambda^* \ln p'$$

where e^*_{IC} is the voids ratio of the reconstituted soil when p' = 1 kPa during virgin isotropic compression. We seek now to generalize eq. [9] for a soil that possesses structure.

Fig. 2. The yield surface for structured soils.



Consider loading where the current stress state stays on the yield surface, and the size of the current yield surface is denoted as p'_s. For monotonic loading, virgin yielding occurs if $p'_s \ge p'_{y,i}$. Under the assumption that the hardening of structured soil is dependent on the plastic volumetric deformation, the yield surface for a structured soil is defined by all stress states that have the same accumulation of absolute plastic volumetric strain. In such a model, the plastic volumetric deformation is dependent on the change in size of the yield surface only. If the elastic deformation for a structured soil is assumed to be the same as that of the reconstituted soil, any change in the additional voids ratio due to soil structure must also be associated with plastic volumetric deformation, and therefore is also dependent on the size of the yield surface. Consequently, the variable p' in eq. [7] may be written in terms of the size of the current yield surface p'_s. On substituting eq. [9] into eq. [7], the following expression for the variation of the voids ratio is obtained:

$$[10] \qquad e = e^*_{IC} + \Delta e_i \left(\frac{p'_{y,i}}{p'_s}\right)^b - \lambda^* \ln p' \qquad \text{for } p'_s \ge p'_{y,i}$$

where $p'_{y,i}$ is the value of the mean effective stress at the initial yield point for an isotropic stress state and is numerically equal to the size of the initial yield surface associated with the initial soil structure.

According to Schofield and Wroth (1968), during compression along a general stress path the volumetric deformation of a reconstituted soil is defined by $\lambda^* \ln p'$, which can be divided into two parts. The elastic part is defined by $\kappa^* \ln p'$, which is dependent on the current mean effective stress, and the plastic part is given by $(\lambda^* - \kappa^*) \ln p_s'$, which is dependent on the size of the yield surface. The voids ratio for a structured soil during virgin compression along a general stress path may therefore be expressed as

$$[11] \qquad e = e^*_{IC} + \Delta e_i \left(\frac{p'_{y,i}}{p'_s}\right)^b - (\lambda^* - \kappa^*) \ln p'_s - \kappa^* \ln p'$$

The general compression eq. [11] states that voids ratio for a structured soil during virgin compression is composed of two parts; an elastic part which is dependent on the current mean effective stress and a plastic part which is dependent on the size of the current yield surface. The plastic part is further subdivided into two parts; the part associated with the intrinsic properties of the soil and that associated with the soil structure.

Differentiating eq. [11] and noting eqs. [6] and [7], the total volumetric strain increment for compression along a general stress path is obtained as follows

$$[12] \qquad d\varepsilon_v = (\lambda^* - \kappa^*) \frac{dp'_s}{(1+e)p'_s} + b\Delta e \frac{dp'_s}{(1+e)p'_s} + \kappa^* \frac{dp'}{(1+e)p'}$$

The last part of the expression for the total volumetric strain increment, i.e., the last term on the right hand side of eq. [12], is associated with elastic deformation. Hence

$$[13] \qquad d\varepsilon^e_v = \kappa^* \frac{dp'}{(1+e)p'}$$

and furthermore

$$[14] \qquad d\varepsilon^p_v = (\lambda^* - \kappa^*) \frac{dp'_s}{(1+e)p'_s} + b\Delta e \frac{dp'_s}{(1+e)p'_s}$$

Considering the mechanism of shearing, it is rational to assume that destructuring and the associated plastic volumetric deformation should be dependent on both the change in size of the yield surface and the magnitude of the current shear stress. Therefore, a modification of eq. [14] is made so that the effect of shear stress on destructuring is also considered, i.e.,

$$[15] \qquad d\varepsilon^p_v = (\lambda^* - \kappa^*) \frac{dp_s'}{(1+e)p_s'} + b\Delta e \left(1 + \frac{\eta}{M^* - \eta}\right) \frac{dp_s'}{(1+e)p_s'}$$

where η is the shear stress ratio defined in eq. [3]. It may be seen from eq. [15] that the effect of destructuring, which is described as the reduction of the additional voids ratio, increases with the value of the current stress ratio. For a reconstituted soil, $\Delta e \equiv 0$, and the volumetric behaviour predicted by the Modified Cam Clay model is recovered.

Equation [15] can be rewritten as

$$[16] \qquad d\varepsilon^p_v = (\lambda^* - \kappa^*) \frac{dp'_s}{(1+e)p'_s} + b\Delta e \left(\frac{M^*}{M^* - \eta}\right) \frac{dp'_s}{(1+e)p'_s}$$

Because of the modification made in proceeding from eq. [14] to eq. [15], the new hardening rule for a structured soil is no longer dependent only upon the plastic volumetric deformation. It also depends on the shear stress ratio, η.

In summary, structured clay is idealized here as an isotropic hardening and destructuring material with elastic and virgin yielding behaviour. It is assumed that during virgin yielding the yield surface includes the current stress state and expands isotropically causing destructuring of the material.

Flow rule

In the Modified Cam Clay model associated plastic flow is assumed. Thus the yield surface is also the plastic potential and the flow rule is given as

$$[17] \qquad \frac{d\varepsilon^p_d}{d\varepsilon^p_v} = \frac{2\eta}{M^{*2} - \eta^2}$$

The structure of soil also has an influence on the flow rule. It is observed that a structured clay with positive Δe generally has a lower value for the strain increment ratio $d\varepsilon^p_d/d\varepsilon^p_v$ than the corresponding reconstituted soil at the same virgin yielding stress state (e.g., Olson 1962; Graham and Li 1985; Cotecchia and Chandler 1997). The following equation is therefore proposed as a flow rule for structured clay:

$$[18] \qquad \frac{d\varepsilon^p_d}{d\varepsilon^p_v} = \frac{2(1 - \omega\Delta e)\eta}{M^{*2} - \eta^2}$$

where ω is a new model parameter that describes the influence of soil structure on the flow rule. The modifier should not be negative; otherwise the plastic strain increment vector will always be directed inside the yield surface. Hence, based on the need to meet this condition at all times, including the start of virgin yielding, the following constraint is imposed

$$[19] \qquad 0 < 1 - \omega\Delta e_i \leq 1$$

and therefore

$$[20] \qquad 0 \leq \omega \leq \frac{1}{\Delta e_i}$$

Equation [18] implies a nonassociated plastic flow rule for the new model. This feature has important consequences for numerical solution schemes employing the model to solve boundary value problems. In particular, it generally results in the governing equations being nonsymmetric.

Stress–strain relationships

Elastic deformation

For stress excursions within the current yield surface, only elastic deformation occurs. The elastic deformation of a structured soil is assumed to be independent of soil struc-

Fig. 3. Influence of parameter b on isotropic compression behaviour.

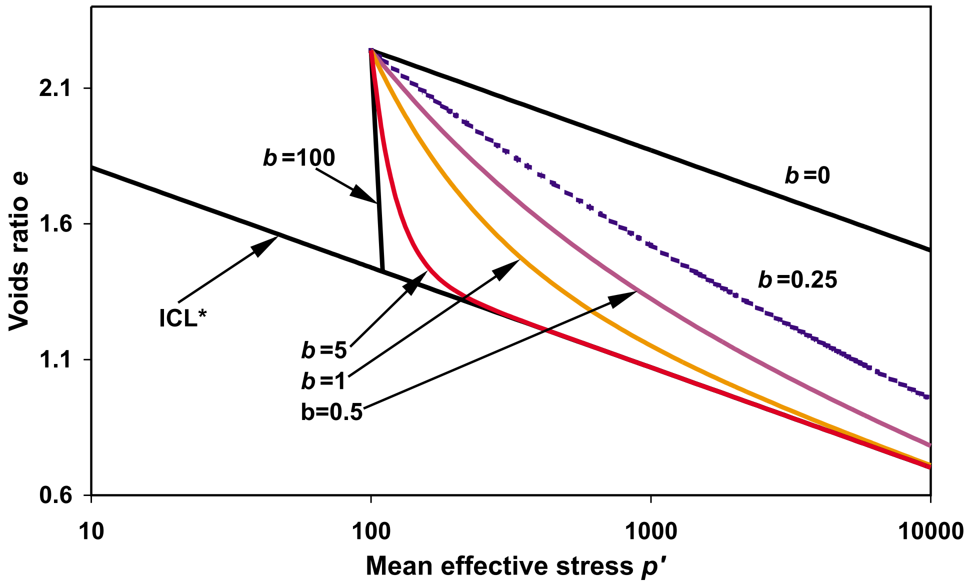


Table 1. Model parameters.

Parameter	M^*	λ^*	κ^*	e^*_{IC}	ν^*
Value	1.20	0.16	0.05	2.176	0.25

ture. According to the Modified Cam Clay model, the elastic strain increment can be expressed as

$$[21] \qquad d\varepsilon_v^e = \left(\frac{\kappa^*}{1+e}\right)\frac{dp'}{p'}$$

$$[22] \qquad d\varepsilon_d^e = \frac{2\,(1+\nu^*)}{9\,(1-2\nu^*)}\left(\frac{\kappa^*}{1+e}\right)\frac{dq}{p'}$$

where ν^* is Poisson's ratio. The assumption of a constant Poisson's ratio leads to a nonconservative response to cyclic loading but is a common feature of the Cam Clay models.

Virgin yielding

For stress states on the yield surface and with $dp'_s > 0$, virgin yielding occurs. Based on the plastic volumetric deformation, i.e., eq. [16], the flow rule given by eq. [18], and considering the elastic deformation, the following stress and strain relationships for virgin yielding are obtained:

$$[23] \qquad d\varepsilon_v = \kappa^*\frac{dp'}{(1+e)p'} + (\lambda^* - \kappa^*)\frac{dp'_s}{(1+e)p'_s} + b\Delta e\left(\frac{M^*}{M^*-\eta}\right)\frac{dp'_s}{(1+e)p'_s}$$

$$[24] \qquad d\varepsilon_d = \frac{2\,(1+\nu^*)}{9\,(1-2\nu^*)}\left(\frac{\kappa^*}{1+e}\right)\frac{dq}{p'} + \frac{2\eta(1-\omega\Delta e)}{(M^{*2}-\eta^2)} \times \left[(\lambda^*-\kappa^*) + b\Delta e\left(\frac{M^*}{M^*-\eta}\right)\right]\frac{dp'_s}{(1+e)p'_s}$$

Softening

Soil is regarded as an elastic material for loading inside the yield surface. When the current stress state reaches the virgin yield surface at a point with $dp'_s > 0$, virgin yielding occurs. If the soil reaches the yield surface with $\eta > M^*$, softening occurs if the boundary conditions allow appropriate adjustment of the stress state. Otherwise, catastrophic failure will be predicted. During the softening process, the soil structure will be broken down, and the yield surface shrinks with the current stress state always remaining on it. In such cases, the yield surface contracts until the soil reaches a critical state of deformation where the structure of the soil is completely removed. It follows, therefore, that the softening behaviour of a soil should be described by virgin yielding equations.

