Structured Cam Clay Model with Cementation Effect

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ABSTRACT: In this paper, the theoretical framework of the Structured Cam Clay (SCC) model is extended to describe the behaviour of cemented clay. An operative mean effective stress parameter for soils with cohesion/cementation is introduced to include the influence of cementation on the strength and the deformation of cohesive soils. For simplicity, the removal of cementation is assumed to take place during the process of the rearrangement of soil particles to form the final critical state of deformation. Based on experimental observation, a simple destructuring function is proposed for the removal of cementation structure; especially, for artificially strongly-cemented clays. The model is suitable for describing the behaviour of clays in reconstituted, naturally structured, and artificially cemented states under monotonic loading or with simple stress reversal. The revised model is then employed to simulate the behaviour of cemented clays with various degrees of cementation and confining stresses. It is seen that main features of the complicated behaviour of cemented clays can be represented reasonably well by the model. Some studies on model parameters are also presented.

Keywords: Clays, Cementation, Soil structure, Strength, Constitutive relations.

1. INTRODUCTION

The formulation of Cam Clay model in the 1950's marks the establishment of soil mechanics as a science and various unrelated topics in the subject have been united into one cohesive framework, the critical sate soil mechanics (Schofield and Wroth 1968). As compared with metal plasticity (Hill, 1950), the most significant progress in the model is the introduction of the soil voids ratio into constitutive modelling of soil behaviour and the modelling of the stress-ratio-dependent volumetric deformation. Since then a large number of models have been developed within the theoretical framework of Cam Clay, characterised by the existence of a critical state of deformation as the final failure state and volumetric-deformation-dependent hardening (e.g., Gens and Potts, 1988; Yu, 1998; Liu and Carter, 2000a). These models generally successfully represent the behaviour of laboratory reconstituted soils under various circumstances. There are only two basic types of soil behaviour modelled: "the wet behavior" and "the dry behavior", as suggested by Schofield and Wroth (1968). The performances of these models are much less satisfactory for natural soils because of the complexity of soil behaviour associated with the influence of soil structure.

By "soil structure" the arrangement and bonding of soil constituents are meant, and for simplicity it encompasses all features of a soil that are different from those of the corresponding reconstituted soil. There has been large amount of experimental work on the stress-strain behaviour of natural soils in the last five decades (*e.g.*, Burland, 1990; Leroueil and Vaughan, 1990; Amorosi and Rampello, 2007; Hong et al., 2012). Meanwhile great progress has also been made in the constitutive modelling of the behaviour of clays with various structures as found in situ (e.g., Wheeler et al., 2003; Kavvadas and Amorosi, 2000; Rouainia and Muir Wood, 2000; Baudet and Stallebrass, 2004; Chai et al., 2004; Koliji et al., 2010; Masín, 2007).

In geotechnical engineering practice, soft clays are often encountered. Effective soil improvement techniques are often required to improve the stiffness as well as the strength of these soils. One of the widely used methods is cement stabilisation. During the past three decades, a large amount of laboratory and site investigations have been performed in order to understand the mechanical properties of the cemented soft soil and to characterise its behaviour (Huang and Airey, 1998; Horpibulsuk et al., 2004a and b; Lorenzo and Bergado, 2004; Consoli et al., 2006; Lee et al., 2005). There are also some important developments on modelling the behaviour of cemented soil (e.g., Asaoka et al., 2000; Yan and Li, 2011; Suebsuk et al., 2011). However, these constitutive models are generally complicated and further research is still needed in order to provide models suitable for numerical analyses for practical problems.

A simple predictive model, the Structured Cam Clay (SCC) model was proposed by Liu and Carter (Liu and Carter, 2002; Carter and Liu, 2005). The model was formulated based on the introduction of the effect of soil structure into the well known Modified Cam Clay model. It has been shown that the simple SCC model has successfully captured many important features of the behaviour for naturally structured clay with negligible or low cementation. Horpibulsuk et al. (2010) made a study on extending the SCC model for the behaviour of cemented clays and demonstrated that the model captured some main feature of artificially cemented clays. In this paper, the model proposed by Horpibulsuk et al. (2010) is introduced with some modifications, especially for artificially strongly cemented clays. For this type of soil, the cementation structure will improve the shear strength and stiffness of the parent soil significantly before the removal of the structure. These improvements remain even when the soil is sheared to failure at large shear strain as can be performed in conventional laboratory tests. A basic concept in the model for capturing the influence of cementation on soil mechanical behaviour is "the operative mean effective stress", in which the cementation effect is represented as an isotropic increment of the effective stress. A simple destructuring function is suggested to describe the removal of cementation structure. The model introduced in this paper is capable of describing the behavior of clay in reconstituted states (i.e., identical to that of the Modified Cam Clay Model), the behavior of structured clay without cementation (i.e., identical to that the original Structured Cam Clay Model), the behavior of structured clays with cementation structure which can be removed by shearing to large deformation, and the behavior of structured clays with cementation structure part of which cannot be removed by shearing to large deformation. The model is employed to simulate both the compression behaviour and shearing behaviour of two types of cemented clay, and the model is evaluated.

In this paper, the behavior of the parent clay without cement in reconstituted state is used as reference state to measure the influence of cementation structure. Consequently, the influence of soil structure including that of cementation effect is measured as the difference in the mechanical response between a cemented soil and the parent soil without cement in reconstituted states. This assumption of reference state behavior enables the prediction of the behavior of soil with various cement contents without the requirement of tests on the soil with individual cement content in reconstituted states, providing that model parameters for cementation structure are properly determined. However, it should be pointed out that the behavior of the parent clay without cement in reconstituted states and that of the clay with cement in reconstituted states are expected to be different. Some experimental data on the compression behaviour can be found in a paper by Xiao and Lee (2014).

