Journal of Rock Mechanics and Geotechnical Engineering 9 (2017) 1094–1103

Contents lists available at ScienceDirect

Journal of Rock Mechanics and Geotechnical Engineering

journal homepage: www.rockgeotech.org

Full Length Article

A critical state based thermo-elasto-plastic constitutive model for structured clays

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ARTICLE INFO

Article history:
Received 14 March 2017
Received in revised form
9 September 2017
Accepted 20 September 2017
Available online 8 November 2017

Keywords:
Thermo-mechanical constitutive model
Structured clays
Cam Clay
Normal consolidation line
Boom Clay

ABSTRACT

In this paper, a critical state based thermo-elasto-plastic constitutive model is developed for destructured, naturally structured and artificially structured saturated clays. The model is an extension of the previously developed thermo-mechanical model by the authors for saturated clays, considering the effects of structure on the mechanical behaviors of the soil. It is based on change in the position of normal consolidation line (NCL) in a compression plane ($e - \ln p'$) due to the soil's structure and variation of temperature. The present model is able to simulate the mechanical behavior of structured saturated clays in a triaxial plane at elevated temperatures lower than the boiling point of water. An attempt has been made to use the lowest possible number of parameters compared with that of Cam–Clay model and to ensure that these new parameters have clear physical interpretations. The sufficiency of the model was verified by the test results on artificially and naturally structured soils using thermal triaxial tests.

1. Introduction

Due to the rapid industrial development in recent decades, evaluation of the effect of thermal gradients on the mechanical behavior of soils has been a challenging issue. Disposal of radioactive nuclear wastes, temperature increase around high voltage cables, geothermal energy storage, geothermal structures, oil extraction and the use of heating for an increase in soil permeability around drainage systems and a decrease of soil texture disturbance on permeability are examples to define the necessity of studying the soil behavior at elevated temperatures.

Variations of temperature affect the physical and mechanical properties of soil microscopically and macroscopically. These changes also affect properties such as volume, shear strength, soil stability and pore water pressure. In this circumstance, ability to predict the engineering behavior of soils is of critical importance; in this regard, development of a constitutive model that is able to predict soil response to temperature change is needed.

Over the past decades, a number of studies on the temperature-dependent clay behavior under increased temperature have been conducted. These observations proved that temperature changes both the physical and mechanical characteristics of soil. For this purpose, various thermo-mechanical models have been developed which are able to represent the main characteristics of clays under thermal gradients (Hueckel and Borsetto, 1990; Robinet et al., 1996; Thomas et al., 1996; Cui et al., 2000; Hamidi and Khazaei, 2010; Yao and Zhou, 2013; Xiong et al., 2014, 2017; Bellia et al., 2015; Hamidi et al., 2015; Tourchi and Hamidi, 2015; Hong et al., 2016; Kurz et al., 2016; Hamidi and Tourchi, 2017).

For normally consolidated clays, the contraction of volume change due to thermal loading is irreversible; however, these changes are of reversible dilation for overconsolidated clays (Baldi et al., 1988; Cui et al., 2000; Abuel-Naga et al., 2007). A number of studies show increase of drained and undrained shear strengths of the soil by elevated temperature (Kuntiwattanakul et al., 1995; Graham et al., 2001; Cekerevac and Laloui, 2004; Abuel-Naga et al., 2007). Other researchers showed strength reduction by an increase of temperature (Houston et al., 1985; Hueckel and Baldi, 1990). Also, some researchers witnessed independence of soil strength to the thermal situation (Burghignoli et al., 1992).

Hueckel and Borsetto (1990) studied the plastic behaviors of clays under isothermal condition. They found that the elastic yield surface is temperature-dependent and the dependency appears as a decrease in yield stress by an increase in temperature. Robinet et al. (1996) used modified Cam–Clay model and developed a

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Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.

https://doi.org/10.1016/j.jrmge.2017.09.002

thermo-elasto-plastic model for saturated clays with two separate yield mechanisms. The new feature of the model in comparison to previous ones was the introduction of permanent thermal strains using two yield surfaces for the prediction of both thermal and mechanical softening behavior. Thomas et al. (1996) developed a thermo-hydro-mechanical model for unsaturated soils based on thermo-elasto-plasticity, and considered deformation-temperature coupling and energy balance in which moisture and heat storage were included.