The plastic volumetric strain increment given by eq. [16] is valid for the softening process. It may be noticed that because the yield surface shrinks, the volumetric deformation associated with intrinsic soil properties is negative, i.e., expansive. However, the volumetric deformation associated with the destructuring is determined by Δe, because both the terms $(M^* - \eta)$ and dp'_s are negative. For example, for a structured clay with positive Δe, the volumetric deformation associated with destructuring will be positive, i.e., compressive. Consequently, unlike a reconstituted clay, softening of structured clays can be accompanied by either overall volumetric expansion or overall volumetric compression. The occurrence of compressive volumetric deformation for clays both naturally and artificially structured has been widely reported by several researchers, including Lo (1972), Nambiar et al. (1985), Georgiannou et al. (1993), Burland et al. (1996), and Carter et al. (2000).

Fig. 4. Influence of destructuring index b on the shearing behaviour of soil. (a) Stress path in the e – ln p' space. (b) Stress and strain relationship. (c) Deviatoric stress and strain relationship at different scales.

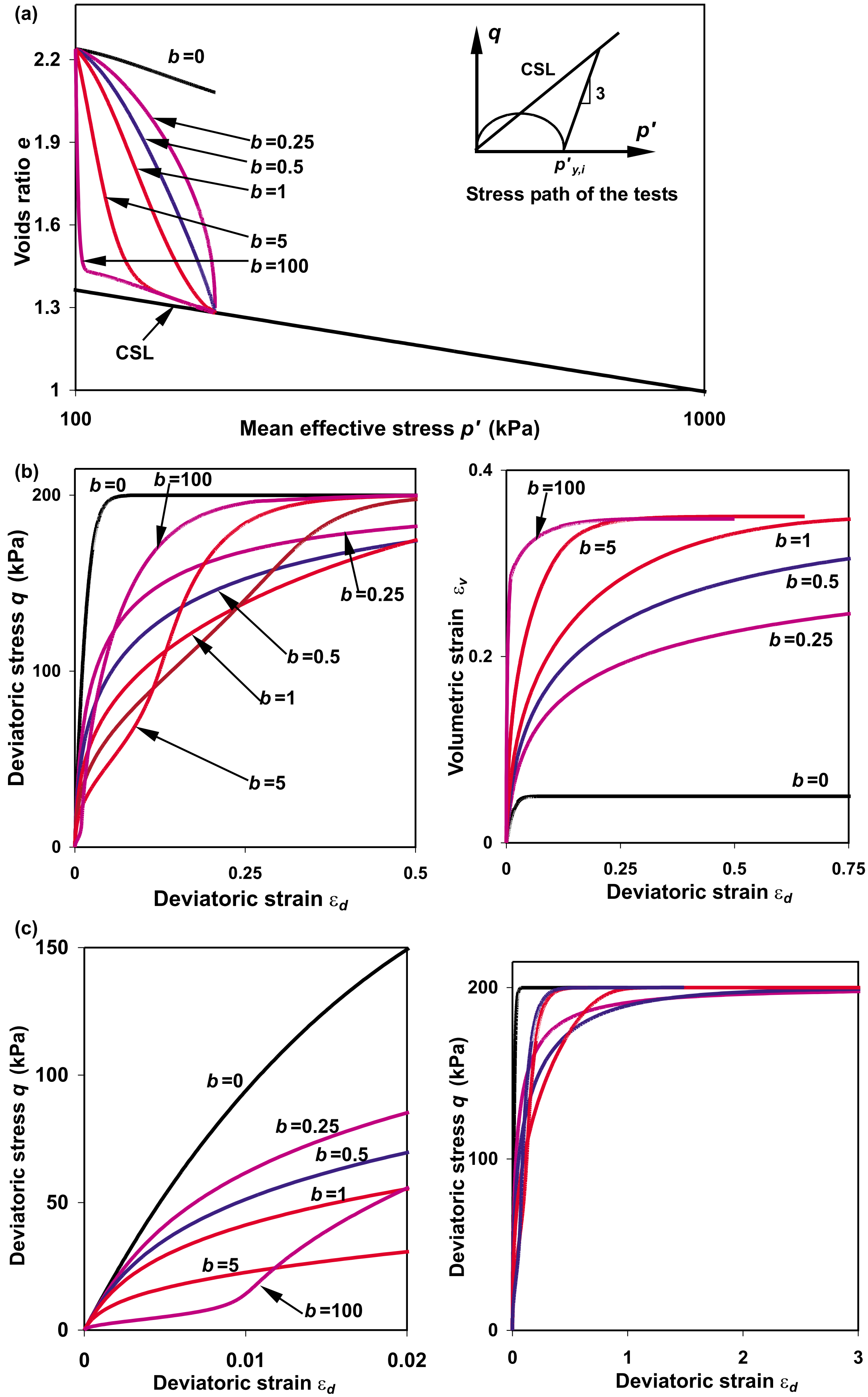
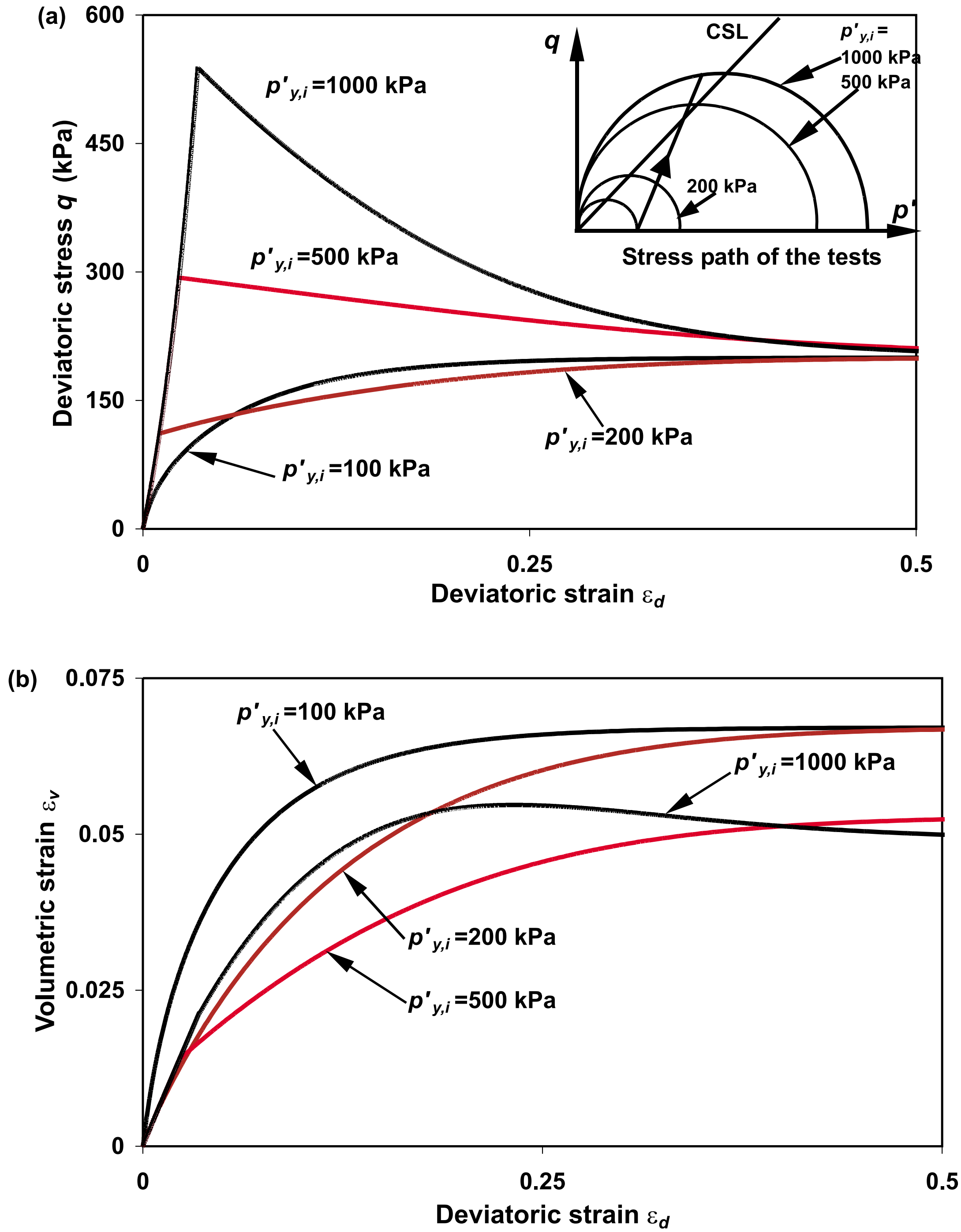


Fig. 5. Influence of the size of the initial yield surface on simulated soil behaviour. (*a*) Deviatoric stress and strain. (*b*) Deviatoric and volumetric strains.