2. STRUCTURED CAM CLAY MODEL WITH CEMENTATION EFFECT

In this section, a Structured Cam Clay model for cemented clay is introduced based on the theoretical framework of the Structured Cam Clay Model proposed by Liu and Carter (Liu and Carter, 2002; Carter and Liu, 2005) and the work by Horpibulsuk et al (2010). Modifications are made to improve the performance of the model especially with strong cementation. The definitions of stress and strain parameters are the same as those given by Liu and Carter (2002).

The Structured Cam Clay model is formulated for providing a model suitable for solution of boundary value problems encountered in geotechnical engineering practice. It is therefore necessary to keep the model relatively simple. Special care is taken so that the effect of cementation on mechanical behaviour is incorporated into the model straightforwardly and parameters describing the influence of cementation structure can be determined conveniently for numerical simulation of geotechnical problems.

2.1 Operative effective mean stress for cohesive soils

Based on the analysis of experimental data (e.g., Coop and Atkinson, 1993; Gens and Nova, 1993; Horpibulsuk, 2001; and Kasama et al., 2000; Rios et al., 2014), it is concluded that the behaviour of cemented soils principally follows rules of reconstituted soils. Both the peak strength and the final failure strength at large shear deformation follow the Mohr-Coulomb failure criterion. Soil behaviour can be divided into small deformation and large deformation by a yielding boundary. The surface experiences enlargement mainly with volumetric deformation. Dilatancy behaviour is clearly seen at low stress level or at high density. However, the behaviour of cemented soil is significantly different from that of the parent soil not only with improved strength and stiffness, but also with some basic features such as ability to carry tensile stresses, and softening behaviour for soil in normally consolidated states.

Unlike reconstituted soils, cemented soil has the capacity to withstand tensile stress. Thus, the boundary of the stress states applicable to cemented soil, or the yield surface, covers some of the part with negative mean effective stress. A state boundary surface for Ariake clays with different cement contents is shown in Figure 1 (Suebsuk et al., 2010). The stress parameters are normalised by (p'_0+C). p'_0 is the pre-shear effective stress or the yield stress in isotropic compression condition. C is the cementation strength. This normalised curve is the conventional state boundary surface (Atkinson and Bransby, 1978) where the structure strength is zero in case of destructured clay. The state boundary surface and the modified effective stress concepts are fundamental to the development of the SCC model with cementation effect.

Therefore, an operative mean stress parameter for cohesive soil is proposed as follows:

$$\bar{p}' = p' + C/\mathrm{M} \tag{1}$$

where p' is the mean effective stress, M is the critical state shear stress ratio of the cemented clay, and C is a parameter related to the shear strength contributed by cementation. Consequently, in the formulation of the SCC model for cemented clay, the operative shear stress ratio is modified as:

$$\bar{\eta} = \frac{q}{\bar{p}'} \tag{2}$$

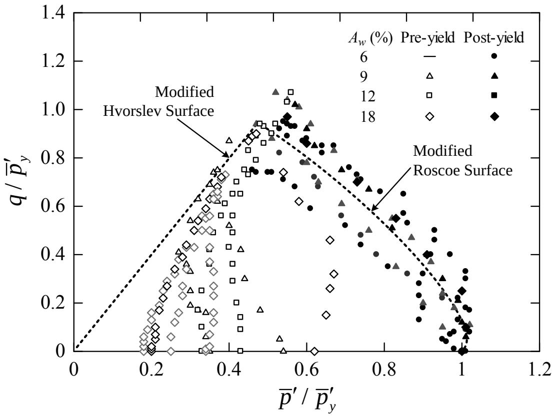


Figure 1 Normalized effective stress paths for undrained tests on cemented clays

2.2 Cemented structure

A comparison of the compression behaviour of clay in reconstituted states, naturally structured states and cemented states is shown in Figure 2 (A_w stands for cement content by weight. Test data were from Lorenzo and Bergado, 2004). For artificially cemented clays the compression behaviour is usually not asymptotic to that of the parent material in a reconstituted state. It is seen that part of the cementation structure does not disappear. The type of the induced cementation structure may be classified as meta-stable (Cotecchia and Chandler, 2000). The same conclusion is also seen for the final failure states (e.g., Horpibulsuk et al., 2004b). Therefore, it is proposed that unlike natural soft clay, the structure formed by artificial cementation may not be removed completed by pure loading.

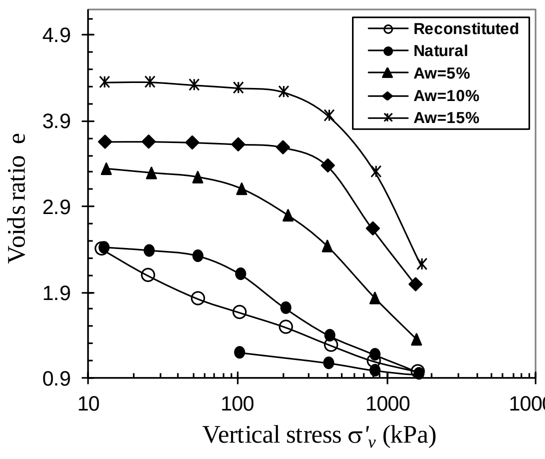


Figure 2 One dimensional compression behaviour of soft Bangkok clay with various structures (Lorenzo and Bergado, 2004)