Cui et al. (2000) suggested a thermal Cam Clay based model that was able to predict the thermoplastic strains at different preconsolidation ratios. They defined temperature effects on clays as an inverse relation between preconsolidation pressure and temperature.

Abuel-Naga et al. (2009) proposed a thermo-mechanical model for saturated clays under triaxial test conditions based on the model proposed by Masad et al. (1998). The first part was an isotropic thermo-elasto-plastic model with the ability to simulate thermally induced volume change for normally and overconsolidated saturated clays. The second part extends the parameters of the first part for prediction of strains under deviatoric stress condition.

Hamidi and Khazaei (2010) developed a thermo-elasto-plastic model based on modified Cam Clay concept. It was capable of predicting the mechanical behavior of saturated clays at temperatures above the ambient one under triaxial stress state condition. The main idea originated from the difference between normal consolidation line (NCL) of the soil in ambient and higher temperatures. Yao and Zhou (2013) presented a non-isothermal constitutive model with constant hardening based on a previous model developed by Yao et al. (2009) using unconfined hardening to simulate the thermo-elasto-plastic behavior of normally and overconsolidated clays. Hamidi et al. (2015) modified the original model (i.e. Hamidi and Khazaei, 2010) based on critical state theory, using a different equation for the yield surface and a temperature-dependent flow rule to improve the simulation results for saturated clays.

Xiong et al. (2014) developed a thermo-elasto-viscoplastic model for soft rocks with the ability to analyze thermo-hydro-mechanical (THM) coupling of heating tests, considering the effect of intermediate stress on the initial model of Zhang and Zhang (2009). Using the modified model, they developed a program called SOFT to simulate THM behavior of geological materials and concluded that the thermal volume change response is greatly dependent on overconsolidation ratio (OCR).

Hong et al. (2016) presented a thermo-mechanical model based on the model of Cui et al. (2000) using two yield surfaces, i.e. the current thermo-mechanical yield surface and the other one inside it which was formed based on thermal effects on yield stress. The model was able to predict the behavior of saturated clays at the non-isothermal condition. Bellia et al. (2015) proposed a thermo-mechanical model for unsaturated clays based on the effective stress concept and bounding surface plasticity, and they used suction and temperature to predict the behavior of unsaturated soils. Wang et al. (2016) also developed a non-isothermal model for soft, saturated clays based on modified Cam–Clay model which was capable of defining the thermal effects on the undrained shear strength. Tourchi and Hamidi (2015) and Hamidi and Tourchi (2017) proposed an isothermal model for unsaturated clays based on the modification of their previous model (i.e. Hamidi et al., 2015), considering Bishop's stress and suction as independent stress parameters and modifying the hardening rule and yield criterion to take into account the role of suction.

Kurz et al. (2016) proposed a semi-empirical thermo-elasto-viscoplastic model for clay considering creep behavior in compression and shear using a creep rate coefficient that varied with plasticity index and temperature but not with stress level and overconsolidation ratio.

Xiong et al. (2017) presented a unified thermo-elasto-viscoplastic model for soft rocks in the critical state framework. Two evolution equations were introduced for the shear strength and overconsolidation to take into account the influences of the confining stress and time-dependent behaviors, respectively. The model was able to describe the fundamental mechanical behavior of soft rock, such as strain softening and hardening, time-, temperature- and confinement-dependency in addition to the intermediate principal stress.