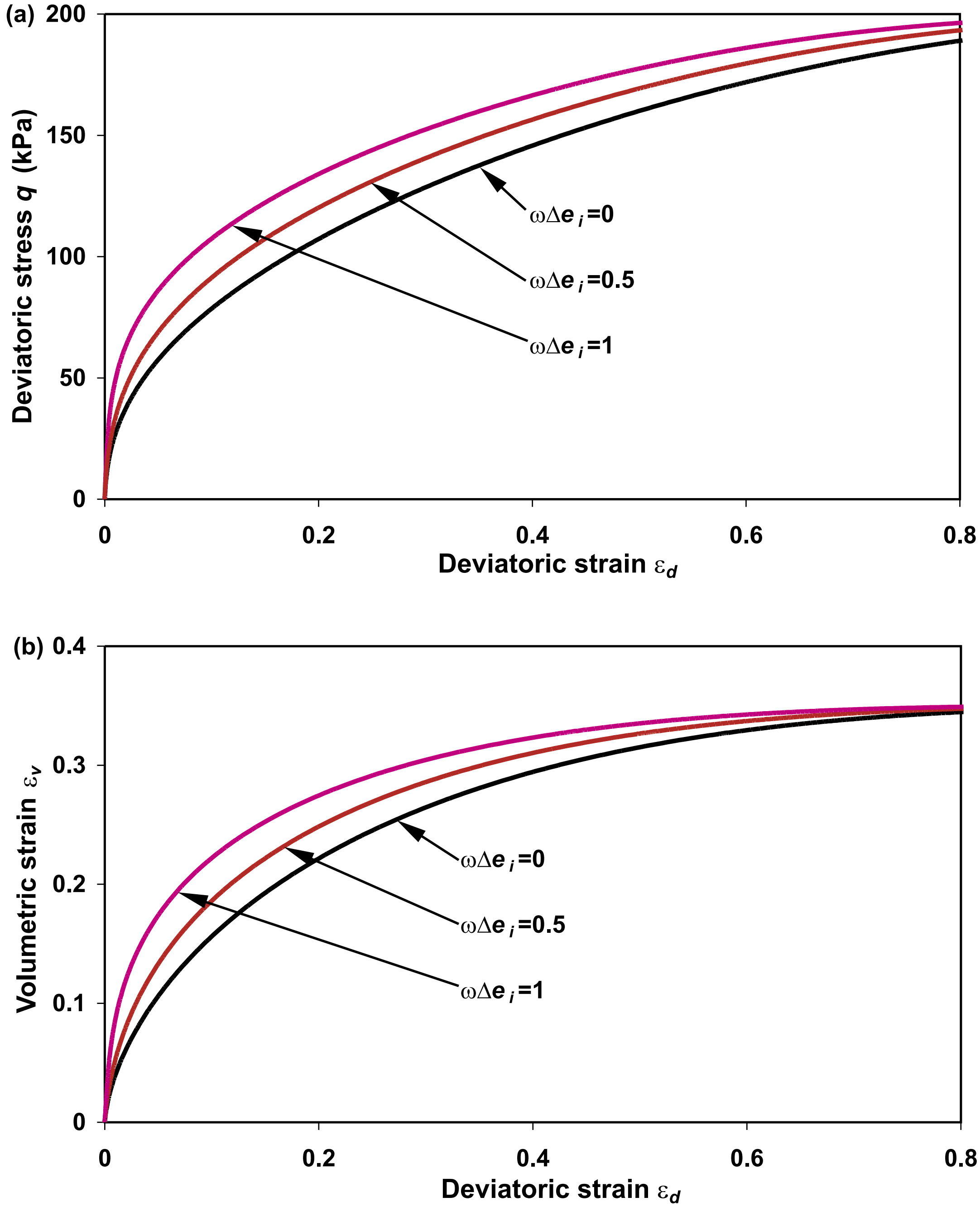


The expression for the plastic deviatoric strain increment contained within eq. [24] is thus modified to accommodate softening as follows:

$$\text{[25]} \qquad d\varepsilon_d^p = 2(1-\omega\Delta e)\left[(\lambda^*-\kappa^*) - b\Delta e\left(\frac{M^*}{M^*-\eta}\right)\right] \times \frac{\eta}{(M^{*2}-\eta^2)}\,\frac{dp'_s}{(1+e)p'_s}$$

In this case, only the sign of the plastic deviatoric strain associated with destructuring has been changed, so that the strain increment vector will point outside the yield surface. The elastic part of the deformation can be calculated by eqs. [21] and [22]. The total strain increments during softening are thus fully determined. As softening is a strained-controlled process, the change in the stress state can be decided from the size of the current structural yield surface. When the condition $\eta = M^*$ is reached, the structure of the soil is usually completely destroyed with $\Delta e = 0$, and there-

Fig. 6. The influence of parameter ω on simulated soil behaviour. (*a*) Deviatoric stress and strain. (*b*) Deviatoric and volumetric strains.



fore the structured clay has reached the critical state of deformation.

It may be noticed that for both virgin yielding and softening behaviour the soil may reach a state with $\eta = M^*$ but with $\Delta e \neq 0$. A specific example is the case where a soil reaches critical state by loading entirely inside the yield surface. In this case virgin yielding commences once the yield surface is reached and, according to eqs. [23] and [24], the soil is also in a state where it can be distorted continuously at constant volume ($d\varepsilon_v^p = 0$ and $d\varepsilon_v^p \rightarrow \infty$). Hence, the proposed model predicts that under special stress paths a soil may reach a critical state of deformation with its structure having not been removed completely. Consequently, in such cases the soil state will not be on the critical state line defined in $e - p'$ space. If there is evidence that the structure of a soil is destroyed completely after the soil reaches the critical state of deformation, then destructuring could be described by the plastic distortional strain instead of the current stress ratio. However, this possibility has not been pursued here.

Parameter determination

Eight parameters define the proposed model, i.e., M^*, e^*_{IC}, λ^*, κ^*, ν^*, b, $p'_{\mathrm{y,i}}$, and ω. The first five parameters, denoted by the * symbol, are intrinsic soil properties and are inde-

Fig. 7. Behaviour of Leda clay in an oedometer test (test data after Yong and Nagaraji 1977).

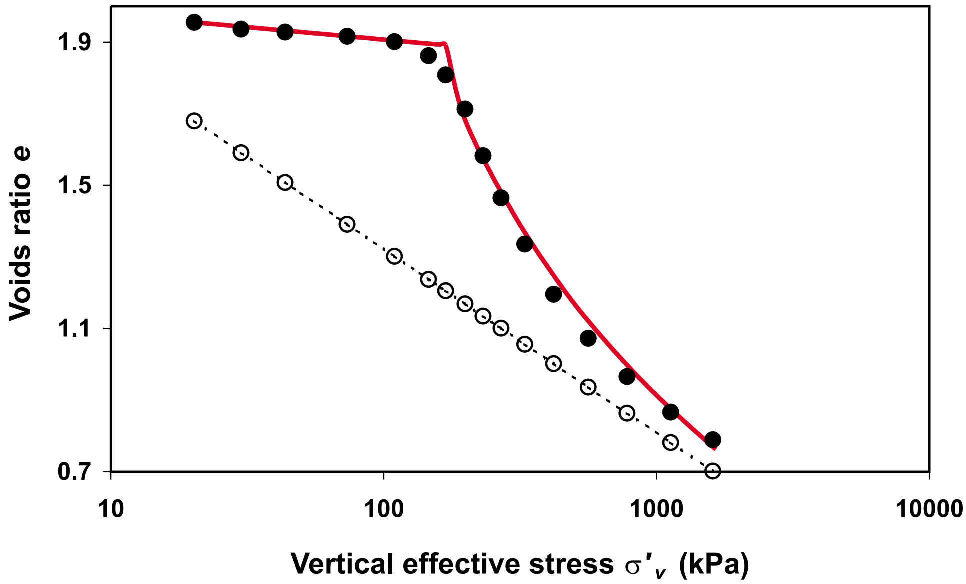


Table 2. Model parameters for Leda clay.

Parameter	M^*	λ^*	κ^*	e^*_{IC}	b	$\sigma'_{\mathrm{vy,i}}$ (kPa)
Value	1.2	0.223	0.03	2.338	1	169

Table 3. Model parameters for weathered Bangkok clay.

Parameter	M^*	λ^*	κ^*	e^*_{IC}	b	$\sigma'_{\mathrm{vy,i}}$ (kPa)
Value	0.9	0.4	0.1	3.82	0.55	35

pendent of soil structure. These five intrinsic parameters are the same as those adopted in the Modified Cam Clay model (Roscoe and Burland 1968). The influence of these parameters will not be investigated in this paper since studies of them are well documented (e.g., see Muir Wood 1990).

Three new parameters, i.e., b, $p'_{\mathrm{y,i}}$ and ω have been introduced to describe the influence of structure on the soil's mechanical behaviour. The value of the destructuring index, b, indicates the rate of destructuring during virgin yielding (Fig. 1), and $p'_{\mathrm{y,i}}$ represents the size of the initial yield surface for a structured soil (Fig. 1). The values of these two parameters can be determined directly from an isotropic compression test on an intact (undisturbed) soil specimen. Parameter ω was introduced to describe the influence of soil structure on the flow rule (e.g., see eq. [18]), and its value can be determined by applying the flow rule to the strains measured in a shearing test on an intact specimen provided that the elastic properties of the soil are known.

The values of the model parameters e^*_{IC}, λ^*, κ^*, b, and $p'_{\mathrm{y,i}}$ can be determined from isotropic compression tests. The values of parameters λ^*, κ^*, and b can be determined directly from any compression tests with constant η, whereas the values of e^*_{IC} and $p'_{\mathrm{y,i}}$ can only be determined directly from isotropic tests.

In geotechnical engineering practice, oedometer tests on soils are much more widespread than isotropic compression tests. Therefore, approximate methods for obtaining the parameters e^*_{IC} and $p'_{\mathrm{y,i}}$ from oedometer tests are also suggested. The approximate method is based on the assumption that the behaviour of reconstituted clay obeys the principles of critical state soil mechanics (e.g., see Muir Wood 1990 for details) and an empirical equation suggested by Jacky (1944) for one-dimensional compression. Details of this analysis can be found in a report by Liu and Carter (2002), from which the following approximate expressions have been derived:

$$[26]\quad p'_{\mathrm{y,i}} = \left(1 - \frac{2}{3}\sin\varphi_{\mathrm{cs}}\right)\left[1 + \left(\frac{3 - \sin\varphi_{\mathrm{cs}}}{6 - 4\sin\varphi_{\mathrm{cs}}}\right)^2\right]\sigma'_{\mathrm{vy,i}}$$

and

$$[27]\quad e^*_{\mathrm{IC}} = e^*_{\mathrm{ID}} + (\lambda^* - \kappa^*) \times \ln\left\{\left(1 - \frac{2}{3}\sin\varphi_{\mathrm{cs}}\right)\left[1 + \left(\frac{3 - \sin\varphi_{\mathrm{cs}}}{6 - 4\sin\varphi_{\mathrm{cs}}}\right)^2\right]\right\}$$

where φ_{cs} is the critical state friction angle measured from a triaxial compression test, $\sigma'_{\mathrm{vy,i}}$ is the value of the vertical effective yield stress, and e^*_{ID} is the value of the voids ratio at $p' = 1$ kPa in an oedometer test on a reconstituted soil.