The compression equation proposed by Liu and Carter (1999, 2000b) is modified to allow for the part of cementation structure that cannot be removed by loading. The following compression equation for cemented clay is proposed (Horpibulsuk et al., 2010):

$$e = e^* + (\Delta e_i - c)\left(\frac{p'_{y,i}}{p'}\right)^b + c \tag{3}$$

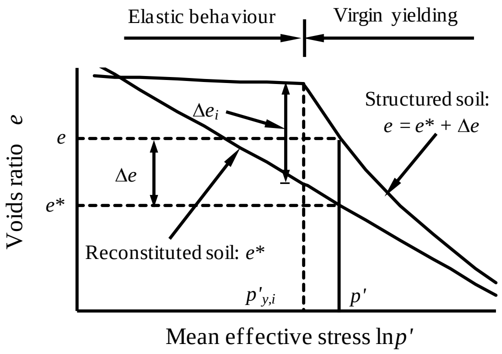
where e represents the voids ratio for a cemented clay, e^* is the voids ratio of the reconstituted clay at same stress state with the same yield surface. Based on Cam Clay model (Schofield and Wroth, 1968), the voids ratio of a clay during an isotropic compression test is dependent on the current mean effective stress p' and the historical maximum mean effective stress, which is also equal to the size of the yield surface p'_o, as

$$e^* = e^*_{IC} - (\lambda^* - \kappa^*)\ln p'_o - \kappa^* \ln p' \tag{4}$$

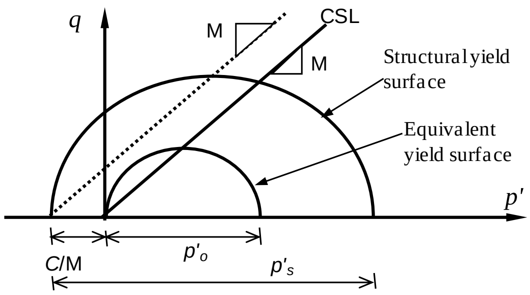
where λ^* and κ^* represent the compression and the swelling indices of reconstituted clay, respectively, and e^*_{IC} is the voids ratio of the reconstituted isotropic compression line at $p' = 1$ kPa.

Parameter $p'_{y,i}$ is the mean effective stress at which virgin yielding of the structured soil begins. b and c are soil parameters describing the additional voids ratio sustained by cementation. Δe is the additional voids ratio sustained by soil structure (i.e., the difference in voids ratio between a structured soil and the soil of the same mineralogy in reconstituted state under the same stress condition), and Δe_i is the value of the additional voids ratio at the start of virgin yielding (Figure 3a). Parameter c is the part of cementation structure that cannot be removed by loading, defined by the following equation,

$$c = \lim_{p' \to \infty} \Delta e \tag{5}$$



(a) Compression behaviour of structural soils



(b) Structural and equivalent yield surfaces

Figure 3 Material idealisation for Structured Cam Clay model with cementation effect

2.3 Material idealisation

In the SCC model for cementation effect, cemented clay is idealised as an isotropic material with elastic and virgin yielding behaviour. The yield surface varies isotropically with plastic volumetric deformation. Soil behaviour is assumed to be elastic for any stress excursion inside the current structural yield surface. Virgin yielding occurs for a stress variation originating on the structural yield surface and causing it to change. During virgin yielding, the current stress stays on the structural yield surface. The removal of cementation is assumed to occur mainly during the rearrangement of soil particles to form a critical state of deformation and therefore is represented only after soil reaches its peak strength.

The idealisation of the mechanical behaviour of cemented clays is illustrated in Figure 3. The reconstituted compression line is taken as a reference for describing the cemented compression curve as successfully done by Liu and Carter (2003) and Liu et al. (2003). In this figure, $p'_{y,i}$ is the mean effective stress at which virgin yielding of the cemented soil begins, and Δe, the additional voids ratio, is the difference in voids ratio between the cemented clay and the reconstituted clay at the same stress state.

The Isotropic Compression Line (ICL) forms the boundary of the soil states in the $e - \ln p'$ space. The virgin compression behaviour of a cemented soil is related to that of its parent clay in reconstituted states by the following equation:

$$e = e^* + \Delta e \tag{6}$$

Considering the consistency with the introduction of operative mean effective stress parameter $\bar{p}'$, the following compression equation for cemented clay is proposed:

$$e = e^* + (\Delta e_i - c)\left(\frac{p'_{y,i} + C/\mathrm{M}}{\bar{p}'}\right)^b + c \tag{7}$$

Following the tradition of the Modified Cam Clay model, the yield surface of a cemented clay in p'-q space is also assumed to be elliptical and is described as (3b),

$$f = q^2 - \mathrm{M}\bar{p}'(p'_s - \bar{p}') = 0 \tag{8}$$

where p'_s is the size of the yield surface, and is numerically equal to the length of the diameter of the elliptical yield surface along p' axis. For cemented soils, the yield surface is effectively shifted to the left along the p' axis by a distance of C/M.