The influence of soil structure on the mechanical properties of soil has long been recognized. Here, the term "structured soil" is used to represent the arrangement and bonding of the soil constitutions. Removal of soil structure is referred to as "destructuring" and this is usually a progressive process. Generally, soils have natural structures due to the chemical, thermal, mechanical or environmental interactions and these types of soils behave differently in comparison to the reconstituted soils of similar materials (Burland, 1990; Leroueil and Vaughan, 1990; Cuccovillo and Coop, 1999). While the structure can arise from many causes, its effects follow a simple general pattern that involves stiff behavior followed by yield behavior. This yield behavior can be described in a similar way to that occurring due to overconsolidation, although it is a separate phenomenon (Leroueil and Vaughan, 1990).

Clays in their application as a buffer layer in nuclear waste disposal around high voltage cables or geothermal energy storage structures are permanently exposed to high temperatures which in combination with the surcharge pressures can result in the structured clay. Due to the differences in the mechanical behavior of non-structured clay and structured clay, the thermo-mechanical behaviors should also be different from each other. The effects of structure on the thermal characteristics of clays have not been considered in previously developed constitutive models and it seems necessary to elaborate it in a general framework for simulation of the thermo-mechanical behavior of structured clays.

Liu and Carter (2002) and Horpibulsuk et al. (2006, 2009) proposed a modified structured Cam–Clay (SCC) model that was formulated based on variations of NCL in compression plane. Also, Suebsuk et al. (2010) improved the model for naturally structured, artificially structured and destructured clays, which was considered as the modified structured Cam–Clay (MSCC) model. Yang et al. (2014) presented a model to describe the compression behavior of structured clays at large surcharge pressures using a differential function of NCL considering the effects of particle crushing. Horpibulsuk and Liu (2015) imposed cementation effects on strength and deformation of cohesive soils MSCC by defining a new mean effective stress parameter for cohesive/cemented soils. Xiao et al. (2017) developed a model to predict the behaviors of cemented clays considering the effects of cemented bonds by defining a cohesion parameter in flow rule of Cam–Clay model.

In this paper, the thermo-mechanical constitutive model presented by Hamidi et al. (2015) is revisited by a combination of thermal and structural effects on the void ratio of saturated clay and NCL. The model is able to predict the behavior of saturated structured clay in temperatures ranging from ambient one up to the boiling point of water. It is applicable to isothermal boundary value problems for naturally or artificially structured clays based on the modified Cam–Clay model and critical state soil mechanics concepts.

2. Basic concepts of the proposed model

In this section, the thermo-mechanical model proposed by Hamidi et al. (2015) for saturated clays is modified in consideration of structure effects on the soil behavior, adopting the model assumptions of Liu and Carter (2002), Horpibulsuk et al. (2006, 2009) and Suebsuk et al. (2010).

The basic changes made to the basic model are considering the structure effects as follows:

(1) Contrary to the temperature effects on compression behaviors of clays which shift down NCL in the $e-\ln p'$ plane, structuring of clays results in an increase in void ratio and shifting up of NCL in that space. Here, e is the void ratio and p' is the mean effective stress.
(2) The slopes of critical state line (CSL) in compression plane ($e-\ln p'$) is not considered as temperature-dependent. This is in accordance with the experimental results of Graham et al. (2001) and Cekerevac and Laloui (2004).
(3) The slope of NCL is higher or equal to that of CSL in compression plane. It decreases with increase in temperature and converges to the slope of CSL at the highest endurable temperature of the soil. Also, the slope of the unloading-reloading line (URL) in compression plane is considered to be temperature-dependent in the present study.
(4) At a constant temperature, the mean effective stress, $p' = (\sigma'_1 + 2\sigma'_3)/3$ of structured clay, is larger than remolded one.

2.1. Void ratio transitions

According to Cekerevac and Laloui (2004) and Abuel-Naga (2005), idealization of the isotropic compression line for thermal changes can be defined by moving toward smaller values of void ratios (i.e. line shifts toward left and bottom of the normal compression plane). Also, based on Leroueil and Vaughan (1990), structure bonds increase the void ratio of clay in a constant mean effective stress by shifting NCL towards the right and over the compression plane.