Fig. 8. Compression behaviour of Leda clay with different values of η (test data after Walker and Raymond 1969).

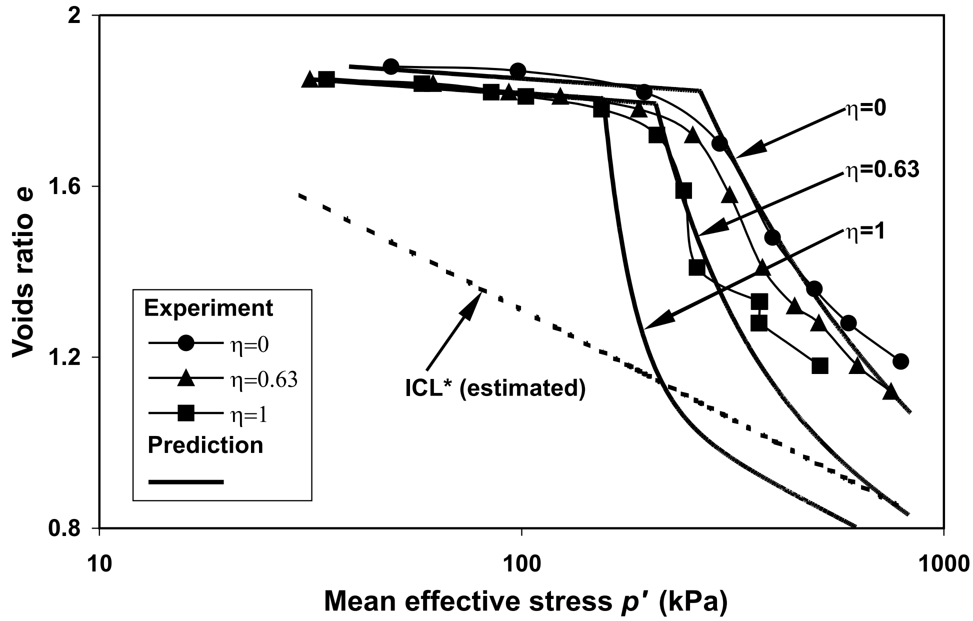
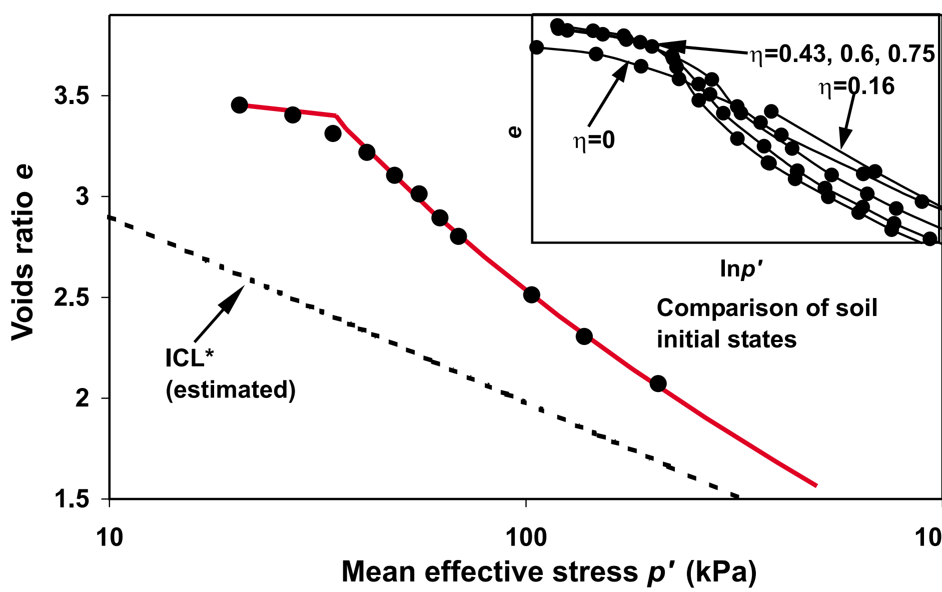


Fig. 9. Isotropic compression behaviour of weathered Bangkok clay (test data after Balasubramanian and Hwang 1980).



Features of the model

The influence of the three new parameters b, $p'_{y,i}$, and ω, and some features of the new Structured Cam Clay model are described in this section. Example calculations have been made using the new model, and the values of the intrinsic soil properties adopted are listed in Table 1. Based on these values, it is found that $e^*_{cs} = 2.1$, where e^*_{cs} is the well-known parameter defining the position of the critical state line in $e - p'$ space.

Parameter *b*

The influence of the destructuring index b is demonstrated by the simulations shown in Figs. 3 and 4. The following values of parameters for the soil structure were employed in these calculations: $p'_{y,i} = 100$ kPa, and $\omega = 1$. Six different values of b were assumed and they are 0, 0.25, 0.5, 1, 5, and 100. The initial stress state was defined by $p' = 100$ kPa and $q = 0$, and the initial value of the additional voids ratio was $\Delta e_i = 0.8$.

It can be seen in Fig. 3 that the rate of reduction in the additional voids ratio maintained by soil structure increases with the magnitude of b. For $b = 0$, the virgin compression line for a structured soil and that for its corresponding reconstituted soil are parallel in the $e - \ln p'$ space. Theoretically, in this case no destructuring takes place during the virgin yielding and the value of Δe remains unchanged.

The simulated shearing behaviour of a structured soil with different values of the destructuring index b is shown in Fig. 4. The effective stress paths simulated follow those in a

Fig. 10. Compression behaviour of weathered Bangkok clay (test data after Balasubramanian and Hwang 1980).

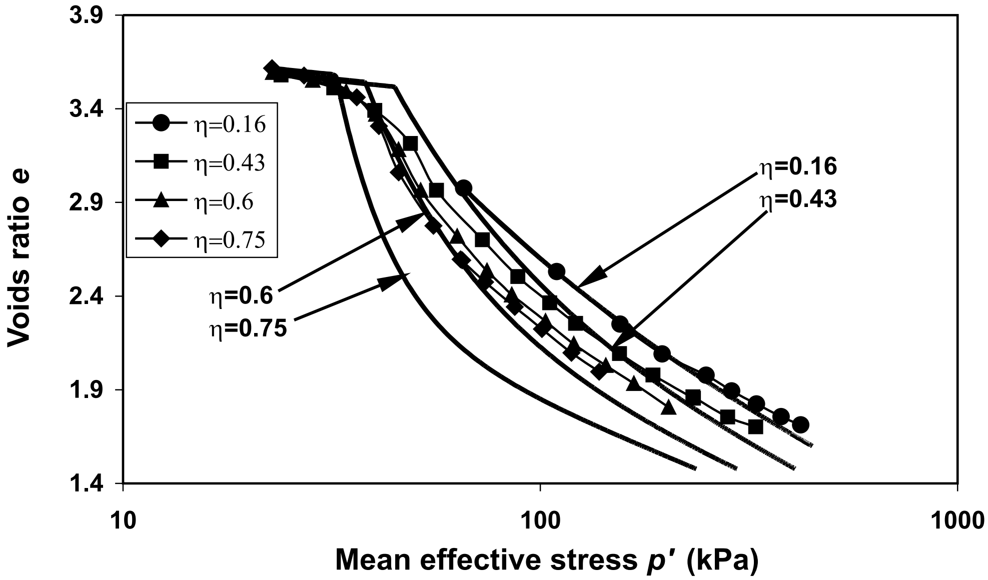
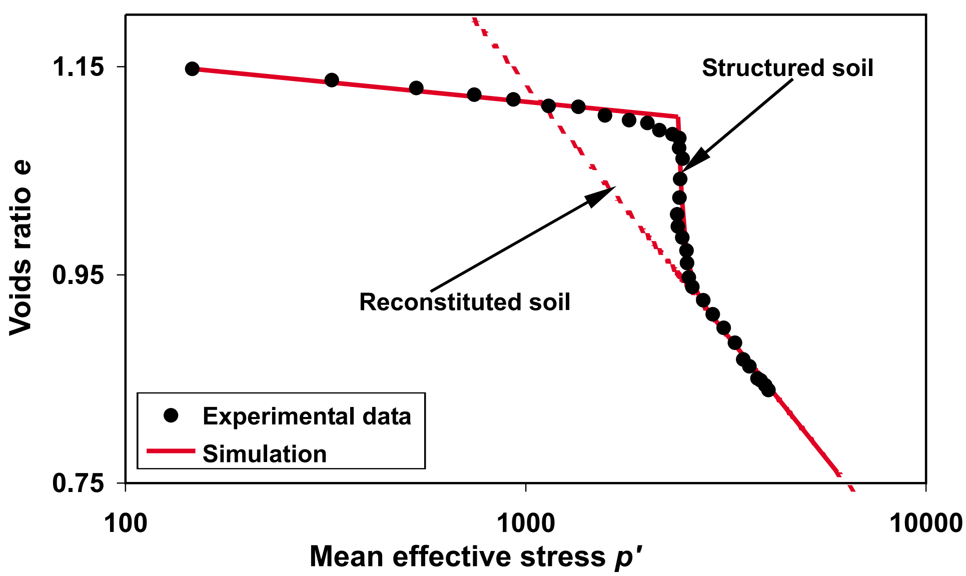


Table 4. Model parameters for a natural calcarenite

Parameter	M^*	λ^*	κ^*	e^*_{IC}	ν^*	b	ω	$p'_{\mathrm{y,i}}$ (kPa)
Value	1.45	0.208	0.0165	2.57	0.25	30	3.33	2400

Fig. 11. Isotropic compression test on calcarenite (test data after Lagioia and Nova 1995).



conventional drained triaxial test. Except for the unlikely situation with $b = 0$, the final state for a structured soil under monotonic shearing is independent of soil structure and corresponds to the critical state of deformation. At this state the soil structure has been completely removed. Where the initial stress state, voids ratio, and the loading path are exactly the same, the final states for all seven cases fall onto the same point on the critical state line (Fig. 4*a*), and the final values of the deviatoric stress and the volumetric strain are the same (Figs. 4*b* and 4*c*), indicating that parameter b has no influence on the final state of the soil under monotonic shearing if $b > 0$.