The size of the initial yield surface, $p'_{s,i}$ is thus linked to the initial mean effective yield stress $p'_{y,i}$ by the following equation (3b),

$$p'_{s,i} = p'_{y,i} + C/\mathrm{M} \tag{9}$$

2.4 Elastic behaviour

For stress excursions within the current virgin yielding boundary, only elastic deformation occurs. For simplicity, elastic deformation of cemented clay is assumed to be described by Hooke's law, i.e.,

$$d\varepsilon_v^e = \frac{3(1-2\nu^*)}{E} dp' \tag{10}$$

$$d\varepsilon_d^e = \frac{2(1+\nu^*)}{3}\frac{dq}{E} \tag{11}$$

where ν^* is the Poisson's ratio and E is the Young's modulus. E, ν^*, $\bar{p}'$, and the elastic swelling index κ are related by

$$E = \frac{3(1-2\nu^*)(1+e)}{\kappa}\bar{p}' \tag{12}$$

It was observed experimentally that the elastic deformation stiffness generally increases with cementation (e.g., Huang and Airey, 1998; Horpibulsuk et al., 2004a). As a result, the elastic deformation stiffness is linked to the operative mean effective stress, which increases with cementation.

2.5 Virgin yielding behaviour

For stress states on the yield surface and with $dp'_s > 0$, virgin yielding occurs. The deformation of the soil is made up of elastic part and plastic part. The elastic part of soil deformation can be described by Eqs. (9) and (10). The plastic volumetric strain increment for the original SCC model was derived from the assumption that both hardening and destructuring of clays are dependent on volumetric deformation, with a consideration of the destructuring associated with shearing. A detailed study on constitutive modelling of geomaterials based on plastic volumetric hardening and destructuring can be found in papers by Liu and Carter (2002) and Liu et al. (2011). The plastic volumetric strain increment for the SCC model was obtained as

$$d\varepsilon_v = d\varepsilon_v^e + \left\{(\lambda^* - \kappa^*) + b\left[\langle \Delta e - c\rangle + \frac{\gamma\eta\Delta e}{\mathrm{M}-\eta}\right]\right\}\frac{dp'_s}{(1+e)\,p'_s} \qquad (13)$$

where γ is a soil parameter describing the destructuring associated with shearing, and

$$\langle \Delta e - c\rangle = \begin{cases} \Delta e - c & \text{if } \{|\Delta e| - |c|\} \geq 0 \\ 0 & \text{if } \{|\Delta e| - |c|\} < 0 \end{cases} \qquad (14)$$

$|x|$ represents the absolute value of the quantity x.

In this model formulation, cemented soil behavior is described in terms of the operative mean effective stress (i.e., Eq. 1), shear stress, and the operative shear stress ratio (i.e., Eq. 2). The elastic swelling index κ and the final failure shear strength M for cemented soil are allowed to be different for the parent soil in reconstituted states. Moreover, unlike that of natural soft clay the final failure state for a cemented soil and that for the same soil in reconstituted state may not be the same for the same test. The difference in voids ratio between cemented and reconstituted samples at large effective stress is represented by c. Considering these features of cemented clay, Eq. (13) is modified as follows:

$$d\varepsilon_v = d\varepsilon_v^e + \left\{(\lambda^* - \kappa) + b\langle \Delta e - c\rangle\left[1 + \frac{\gamma\bar{\eta}}{\mathrm{M}-\bar{\eta}}\right]\right\}\frac{dp'_s}{(1+e)\,p'_s} \qquad (15)$$

The deviatoric strain increment is worked out based on the plastic flow rule. In the formulation of the flow rule suitable for cemented clays, the following conditions have been considered:

(1) The flow rule should be identical to that of the MCC model if the soil has no cementation or if the cementation structure has been completely removed, i.e., $\Delta e = 0$.

(2) A final failure state has yet to be reached if all the destructible parts of cementation structure have not been completely removed, i.e., $(\Delta e - c) > 0$.

(3) Soil reaches the critical state, i.e., $\dfrac{d\varepsilon_d^p}{d\varepsilon_v^p} = \infty$ when $\bar{\eta} = \eta = \mathrm{M}$, and $(\Delta e - c) = c$.

$$\frac{d\varepsilon_d^p}{d\varepsilon_v^p} = \frac{2\bar{\eta}}{\left|\mathrm{M}^2 - \bar{\eta}^2\right| + \omega\left|1 - \sqrt{p'_o/p'_s}\right|} \qquad (16)$$

where ω is a model parameter, and p'_o is the size of the equivalent yield surface.

The equivalent yield surface is defined to take into consideration of that part of the additional voids ratio c, which does not disappear by loading. Thus, based on the intrinsic isotropic compression equation and considering the influence of the additional voids ratio c, the following equation for p'_o is found:

$$p'_o = \frac{e^{\left(\frac{e^*_{IC} - e + c}{\lambda^* - \kappa}\right)}}{p'^{\left(\frac{\kappa}{\lambda^* - \kappa}\right)}} \qquad (17)$$

When a stress state reaches the yield surface and with $\bar{\eta} > \mathrm{M}$, softening occurs. During the softening process, the yield surface shrinks, i.e., $dp'_s < 0$. The volumetric deformation of soil is described by the same equation as that for virgin yielding, i.e., Eq.(15). However, a modification to the deviatoric strain increment is made to ensure that the deviatoric deformation contributed by destructuring is always positive, i.e.,

$$d\varepsilon_d^p = -\frac{2\bar{\eta}}{\left|\mathrm{M}^2 - \bar{\eta}^2\right| + \omega\left|1 - \sqrt{p'_o/p'_s}\right|} \times \left[(\lambda^* - \kappa) - \frac{\gamma\bar{\eta}b\langle \Delta e - c\rangle}{\mathrm{M}-\bar{\eta}}\right]\frac{dp'_s}{(1+e)\,p'_s} \qquad (18)$$

2.6 Removal of cementation structure

The behaviour of cemented clay is fundamentally different from that of reconstituted clay. The soil usually has two strengths: the peak strength and the final failure strength at shear large deformation, irrespective of the initial stress state or the value of OCR. This feature is very pronounced in undrained shearing tests (e.g., Huang and Airey, 1998; and Horpibulsuk, 2001; Lee et al., 2005). This feature is attributed to the breakdown of cementation structure.