Assembling the above-mentioned assumptions, the general form of Fig. 1 can be presented. As it is demonstrated in this figure, increase in temperature has a regressive effect on void ratio and the development of structure bonds is further contributing to this effect. The value of Δe_{max} (the maximum difference between void ratios of NCL and CSL in ambient temperature) can be determined based on the differences of NCL for structured and destructured soils in this plane at the preconsolidation pressure value.

In this figure, $NCL_{(TA)}$ introduces isotropic compression line of a reconstituted soil at ambient temperature (T_A); $NCL_{(T)}$ and $NCL_{(TS)}$ also represent isotropic compression lines at elevated temperature (T) for remolded clay and structured clay, respectively.

Following the above-mentioned assumptions, a void ratio of structured clay at elevated temperature (e_{TS}) can be calculated using

$$e_{TS} = e_A - \Delta e_T + \Delta e_S \tag{1}$$

where e_A is the void ratio for non-structured clay in ambient temperature; Δe_T is the difference of void ratios in elevated and ambient temperatures, which is considered according to Eqs. (2) and (3) as the percentage of Δe_{max} (Hamidi et al., 2015). Also, Δe_S represents the difference between void ratios of structured and remolded clays (Liu and Carter, 2002) based on Eq. (4).

$$\Delta e_T = \omega \Delta e_{max} \left(\frac{p'_0}{p'_c}\right)^n \tag{2}$$

$$\omega = 1 - \exp\left[\chi\left(1 - \frac{T}{T_A}\right)\right] \tag{3}$$

$$\Delta e_S = \Delta e_i \left(\frac{p'_0}{p'}\right)^b \tag{4}$$

The parameter ω in Eq. (2) is defined according to Eq. (3) based on the ambient and elevated temperatures and the model parameter, χ. The parameter ω equals zero in ambient temperature and increases with respect to temperature elevation up to one at the peak temperature. As a result, void ratio remains constant at the ambient temperature and decreases from its primary value until it eventually reaches the maximum difference from its initial value (Δe_{max}). Also, χ in Eq. (3) is a dimensionless parameter that can be calibrated by tests and n is a model parameter.

If the slope of NCL is considered to be temperature-dependent, n takes a value greater than zero. Then, p'_0 and p'_c are the initial and preconsolidation mean effective stresses, respectively. In Eq. (4), $\triangle e_i$ is the excessive void ratio caused by structure compared to the remolded state at $p' = p'_{y,i}$, as shown in Fig. 2, where $p'_{y,i}$ is the mean effective stress at bond yield condition. The parameter b

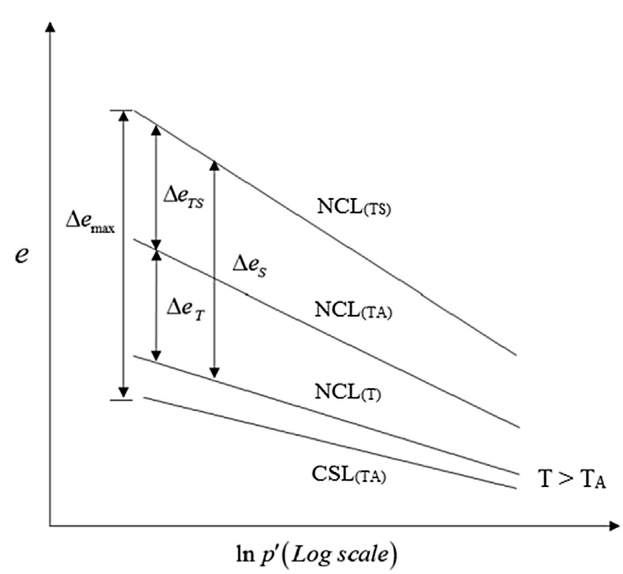

Fig. 1. Idealizing NCL transitions for structured clay at elevated temperature.