It may also be noticed in Figs. 4*b* and 4*c* that for soils with high b values, such as the case for $b = 100$, the shear stiffness may be non-monotonic. This is because for high values of b, soil structure is destroyed rapidly when virgin yielding commences, and hence the stiffness of the soil is low during the early stages of loading. After the structure of the soil has essentially been removed, the behaviour of the original structured soil is almost the same as that of the cor-

Fig. 12. Shearing behaviour of calcarenite at σ'_3 = 25 kPa (test data after Lagioia and Nova 1995). (*a*) Deviatoric stress and strain relationship. (*b*) Volumetric strain and deviatoric strain relationship.

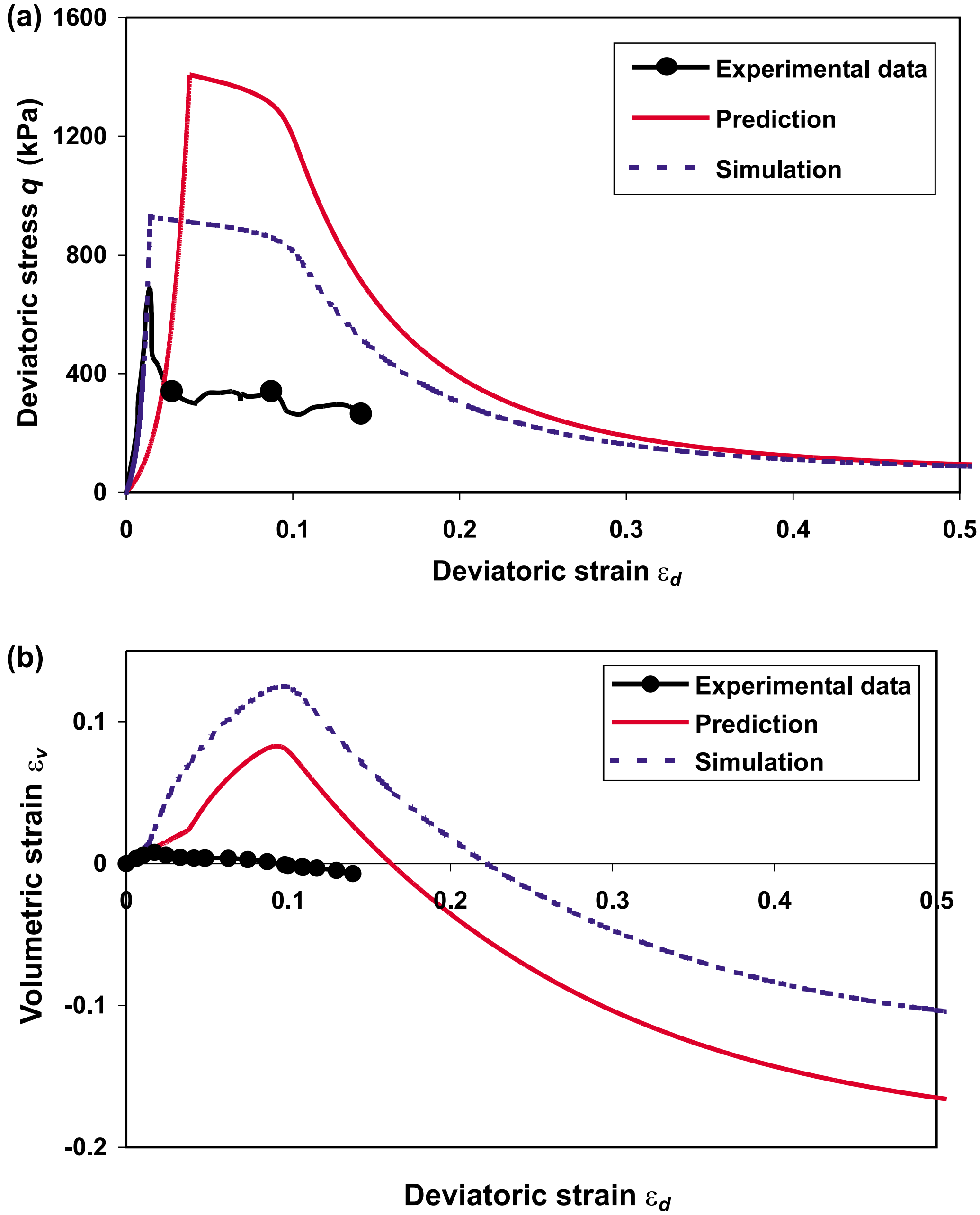


Table 5. Revised values of model parameters for a natural calcarenite.

Parameter	M^*	λ^*	E^* (kPa)	e^*_{IC}	ν^*	b	ω	$p'_{y,i}$ (kPa)
Value	1.45	0.208	76 923	2.383	0.13	30	3	2400[a]

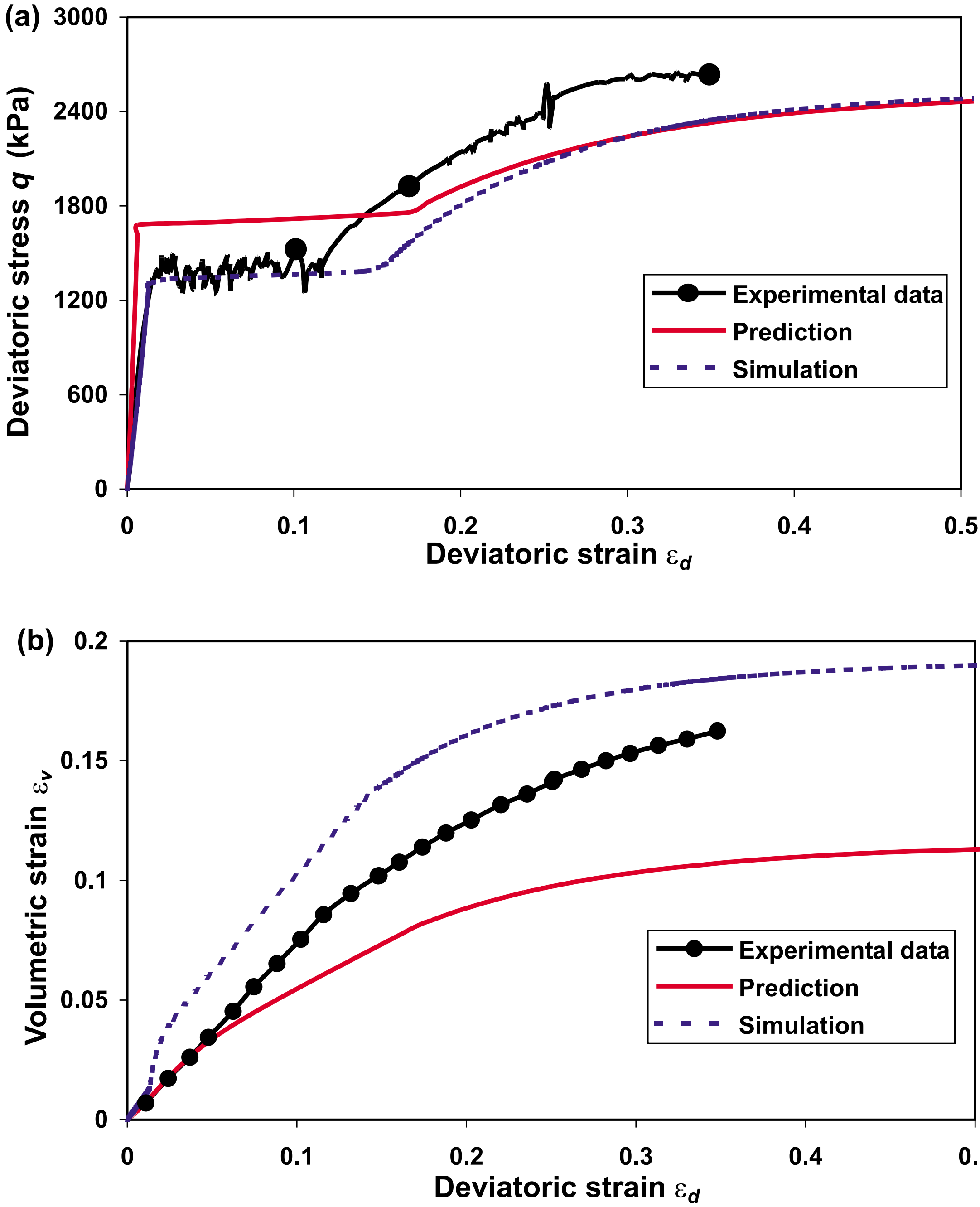
[a]Aspect ratio of the yield surface is 1.12.

responding reconstituted soil and the stiffness of the soil then is usually higher. This phenomenon has been observed experimentally (Lagioa and Nova 1995; Arces et al. 1998).

Parameter $p'_{y,i}$

To illustrate the influence of the size of the initial yield surface defined by $p'_{y,i}$, the soil parameters listed in Table 1, together with b = 1 and ω = 1, were adopted. Four different values of $p'_{y,i}$ were assumed and they are 100, 200, 500, and 1000 kPa. The influence of parameter $p'_{y,i}$ on the simulations is shown in Fig. 5. The effective stress paths simulated follow those in a conventional drained triaxial test in which the axial loading only is increased. The initial stress state for all six cases was the same, with p' = 100 kPa and q = 0, and the initial voids ratio was e_i = 1.439. For this initial soil state and the given intrinsic soil properties, the initial yield surface for

Fig. 13. Shearing behaviour of calcarenite at σ'_3 = 900 kPa (test data after Lagioia and Nova 1995). (*a*) Deviatoric stress and strain relationship. (*b*) Volumetric strain and deviatoric strain relationship.



the corresponding reconstituted soil is p'_o = 100 kPa. Therefore, for the particular situation with $p'_{y,i}$ = 100 kPa the soil is in a reconstituted state and has no structure. The soil is structured for situations with $p'_{y,i}$ > 100 kPa. Thus, this set of computations simulates the development of structure for soil at constant voids ratio and the same stress state. The following features of soil behaviour have been predicted:

(1) The final state of a structured soil under monotonic shearing is independent of the size of the initial yield surface.