The peak strength of cemented clay is made up of two parts: the part contributed by cementation and the strength of the parent clay at corresponding stress and strain conditions. After peak strength, soil deformation is generally much larger than that for pre-peak strength. For artificially strongly cemented soil, the breakdown of cementation mainly takes place at this stage of deformation. The strength of soil after the removal of cementation is inevitably lower than that at the peak. Thus, two strengths are usually observed.

Because soil deformation in the post-peak stage is generally much larger than that for pre-peak stage, it is assumed that the breakdown of cementation only occurs during the post peak stage. This assumption is an approximation and for the purpose of simplicity. The following assumptions are made in order to work out the stress and strain relationship during this process.

(1) The operative effective stress state stays on the line defined by M but may travel along the line either upwards or downwards, depending on hardening or softening, respectively. Therefore,

$$\frac{q}{\bar{p}'} = \mathrm{M} \qquad (19)$$

(2) Based on trial and error, the function for the breakdown of soil-cementation structure is dependent on the size change of the structural yield surface

$$dC = -\left(\frac{C}{C_{in}}\right)\frac{|dp'_s|}{\sqrt{\left(\frac{q}{p'} - \mathrm{M}\right)}} \qquad (20)$$

where C_{in} is the value of the initial cementation strength. At the end of the process, soil reaches the final critical failure state with $C = 0$ and $\Delta e = c$.

3. MODEL PARAMETERS AND MODEL APPLICABILITY

3.1 Model parameters

Eleven parameters define the Structured Cam Clay model, and they are e^*_{IC}, λ^*, ν^*, M, κ, b, c, γ, ω, $p'_{y,i}$, and C as listed in Table 1. The first three parameters, denoted by the symbol *, are intrinsic soil properties. They are independent of soil structure. These three intrinsic parameters are the same as those adopted in the Modified Cam Clay model (e.g., Muir Wood, 1990). Parameters b, c, γ, ω, $p'_{y,i}$ and C are strongly dependent on soil cementation structure. Their values are dependent on the magnitudes of cementation. The physical meanings of these parameters are basically the same as those given in the work by Liu and Crater (2000b), Carter and Liu (2005), and Horpibulsuk et al. (2010). The identification of all the model parameters can be found in the above references.

Table 1 Properties of Structured Cam Clay

Symbol	Description
	Intrinsic soil properties
e^*_{IC}	voids ratio at the p' = 1 on the ICL in e-lnp' space
λ^*	gradient of the normal compression line in e-lnp' space
ν^*	Poisson's ratio
	Parameters defining soil structure
M	critical state stress ratio for cemented clay
κ	gradient of the unloading and reloading line in e-lnp' space
b	destructuring index
Δe_i	additional voids ratio sustained by soil structure at the start of virgin yielding
c	additional voids ratio sustained by soil structure at very large confining pressures
$p'_{y,i}$	initial (structural) yield stress for isotropic compression
γ	parameter defining the plastic potential
ω	flow rule parameter
C	cementation strength

Based on an analysis of experimental data (e.g., Horpibulsuk, 2001; Lee et al., 2005; Consoli et al., 2012), M are κ are also dependent on cementation. In this formulation, these two parameters are allowed to vary with cement content.

Some empirical equations, which may be used for estimating the value of model parameters, are given here.

For M, its value may be assumed as a constant for cemented clay, which is usually greater than M*, the value of the parent clay. Consequently, it is suggested that the critical state shear stress ratio M is to be determined from tests on cemented clay.

For κ, it increases with cement content or cementation strength C. Based on some primary study by Liu (2013), elastic swelling index κ, identifiable from compression tests is proposed as follows

$$\kappa = \kappa_o\left(1 - a_1\left(A_w\right)^{a_2}\right) \tag{21}$$

κ_o is the elastic swelling index for the uncemented soil. Parameter a_1 and a_2 are two material parameters. α = 1 can be assumed if there is not enough data for its identification.

Some empirical equations may be used for SCC model parameters when there are not enough data for their accurate identification. These equations are obtained mainly based on model performance for describing soil behavior under conventional triaxial shearing tests. Based on the work by Horpibulsuk et al. (2010), parameters C and $p'_{y,i}$ are linked directly to the unconfined compressive strength of the cemented clay, q_u, which is routinely measured in geotechnical engineering practice.

$$C = \frac{1}{2}q_u \tag{22}$$

$$p'_{y,i} = q_u \tag{23}$$

Both parameters Δe_i and c vary with cementation structure of the soil, usually represented by the cement content. It is proposed that the ratio c over the initial additional voids ratio Δe_i (without any destructuring) is constant for a given soil and cement material. Thus, parameter c can be estimated from the value of the initial voids ratio sustained by cementation structure. It is given by

$$\frac{c}{\Delta e_i} = \text{constant} \tag{24}$$

3.2 Model applicability

The model presented in this paper is suitable for clay with strong cementation structure, and it is also required that the compression behaviour of the parent clay can be described as linear in the the e – lnp' space. The cementation structure is assumed to break down after soil reaches its peak strength. For soil with weak cementation that is basically removed during plastic deformation before the peak strength is reached, it is suggested to model this type of soil as naturally structured clay without cementation.