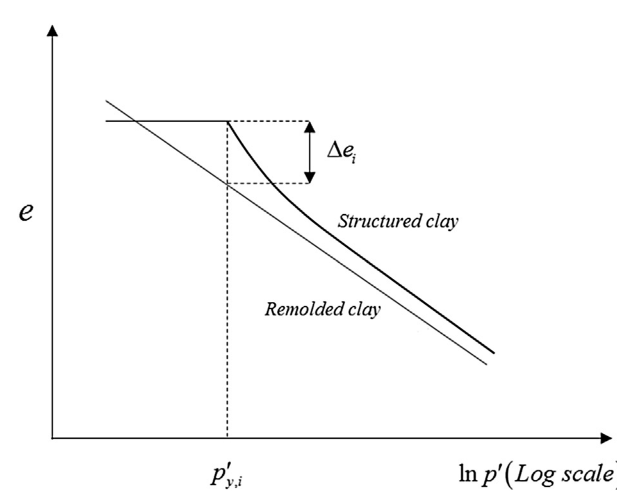

Fig. 2. Idealizing NCL transitions for structured and remolded clays.

qualifies the rate of destructuring (due to the increase in mean effective stresses or structural damage of the bonds). This parameter is assumed to vary from 0 to 30 (Liu and Carter, 2002).

According to the critical state concepts, the void ratio (e) for any mean effective stress (p') within the compression plane can be defined based on Eq. (5) and Fig. 3:

$$e = N - \lambda \ln(p'_c) + \kappa \ln\left(\frac{p'_c}{p'}\right) \tag{5}$$

where N is the specific volume at unit mean effective stress, λ is the slope of NCL and κ is the slope of unloading-reloading line.

Substituting Eqs. (2)–(5) in Eq. (1), the following relation can be derived for predicting the void ratio of structured clay at elevated temperatures:

$$e_{TS} = N - \kappa \ln p' - (\lambda - \kappa)\ln p'_{cT} - \omega\Delta e_{max}\left(\frac{p'_0}{p'_{cT}}\right)^n + \Delta e_i\left(\frac{p'_0}{p'}\right)^b \tag{6}$$

where p'_{cT} is the preconsolidation pressure in temperature T.

2.2. Mean effective stress modification

Tidfors and Sällfors (1989), Boudali et al. (1994), and Moritz (1995) related preconsolidation pressure to the temperature. Cekerevac and Laloui (2004) also proposed Eq. (7) to consider the thermal evolution of the preconsolidation pressure:

$$p'_{cT} = p'_c \exp(-\rho\Delta T) \tag{7}$$

where ΔT is the change in temperature and ρ is the material parameter. The values of ρ for mentioned tests are determined as 0.0043, 0.005, 0.0097 and 0.0065, respectively. Using Eq. (8), ρ can be calculated with a good precision:

$$\rho = \frac{3\rho^p \nu_0}{\lambda - \kappa_T} = 3\rho^p H_0 \tag{8}$$

where ρ^p is the function of thermal expansion coefficient, about 10^{-4} K^{-1} for Boom Clay (Bolzon and Schrefler, 2005) and κ_T is the temperature-dependent slope of unloading-reloading line.

Suebsuk et al. (2010) suggested that the structure influence is considered similar to the effect of an increase in the effective stress and, therefore, the yield surface, which causes an expansion in yield loci.

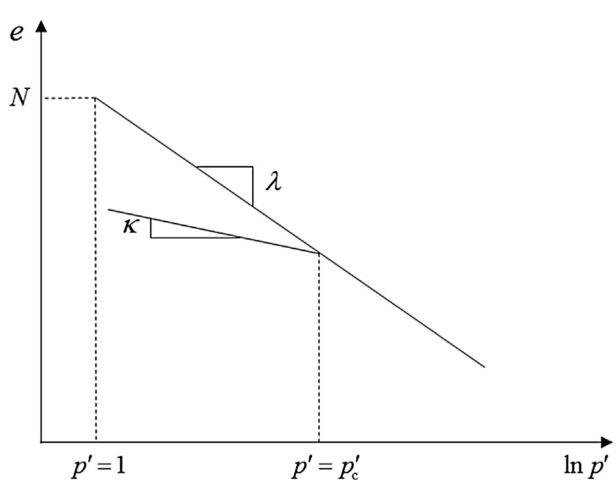

Fig. 3. Isotropic loading-unloading in compression plane.