(2) Unlike a reconstituted soil, the peak strength of a structured soil is dependent on soil structure as well as on the initial stress state, voids ratio, and the stress path.

(3) If softening occurs the peak strength of a structured soil reduces to the critical state strength more rapidly than for a reconstituted soil. This is because the softening of a structured soil may be attributed to two factors, destructuring and volumetric expansion.

(4) The response pattern usually labeled "dry behaviour" (Schofield and Wroth 1968) may not be observed for a structured soil, i.e., volumetric expansion may not occur when the softening process starts. It is seen that for the case with $p'_{y,i}$ = 500 kPa there is a continuous volumetric compression accompanying the softening process. For the simulation with $p'_{y,i}$ = 1000 kPa, volumetric expansion, although

Fig. 14. Shearing behaviour of calcarenite at σ'_3 = 1300 kPa (test data after Lagioia and Nova 1995). (*a*) Deviatoric stress and strain relationship. (*b*) Volumetric strain and deviatoric strain relationship.

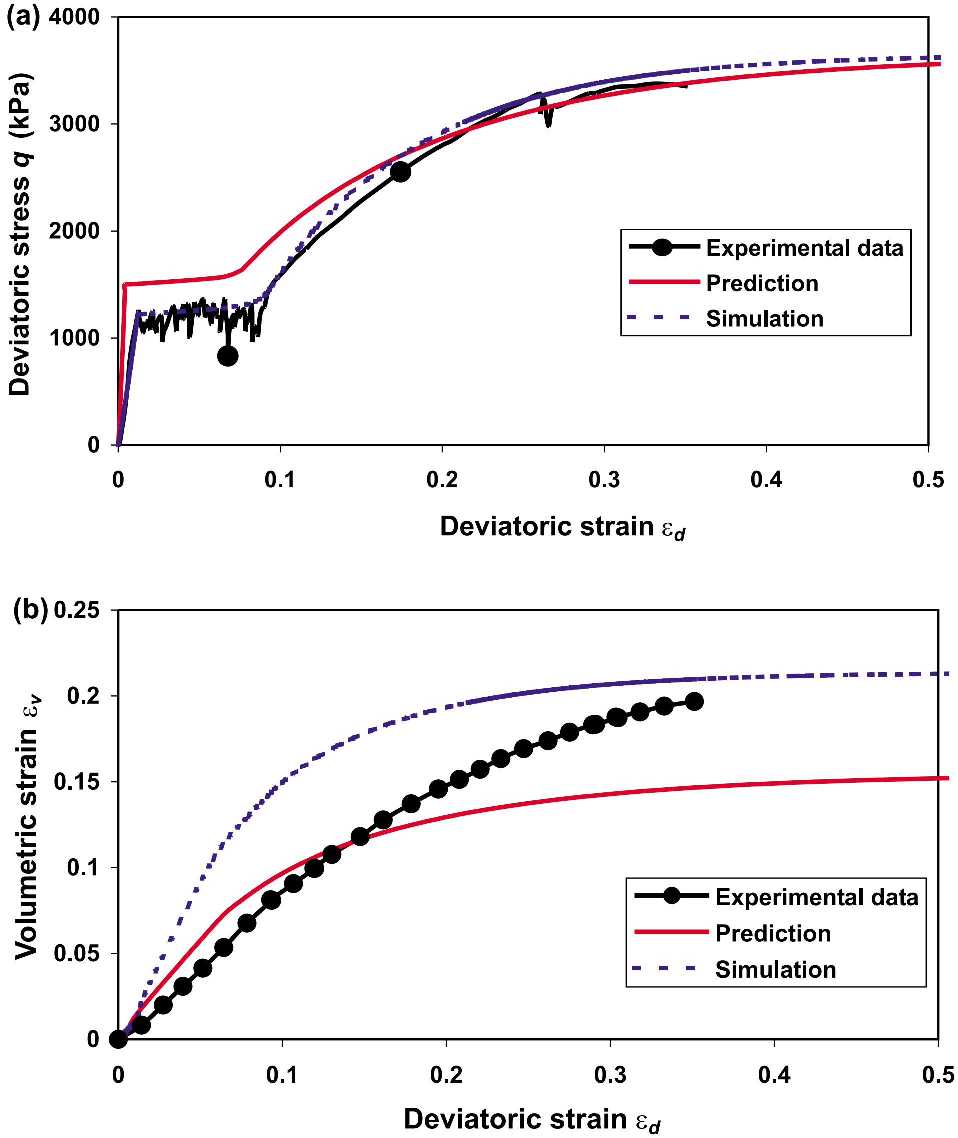


Table 6. Model parameters for natural Corinth marl.

Parameter	M^*	λ^*	κ^*	e^*_{IC}	ν^*	b	ω	$p'_{y,i}$ (kPa)
Value	1.38	0.04	0.008	0.775	0.25	0.4	4.9	3800

observed, starts only after a considerable amount of softening has occurred.

Parameter ω

Figure 6 shows the influence of the parameter ω. Values of the soil parameters listed in Table 1 were adopted for these simulations, together with b = 1 and $p'_{y,i}$ = 100 kPa. The initial stress state is defined as p' = 100 kPa and q = 0, and the initial value of the additional voids ratio was Δe_i = 0.8. Three different values of ω were assumed, i.e., ω = 0, 0.625, and 1.25 (the corresponding values of $\omega\Delta e_i$ are 0, 0.5, and 1, respectively).

Based on the study of the three parameters describing soil structure, it can be concluded that the final state of a structured soil under monotonic shearing predicted by the proposed model for situations with $b > 0$ is the critical state of deformation, which is independent of soil structure. The existence of the critical state of deformation for geomaterials and the implication that their mechanical properties are independent of material structure have been widely observed features of soil behaviour (e.g., Been and Jefferies 1985; Ishihara 1993; Novello et al. 1995; Carter et al. 2000).

Model evaluation

The validity and utility of the proposed model were evaluated by making comparisons between the model predictions and experimental data. The compression behaviour of two

Fig. 15. Shearing behaviour of calcarenite at σ'_3 = 3500 kPa (test data after Lagioia and Nova 1995). (*a*) Deviatoric stress and strain relationship. (*b*) Volumetric strain and deviatoric strain relationship.

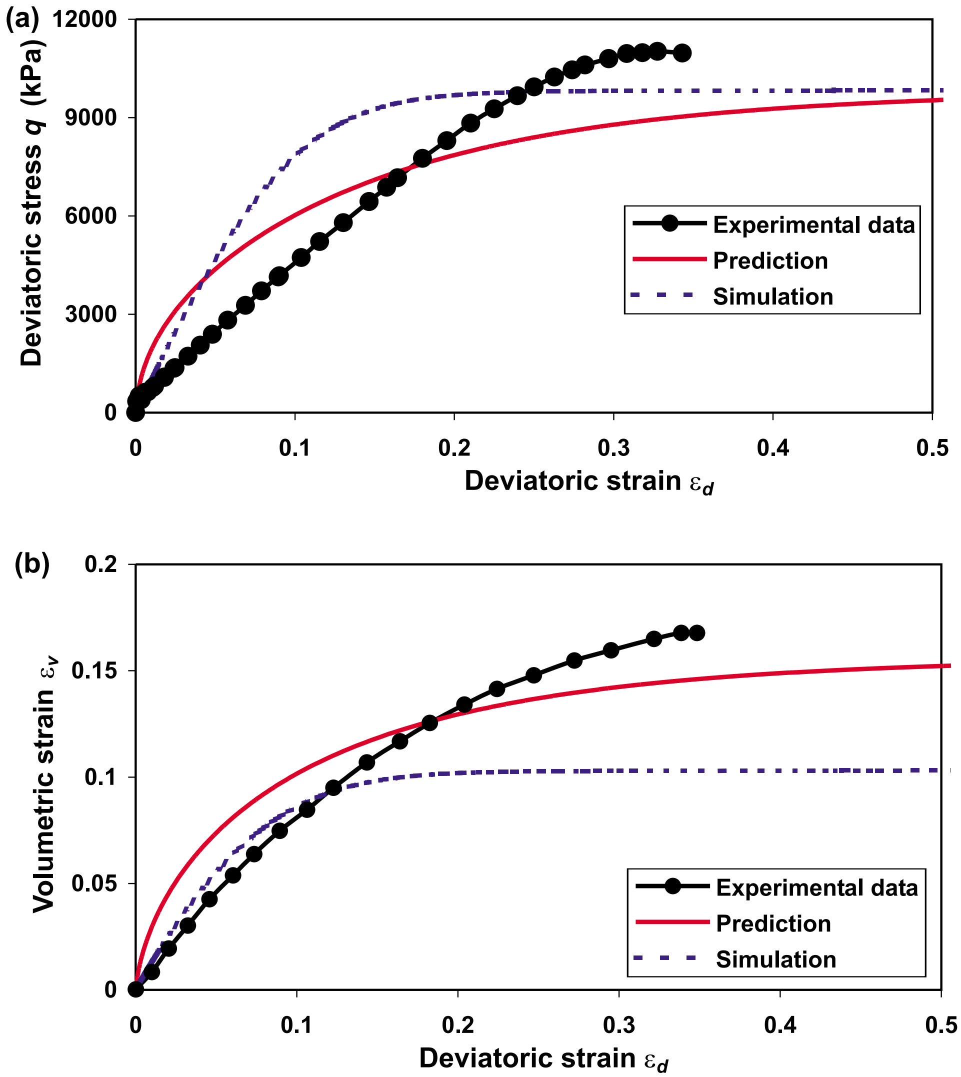


Table 7. Model parameters for a natural clayshale.