This model is identical to SCC model when $C = 0$ (no cementation) and $c = 0$ (the behaviour of the structured soil is asymptotic to that of the reconstituted soil as stress increases) are assumed.

This model is identical to MCC model when $C = 0$ (no cementation) and $\Delta e = 0$ (the behaviour of the structured soil is asymptotic to that of the reconstituted soil as stress increases) are assumed.

The proposed model is ready for implementation into numerical analysis of boundary value problems. As can be seen from constitutive equations (9), (10), (14) and (15), the D matrix for the incremental stress and strain relationship for the cemented SCC model possess a form essential the same as that of the MCC model. The extension of a two dimensional constitutive model into a general stress and strain model and the application of SCC model for FEM analysis can be seen in papers such as Khalili et al. (2008) and Liyanapathirana et al. (2005).

4. PERFORMANCE OF THE STRUCTURED CAM CLAY MODEL WITH CEMENTATION EFFECT

In this section, the Structured Cam Clay Model with cementation effect is used to simulate both the compression and shearing behaviour of cemented clays. For compression behaviour, Bangkok clay under three different states is studied. For shearing behaviour cemented Singapore marine clay and cemented Chennai marine clay are studied.

4.1 Compression behaviour of Bangkok clay with various structures

Based on the introduced framework for quantifying the influence of soil structures on compression behaviour, the compressibility of soils is investigated in different states, such as reconstituted state, natural state, and cemented state. Five one dimensional compression tests are studied on soft Bangkok clay in reconstituted states, natural states, and artificially cemented states. The test data are reported by Lorenzo and Bergado (2004). The values of the parameters determined are listed in Table 2. Parameter e^*_N is the value of the voids ratio at σ'_v = 1 kPa for a soil in reconstituted states in one dimensional compression test. Some studies on parameters describing soil compression behaviour in isotropic compression tests and one dimensional compression tests can be found in the work such as Liu and Carter (1999).

For simulating compression behaviour, only seven parameters are needed (*see* Table 2). Parameter e^*_N is for one dimensional compression line. Comparisons between the theoretical equations and the experimental data are shown in Figure 4.

All three states of the Bangkok clay, reconstituted and natural structured and artificially cemented, are investigated. It is seen that the compression behaviour of the soil in the three states can be described successfully with the behaviour of the same soil in reconstituted soils as a base. Consequently, the focus of structured soil behaviour will be the reduction of the additional voids ratio associated with destructuring, which is modeled with structure-dependent soil parameters.

Table 2 Soil parameters for Bangkok clay

Tests	Intrinsic soil parameters			Structural soil parameters			
Reconstituted	e^*_N	λ^*	κ^*	Δe_i	$\sigma'_{v,yi}$	b	c
Natural	3.0	0.28	0.05				
A_w = 5%				0.53	80	1	0
A_w = 10%				1.6	180	0.7	0.1
A_w = 15%				2.15	370	0.7	0.4

For natural structured clays particularly soft clay, the behaviour of the structured soil is asymptotic to that of the reconstituted soil. However, for strongly cemented soil, it is seen that part of the voids ratio associated with strong cementation structure does not diminish with increasing stress level and the rate of destructuring for soils with strong or stable structure is generally lower than that of soils with weak structure. This is consistent with experimental observation reported, i.e., Huang and Airey (1998), Cotecchia and Chandler (2000), and Horpibulsuk (2001).

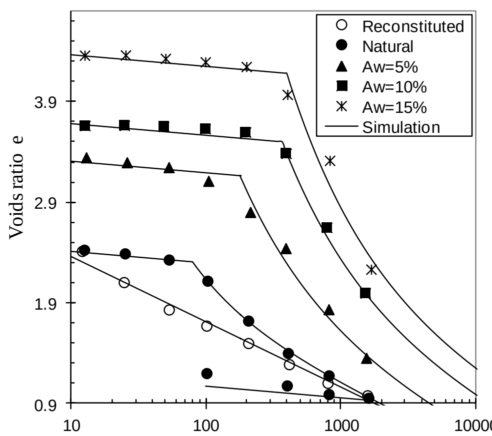


Figure 4 Compression behaviour of Bangkok clay in reconstituted, naturally structured, and artificially cemented states (Test data after Lorenzo and Bergado, 2004)

4.2 Shearing behaviour of cemented Singapore Marine clay

The performance of the proposed model is examined by simulating the behaviour of cemented Singapore Marine clay. Undrained shear behaviour of the cemented clay in conventional triaxial tests with different confining stresses was considered. Four tests with 10% cement content are simulated. The tests are reported by Kamruzzaman et al. (2009). Values of model parameters used for model simulations are listed in Table 3.

Parameters e^*_{IC}, λ^*, κ, Δe_i, $p'_{y,i}$, b, and c are determined from the results of one dimensional compression tests on the cemented clay and the reconstituted clay. Comparisons between the test data and the model simulation for the compression tests are shown in Figure 5. An equation for estimation the relationship between and e^*_{IC} and e^*_N can be found in a paper by Liu and Carter (2002). It is seen that the behaviour of the clay during unloading and initial loading can be assumed as linear elastic in the $e - \ln p'$ space. However, the stiffness of the soil increases, and the value of parameter κ changes from 0.09 for uncemented state to 0.028 for the soil with 10% cement content. It is also found that parameter M (critical state shear stress ratio) for the cemented clay (M = 1.5) is significant higher than that of the parent clay in reconstituted state (M* = 0.9).