In artificially structured clays, an increase of yield stress at higher cement contents is evident in compression and shear tests. Therefore, while two artificially structured clay samples with different amounts of cement content are subjected to the same stress condition, divergent patterns of stress–strain behavior could be observed; also, soil strength parameters of these samples could be dissimilar. The modified mean effective stress of structured clays can be defined by

$$\bar{p}' = p' + p'_b \tag{9}$$

where, $\bar{p}'$ is the modified mean effective stress for structured clay and p'_b is the increase of mean effective stress due to the structure or structural strength.

2.3. Volumetric strains

It is well known that the relation between void ratio and volumetric strain is as follows:

$$de = -d\varepsilon_v(1 + e) \tag{10}$$

where ε_v is the volumetric strain.

According to Liu and Carter (2002), elastic deformation in clay is not structural state related. On the other hand, Hamidi et al. (2015) and Tourchi and Hamidi (2015) declared that if the thermal expansion coefficient of soil is not stress-dependent, volumetric strains can be expressed as the sum of two independent thermal and mechanical components. Thus, elastic deformation of structured clay at elevated temperature can be defined by adding thermal and mechanical components:

$$d\varepsilon_v^e = d\varepsilon_{vT}^e + d\varepsilon_{vp}^e \tag{11}$$

where $d\varepsilon_v^e$ is the increment of elastic volumetric strain; $d\varepsilon_{vT}^e$ is the thermally induced elastic volumetric strain; $d\varepsilon_{vp}^e$ is elastic volumetric strain caused by stress change. Knowing thermal expansion coefficient of soil, the thermal volumetric strain increment can be calculated using:

$$d\varepsilon_{vT}^e = 3\alpha_e dT \tag{12}$$

where dT represents a temperature change increment and α_e is the thermal expansion coefficient of the solid skeleton. This coefficient is mostly related to mineral composition of soil particles; however, stress history and temperature can also be effective.

Considering negligible volumetric strain value of soil particles caused by stress change, the elastic deformations are only depending on mean effective stress state. Therefore, by differentiating Eq. (6) with respect to p', the mechanical component of elastic volumetric strain can be deduced (Eq. (15)) using Eqs. (13) and (14).

$$\frac{de}{dp'} = -(1 + e)d\varepsilon_v^e \tag{13}$$

$$\nu_0 = 1 + e \tag{14}$$

$$d\varepsilon_v^e = \frac{1}{K}dp' \tag{15}$$

$$K = K_0\left(\frac{p'}{p'_0}\right)^a \tag{16}$$

$$K_0 = \frac{\nu_0 p'_0}{\kappa_T} \tag{17}$$

where K represents the bulk modulus, K_0 is the initial value of bulk modulus, ν_0 is the initial specific volume, and a is a parameter that simulates the effect of the nonlinear relation between bulk modulus and effective stress.

As stated before, plastic volumetric strains are the results of the change in preconsolidation pressure (Eq. (16)). Differentiating Eq. (7) with respect to p'_{cT} and substituting the results in Eq. (16) can lead to Eqs. (18) and (19) for plastic volumetric strain prediction.

$$\frac{\mathrm{d}e}{\mathrm{d}p'_{cT}} = -(1+e)\mathrm{d}\varepsilon_v^p \tag{18}$$

$$\mathrm{d}\varepsilon_v^p = \frac{\lambda-\kappa_T}{1+e}\frac{\mathrm{d}p'_{cT}}{p'_{cT}} - \frac{n\Delta e_T}{1+e}\left(\frac{M}{M-\eta}\right)\frac{\mathrm{d}p'_{cT}}{p'_{cT}} + \frac{b\Delta e_S}{1+e}\left(\frac{M}{M-\eta}\right)\frac{\mathrm{d}p'_{cT}}{p'_{cT}} \tag{19}$$

where M is the slope of CSL and η is the stress ratio (ratio of deviatoric stress to mean effective stress). Considering the mechanism of shearing, it is assumed 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. This modification in Eq. (19) is applied by multiplication of $M/(M-\eta)$ according to Liu and Carter (2002). The total volumetric strain of soil is the sum of elastic and plastic volumetric strains increments.