Parameter	M^*	λ^*	E^* (kPa)	e^*_{IC}	ν^*	b	ω	$p'_{\mathrm{y,i}}$ (kPa)
Value	1.45	0.06	73 000	0.668	0.25	0.2	4	3800

different clays has been considered; natural Leda clay and weathered Bangkok clay. The shearing behaviour of three other structured soils, i.e., natural calcarenite, Corinth marl, and a clay shale, have also been considered. Further details of these particular comparisons may be found in a report by the authors (Liu and Carter 2002), together with more detailed explanations of how the model parameters were determined in each case.

Compression behaviour of two clays

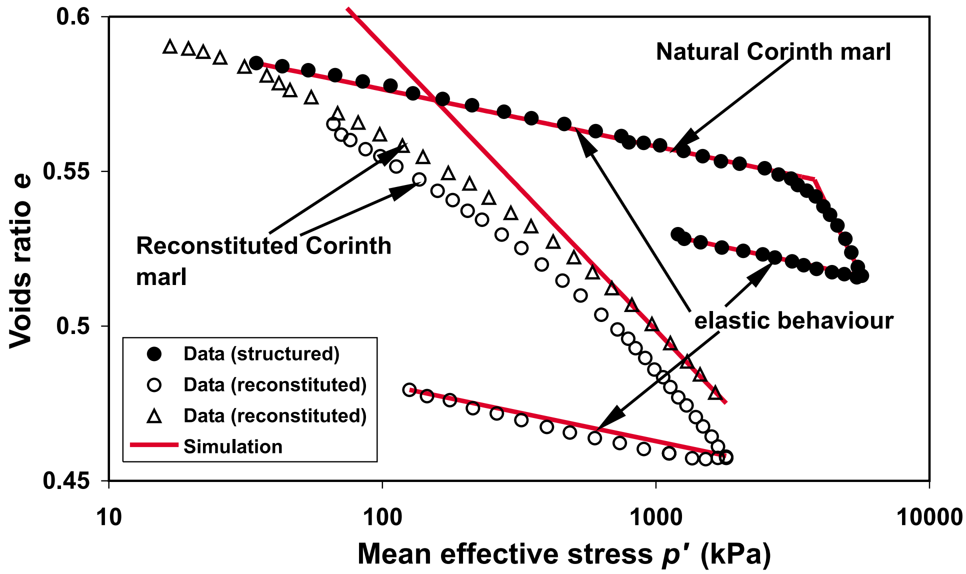
Two sets of compression tests on different soils were simulated and for these tests only the volumetric deformation has been computed. As can be seen from eq. [23], the volumetric deformation is not influenced by the values of parameters ν^* and ω. Hence, for the present purpose there is no need to determine or specify values for these parameters.

Leda clay

The first group of test data includes five compression tests on natural soft Leda clay performed by Yong and Nagaraj (1977) and Walker and Raymond (1969). The two oedometer tests on the natural and reconstituted Leda clay reported by Yong and Nagaraji (1977) were used to identify soil parameters, and their values are listed in Table 2. The initial state of the structured soil is defined by σ'_{v} = 20 kPa and e = 1.96. As shown in Fig. 7, the compression behaviour of Leda clay is well simulated by the model.

Three tests performed by Walker and Raymond (1969) were used to evaluate the model's predictions. Although all

Fig. 16. Isotropic compression behaviour of Corinth marl (test data after Anagnostopoulos et al. 1991).



of the specimens tested by both Yong and Nagaraji (1977) and Walker and Raymond (1969) were Leda clay, they were obtained from different locations in the same area. It is assumed that these specimens differed only in the size of the initial yield surface, i.e., the different Leda clay samples possessed the same mineralogy and type of structure but may have had different degrees of structure. The three compression tests were with $\eta = 0$, 0.63, and 1, respectively. The experimental data for the test with $\eta = 0$ were used to identify the size of the yield surface, and it was found that $p'_{y,i}$ = 265 kPa. The predicted compression behaviour of the Leda clay is shown in Fig. 8. It is seen that the proposed model gives an approximate but reasonable description of the compression behaviour of natural Leda clay.

Bangkok clay

The second group of test data includes the results of five compression tests on weathered Bangkok clay performed by Balasubramanian and Hwang (1980). Values of the model parameters are listed in Table 3. The one-dimensional compression curve for the reconstituted soil type was estimated by the empirical method suggested by Burland (1990), upon which the parameters λ^* and e^*_{IC} were estimated. The simulated behaviour of Bangkok clay is shown in Fig. 9. It may be noticed that the compression behaviour of the Bangkok clay is well simulated in this case.

The predicted compression behaviour of Bangkok clay with $\eta = 0.16$, 0.43, 0.6, and 0.75 is shown in Fig. 10. As may be seen in the inset in Fig. 9, some differences in the initial soil structure exist among the samples used for the five tests. The test specimens were obtained from the field and some variation in the specimens would normally be expected. It can be seen that the initial soil states for the five specimens may be divided into three groups, i.e., the test with $\eta = 0$, the test with $\eta = 0.16$, and tests with $\eta = 0.43$, 0.6, and 0.75. It is assumed in the simulations that the differences in the initial states of the soil can be represented adequately by the differences in the sizes of the initial structural yield surfaces. It may be seen from the compression curve that the initial stress state for the test with $\eta = 0.16$ is on the yield surface, i.e., $p'_{y,i}$ = 67.6 kPa. The size of the initial yield surface for the other three specimens is 45 kPa (the test with $\eta = 0.43$ is used to identify the value of this parameter). Overall, it is seen that the proposed model gives a reasonably good approximation of the compression behaviour of weathered Bangkok clay.

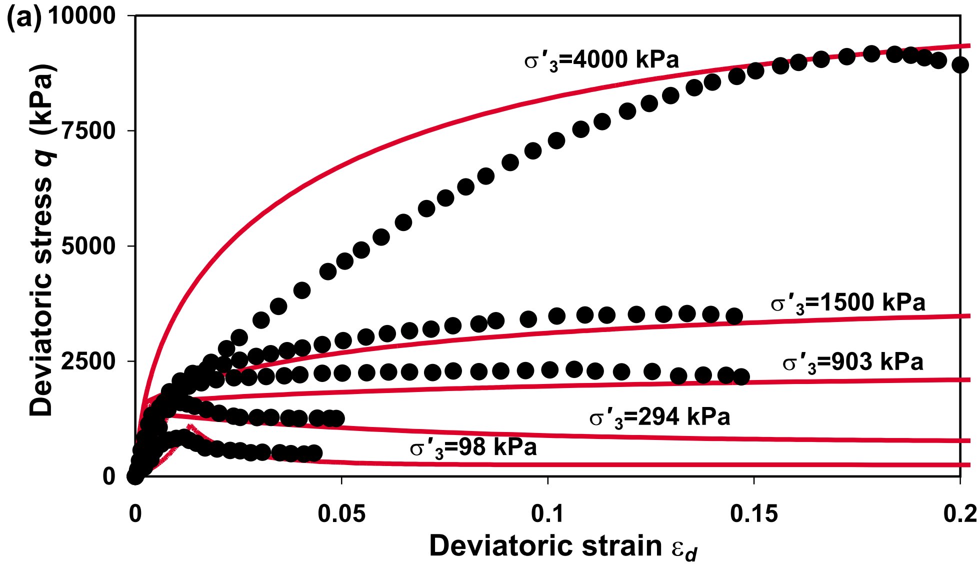
Behaviour of a natural calcarenite

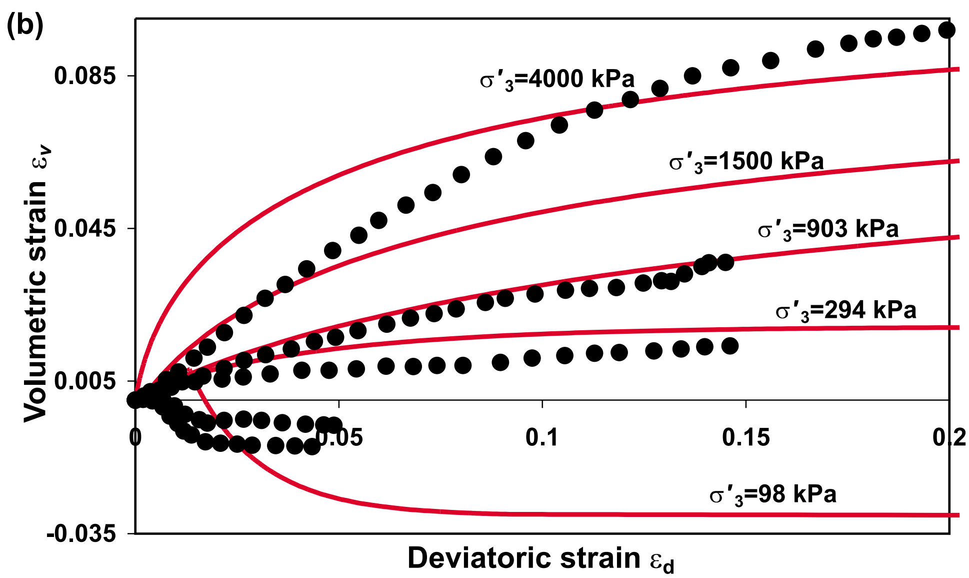
The results of experimental work carried out by Lagioia and Nova (1995) on a natural calcarenite have been compared with the model predictions. The natural calcarenite was formed by marine deposition. It is a coarse-grained material with a high degree of uniformity and calcareous interparticle cement. An isotropic compression test on the cemented soil was used to identify soil parameters and their values are listed in Table 4. The measured and simulated isotropic compression behaviour of the calcarenite is shown in Fig. 11.

The values of parameters e^*_{IC} λ^*, κ^*, b, and $p'_{y,i}$ were obtained directly from the experimental data. The value of Poisson's ratio was assumed. The critical state strength for the natural calcarenite was reported by Lagioia and Nova (1995) as $M^* = 1.45$. The initial state for the structured soil is defined by $p' = 147$ kPa and $e = 1.148$, and so the initial value of the additional voids ratio due to soil structure is $\Delta e_i = 0.15$. The value of parameter ω was estimated by assuming $1 - \omega\Delta e_i = 0.5$ based on the constraint condition eq. [19].