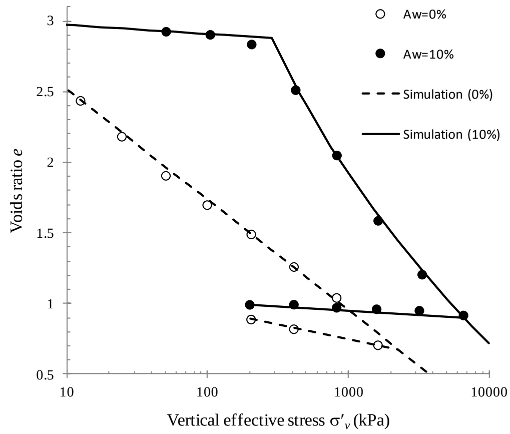
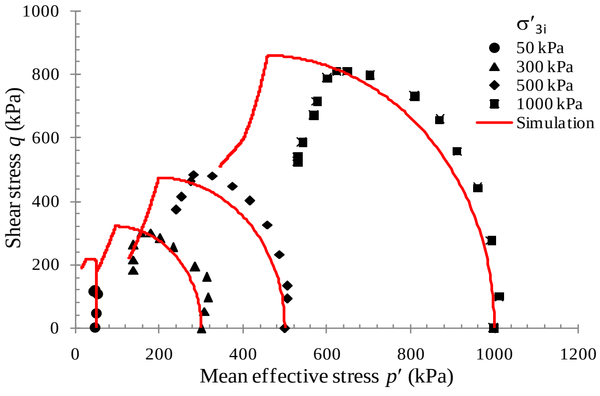


Figure 5 Compression behaviour of uncemented and cemented Singapore Marine clay (Test data after Kamruzzaman et al., 2009)

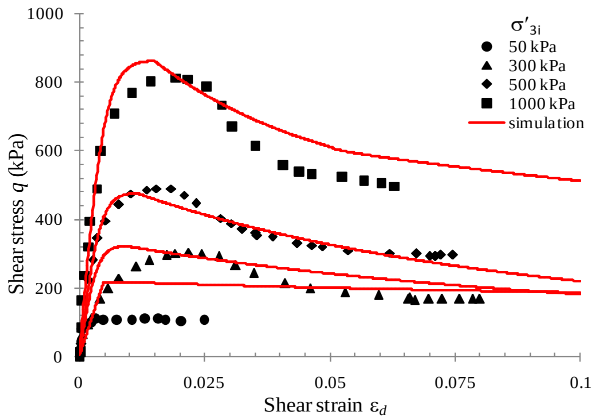
Table 3 Values of model parameters for cemented Singapore Marine clay and Chennai marine clay

Symbol	Soil Type	
	Chennai marine clay	**Singapore Marine clay**
λ^*	0.34	0.27
κ	0.028	0.03
M	1.5	1.4
e_{IC}^*	3.3	2.94
ν*	0.3	0.3
B	0.5	0.9
Δe_i	1.5	0.3
c	0.36	0
$p'_{y,i}$ (kPa)	285	320
γ	0.38	0.06
ω	0.4	1
C (kPa)	180	70

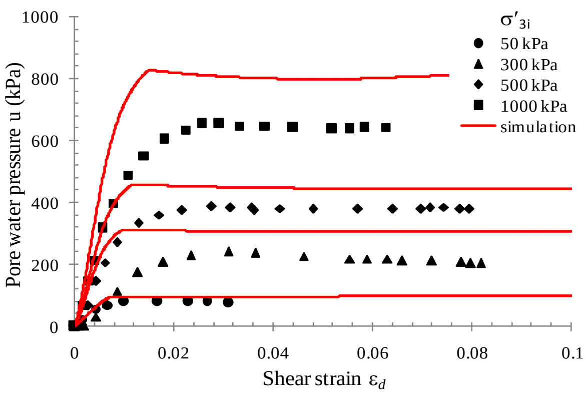
Comparisons of the model simulations and experimental data for Singapore Marine clay are shown in Figure 6. The effective stress paths, the shear stress and strain relationship, and the development of the pore pressure are presented. The size of the yield surface for the cementation structure identified from compression tests (Figure 5) is 285 kPa. Therefore, soil behaves as over-consolidated material for the test with σ'_{3i} = 50 kPa. For other tests, soil behaves as virgin yielding material. Overall, basic features of the shear behaviour of the cemented clay have been simulated reasonably well. Due to the influence of cementation, both the peak strength and final failure strength of the soil have been improved significantly. Cemented soil gives stiffer response due to the enlargement of the yield surface as well as the reduction in the value of κ. Softening behaviour (instability behaviour) after peak is predicted for all the soil specimens, irrespective of the initial confining stresses. This is attributed to the breakdown of cementation.



(a) Effective stress paths



(b) Shear stress and strain relationship



(c) Pore water pressure development

Figure 6 Undrained behaviour of Singapore Marine clay with 10% cement content (Test data after Kamruzzaman et al., 2009)

4.3 Shearing behaviour of cemented Chennai Marine clay

The proposed model is employed to simulate the behaviour of cemented Chennai marine clay. Undrained shear behaviour tests of the cemented clay in conventional triaxial tests with different confining stresses were reported by Pillai et al. (2013). Three tests with 5% cement content are simulated. Values of model parameters used for model simulations are listed in Table 3.