2.4. Shear strains

According to laboratory data reported in previous studies, the shear modulus is a function of the thermal condition in addition to preconsolidation pressure. Eq. (20) presents the preconsolidation pressure effect on the shear modulus of soil (Houlsby and Wroth, 1991). Using this equation, the shear modulus of an overconsolidated soil (OC) at any ratio of overconsolidation can be calculated from its equivalent amount of normally consolidated soil (NC) at ambient temperature. Eq. (21) is proposed by Hamidi et al. (2015) for deriving shear modulus at elevated temperature.

$$G_{OC} = G_{NC}\left(\frac{p'}{p'_c}\right)\left[1 + d\ln\left(\frac{p'_c}{p'}\right)\right] \tag{20}$$

$$G = G_0\left(\frac{p'}{p'_{cT}}\right)^{b^*}\left[1 + D\ln\left(\frac{T}{T_0}\right)\right] \tag{21}$$

where G_{NC} and G_{OC} are the shear modulus for normally consolidated and overconsolidated soil at ambient temperature, respectively; G_0 is the shear modulus at ambient temperature; d, D and b^* are the dimensionless model parameters that can be deduced by calibration.

Since temperature changes have no impact on stress-induced shear strains, elastic shear strains can be defined as the sum of thermal and mechanical components, i.e.

$$\mathrm{d}\varepsilon_s^e = \mathrm{d}\varepsilon_{sT}^e + \mathrm{d}\varepsilon_{sp}^e \tag{22}$$

where $\mathrm{d}\varepsilon_s^e$ is the elastic shear strain increment; $\mathrm{d}\varepsilon_{sT}^e$ and $\mathrm{d}\varepsilon_{sp}^e$ are the thermal elastic shear strain and mechanical elastic shear strain, respectively. Since thermal shear strains are neglected in this study, considering the independency of mechanical shear strain component and using deviatoric increment mentioned in Eq. (23), the elastic shear strain can be calculated using Eq. (24).

$$\mathrm{d}\varepsilon_s^e = \frac{1}{G}\mathrm{d}q \tag{23}$$

$$\mathrm{d}\varepsilon_{sp}^e = \frac{1}{G}\mathrm{d}q \tag{24}$$

where q is the deviatoric stress. Using flow rule, plastic shear strain can be determined as follows:

$$\mathrm{d}\varepsilon_s^p = \frac{\mathrm{d}\varepsilon_v^p}{\psi} \tag{25}$$

where ψ is the ratio of plastic volumetric strain to plastic shear strain, i.e. flow rule. The total shear strain of soil is the sum of elastic and plastic shear strains increments.

2.5. Flow rule

Previous studies on soil behaviors under various thermal conditions proved that temperature elevation leads to greater ratios of $\mathrm{d}\varepsilon_v^p/\mathrm{d}\varepsilon_s^p$ (Abuel-Naga, 2005), smaller (Ghahremannejad, 2003) or even constant values of plastic strain ratio (Graham et al., 2001).

The structure of soil can also affect its flow rule. Structured clay with positive amounts of Δe_i shows a greater rate of strain changes compared with the corresponding reconstituted soil at the same virgin yielding stress state (Olson, 1962; Graham and Li, 1985; Cotecchia and Chandler, 1997), which means that ratio of $\mathrm{d}\varepsilon_v^p/\mathrm{d}\varepsilon_s^p$ increases in structured clay. Therefore, the following equation is proposed as the flow rule for structured clays:

$$\psi = \frac{\mathrm{d}\varepsilon_v^p}{\mathrm{d}\varepsilon_s^p} = \frac{(1-\delta\Delta e_i)\left[M_T^2(\beta-1)-\eta^2\right]}{2\theta\eta} \tag{26}$$

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

$$0 \le \delta \le \frac{1}{\Delta e_i} \tag{27}$$

According to Hamidi and Khazaei (2010), θ is defined in the terms mentioned below:

- If the flow rule is temperature-independent: $\theta = 1$;
- If ψ increases with elevating temperature: $\theta = 1-\omega$; and
- If ψ decreases with elevating temperature: $\theta = 1+\omega$.