By using the values of the model parameters listed in Table 4, the behaviour of the natural calcarenite under conventional drained triaxial tests was predicted. In all, eight tests were considered with the confining pressure σ'_3 ranging from 25 to 3500 kPa. A comparison between test results and the predictions for four tests are shown in Figs. 12–15. For the test with $\sigma'_3 = 3500$ kPa, the initial stress state is much larger than the size of the initial structural yield surface. According to the proposed model, the structure of the soil at $\sigma'_3 = 3500$ kPa is effectively completely destroyed, since the soil has a very high destructuring index, i.e., $b = 30$. Thus,

Fig. 17. Behaviour of natural Corinth marl (test data after Anagnostopoulos et al. 1991). (*a*) Deviatoric stress and strain relationship. (*b*) Volumetric and deviatoric strain relationship.





the soil behaved essentially as a reconstituted material throughout this test. Destructuring of this sample was confirmed by Lagioia and Nova (1995). Considering the wide range of initial stresses, it is seen that the proposed model provides successful predictions of the behaviour of this natural and highly structured calcarenite.

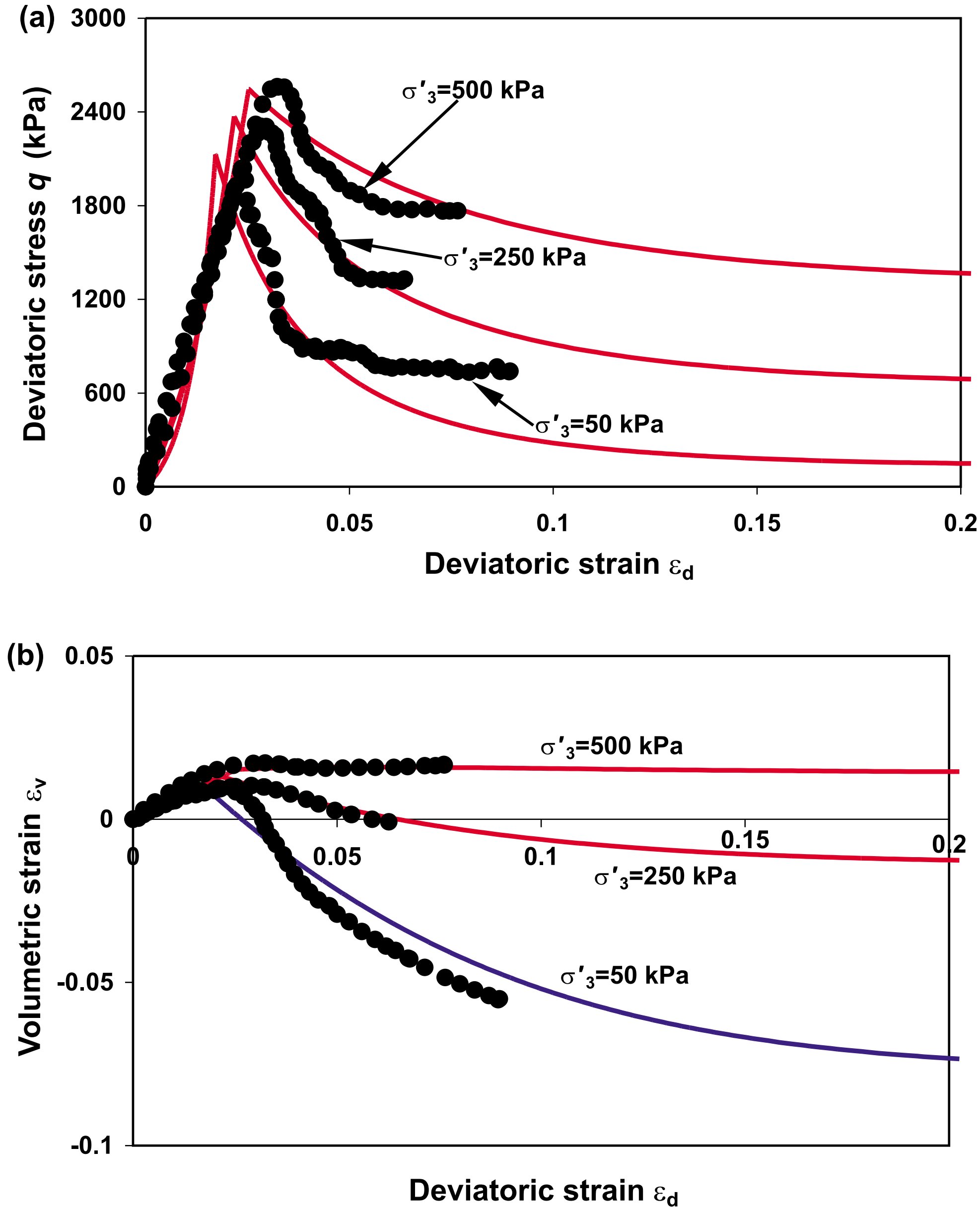
"Simulations" of the behaviour of this calcarenite have also been made, where a part of the test data was first employed to determine the model parameters. The values of the model parameters used for the simulations (as opposed to "predictions") are listed in Table 5.

Comparisons between the theoretical simulations and the experimental data for four tests are also shown in Figs 12–15. The simulations are represented by broken lines. It can be seen that the behaviour of the natural calcarenite has been simulated satisfactorily with the set of revised model parameters. It is noted that the differences in parameter values adopted for the "predictions" (Table 4) and "simulations" (Table 5) are generally relatively minor, with one important exception. It is noted in particular that for the simulations the value of κ^* was permitted to vary from test to test while the value of E^*, Young's modulus for fully drained conditions, was held constant for all tests.

Behaviour of Corinth marl

Results of the experimental work performed by Anagnostopoulos et al. (1991) on a natural Corinth marl have been compared with the model simulations. Two sets of tests were reported: a set of three isotropic compression tests on both natural and reconstituted Corinth marl, and a set of five shearing tests on the intact and partially destructured soil. From the first set of tests, the soil parameters e^*_{IC}, λ^*, κ^*, b, and $p'_{\mathrm{y,i}}$ have been identified, and their values are listed in Table 6. The measured and simulated behaviour of

Fig. 18. Shearing behaviour of clayshale (test data after Wong 1998). (*a*) Deviatoric stress and strain relationship. (*b*) Volumetric strain and deviatoric strain relationship.



the soil in isotropic compression is shown in Fig. 16. Two isotropic compression tests on reconstituted Corinth marl are reported, and some small discrepancies on the isotropic compression line are observed between the two test samples, which were assumed to be identical. For most natural soils, i.e., not artificially manufactured samples, some discrepancy in the stress and strain behaviour is expected due to material variance. The isotropic test that gave the better simulation of the shearing behaviour of the Corinth marl was selected for identifying the model parameters.

The initial value of the additional voids ratio, i.e., Δe_i, was found to be 0.102. The value of parameter ω was estimated by assuming $1 - \omega\Delta e_i = 0.5$, based on eq. [19]. The value of Poisson's ratio was assumed as 0.25. The critical state strength M^* was determined based on the second set of tests, i.e., the final strength of the soil reached under monotonic shearing. The value of M^* is the only parameter determined from the data from the shearing tests. Both the simulations and the experimental data are shown in Fig. 17.

For all five compression tests, the confining pressures were kept constant at 98, 294, 903, 1500, and 4000 kPa, respectively, and the axial loading was increased until the specimens failed. For the first four tests, the initial stress state of the soil was within the initial structural yield surface, defined by $p'_{y,i}$ = 3800 kPa, and therefore the soil samples for these tests are assumed to be intact. For the fifth test, the initial stress state of the soil exceeds the initial structural yield surface and the soil sample has experienced partial destructuring. It is seen that the proposed model gives a reasonable representation of the behaviour of the highly structured, stiff Corinth marl.

Behaviour of La Biche clayshale

The results of the experimental work performed by Wong (1998) on a natural clayshale, a Canadian La Biche shale, have been compared with the model simulations. Table 7 shows the values of the model parameters used. A comparison between the simulations and the experimental data is shown in Fig. 18. It is seen that the proposed model has the capability of modelling the behaviour of this structured clayshale. For the clayshale in the test with σ'_3 = 500 kPa, the volumetric deformation remains compressive even though the shear strength of the soil softens from a peak of 2600 to 1800 kPa. This type of behaviour has been widely observed in structured clays and clayshales (e.g., Bishop et al. 1965; Lo 1972; Georgiannou et al. 1993; Robinet et al. 1999; Carter et al. 2000).

As explained elsewhere by Liu and Carter (2002), adopting a constant value of E^* overall implies a value of κ^* that varies between tests. The following values were used here: κ^* = 0.005 for the test with σ'_3 = 50 kPa, κ^* = 0.012 for the test with σ'_3 = 250 kPa, and κ^* = 0.02 for the test with σ'_3 = 500 kPa.

Conclusion

The Modified Cam Clay model has been generalized so that the isotropic variation of the mechanical properties resulting from the presence of soil structure can be described. A new hierarchal model, referred to as the Structured Cam Clay model, was proposed. Besides the original five parameters introduced in the Modified Cam Clay model, three new parameters have been introduced. They are: b, the destructuring index, $p'_{y,i}$, the size of the initial structural yield surface, and ω, a parameter describing the effect of soil structure on the plastic flow rule. Values of the first two parameters can be determined from an isotropic compression test or an oedometer test. The third parameter can be determined from the volumetric strain and deviatoric strain curve obtained from a shearing test.

Overall, the proposed model has been used to predict both the compression and shearing behaviour of five structured soils. The computations cover wide ranges of stress, initial voids ratio, and soil structure. It has been demonstrated that the proposed model successfully describes many important features of the behaviour of structured soils and has significantly improved the performance of the Modified Cam Clay model by quantifying the important influence of soil structure. In particular, the fact that the new model is able to predict simultaneous material softening and compressive volumetric strain is considered to be a distinct advantage over the Modified Cam Clay model.

Because the Modified Cam Clay model forms the basis of the new model, obviously the new model is suitable only for those soils for which the behaviour of the reconstituted material can be described adequately using the Modified Cam Clay model. Anisotropic features of soil behaviour, either due to the reconstituted parent soil or due to the presence of soil structure, were not studied in this paper but will be the topics of future research.

This paper has concentrated on introducing the new model and its basic features and therefore only the behaviour of soil under fully drained conditions has been considered. The authors also plan a future investigation of the performance of the model for undrained conditions and a systematic study on the identification of model parameters from tests commonly carried out by engineers in geotechnical practice.

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