Parameters e^*_{IC}, λ^*, κ, Δe_i, b, and c are determined from the results of a one-dimensional compression tests on the cemented clay. Comparisons between the test data and the model simulation for the compression tests are shown in Figure 7. For the compression test, the yielding vertical effective stress is found to be 500 kPa. However, some variation in soil specimens is found. Following the suggestion by Pillai et al. (2013), the value for the initial yield surface for the triaxial compression specimens, i.e., $p'_{y,i}$, is adopted to be 320 kPa. The values for M and C are obtained from a study of the strength of soil.

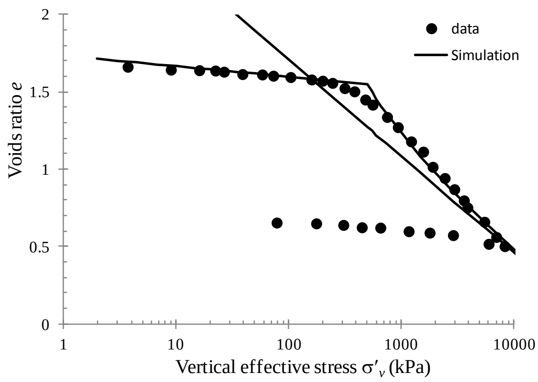
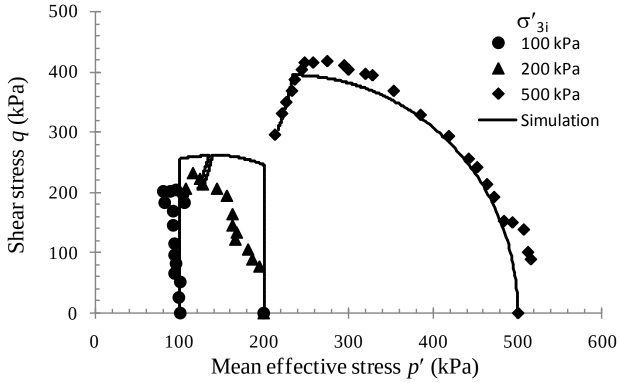
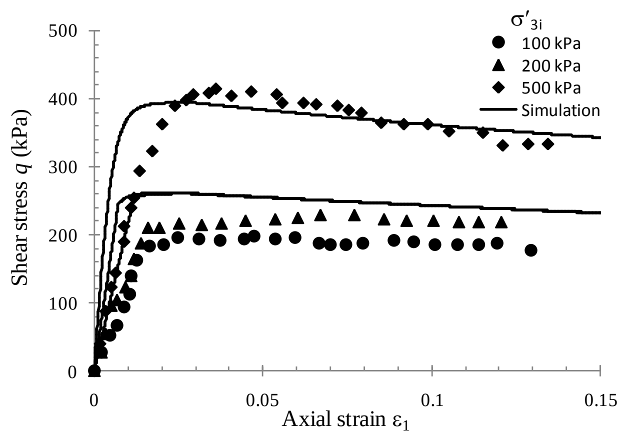


Figure 7 Compression behaviour of Chennai Marine clay with 5% cement content (Test data after Pillai et al., 2013)

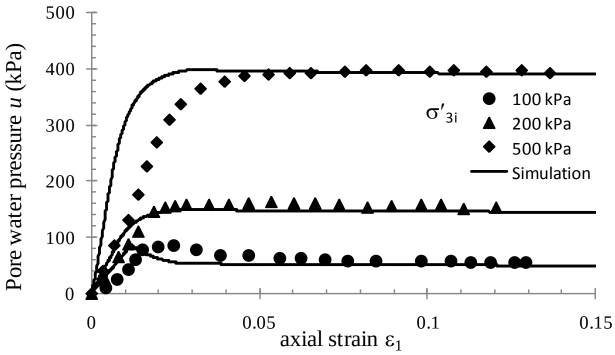
Comparisons of the model simulations and experimental data for the cemented marine clay are shown in Figure 8. Overall, basic features of the shear behaviour of the cemented clay have been represented reasonably well. The reduction of the soil strength after peak is attributed to two factors, the removal of soil structure Δe_i and the breakdown of cementation strength C. For the test with σ'_{3i} = 500 kPa, the reduction of soil strength associated with soil structure Δe_i is relatively low because part of the soil structure has been removed during the isotropic loading to the initial stress state (p' = 500 kPa, q = 0 kPa), because the soil starts yielding when the initial structural yield surface is reached.



(a) Effective stress paths



(b) Shear stress and axial strain relationship



(c) Pore water pressure development

Figure 8 Undrained behaviour of Chennai Marine clay with 5% cement content (Test data after Pillai et al., 2013)

5. CONCLUSIONS

In this paper, a constitutive model for clays with strong cementation is presented. The model is formulated based on the theoretical framework of the Structured Cam Clay model. The basic concept for modelling the influence of cementation effect on soil behavior is the introduction of an operative mean effective stress parameter, which takes into consideration the influence of cohesion/cementation on the strength and the deformation of the soil. For simplicity, the removal of cementation is assumed to take place during the process of the rearrangement of soil particles to form the final critical state of deformation, where large plastic deformation occurs.

The model has been used to simulate both the compression and shearing behaviour of cemented clays. It has been demonstrated that this simple predictive model has captured reasonably well main features of the behaviour of cemented clays. The improvement on soil peak strength and final failure strength and on the stiffness is described well. Softening behaviour (instability behaviour) after peak is predicted for cemented soft soil, irrespective of the OCR values and is consistent with experimental observation. This is attributed to the breakdown of cementation.

6. ACKNOWLEDGEMENTS

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