As α is a soil parameter that represents temperature effects on plastic strain increment, the thermal variations of this parameter can be calculated using experimental data reported by Kuntiwattanakul et al. (1995), Graham et al. (2001) and Abuel-Naga et al. (2007):

$$\alpha_{(T)} = \alpha_{(T_0)} + C\left(\frac{T}{T_0}\right) \tag{28}$$

where $\alpha_{(T_0)}$ and $\alpha_{(T)}$ are the parameter values at ambient and elevated temperatures, respectively. Also, C is a dimensionless model parameter which controls thermal evolution of α and can be deduced from calibration.

In Eq. (26), β is another soil fabric parameter derived by curve-fitting of experimental data at $q-p'$ plane. M_T is the slope of critical state line at elevated temperature. However, previous researches proved that structural state of soil and thermal condition does not have major effects on this parameter.

Eq. (26) implies a non-associated 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 non-symmetric.

2.6. Yield surface

A number of mathematical methods have been suggested for defining yield surface equation (Lagioia et al., 1996; Masad et al., 1998; Collins and Kelly, 2002; Cudny and Vermeer, 2004). Assuming the conservation of energy concept during yielding, the following integral results in the determination of yield surface (Roscoe and Burland, 1968; Desai and Siriwardane, 1984).

$$\int_{p'_c}^{p'} \frac{dp'}{p'} + \int_0^{\eta} \frac{d\eta}{\eta+\psi} = 0 \tag{29}$$

Substituting $p'_c = p'_{cT}$ in Eq. (29) and considering ψ in Eq. (26) yield

$$\int_{p'_{cT}}^{p'} \frac{dp'}{p'} + \int_0^{\eta} \frac{d\eta}{\eta + \frac{(1-\delta\Delta e_i)\left[M_T^2(\beta-1)-\eta^2\right]}{2\theta\eta}} = 0 \tag{30}$$

$$\frac{p'}{p'_c} = \left[\frac{q^2(\omega\Delta e_i - 2\theta_T + 1) - M_T^2(\beta-1)(\omega\Delta e_i + 1)}{M_T^2(1-\beta)(\omega\Delta e_T + 1)}\right]^{\theta_T/(\omega\Delta e_i - 2\theta_T + 1)} \tag{31}$$

Replacing $\eta = q/p'$ in Eq. (31), the yield surface can be determined by the following equation:

$$f = \left[\frac{q^2(\omega\Delta e_T - 2\theta_T + 1) - M_T^2(\beta-1)(\omega\Delta e_T + 1)}{M_T^2(1-\beta)(\omega\Delta e_T + 1)}\right]^{\theta_T/(\omega\Delta e_T - 2\theta_T + 1)} - \frac{p'}{p'_c} = 0 \tag{32}$$

It should be mentioned that substituting $\beta = 2$ and $\alpha = \theta = 1$ at Eq. (32) represents the yield surface of the modified Cam–Clay model.

3. Assessment of model parameters

The modified Cam–Clay model introduces five different parameters, i.e. λ, κ, N, M and G, which all can be derived from triaxial test results. The parameter λ is the slope of NCL while κ is the slope of URL. Also N is the void ratio at unit mean effective stress in compression plane ($e - \ln p'$). χ, n, C and D are parameters connecting each of mentioned five modified Cam–Clay parameters to the thermal condition. The parameters χ and n link the NCL variations to the changes in temperature while C and D illustrate the URL slope (κ) and shear modulus (G) with temperature changes. The new parameter b^* is used for demonstration of a nonlinear relation between shear modulus and mean effective stress while d manages the nonlinear relation between shear moduli of NC and OC soils. The parameters δ, β and $\theta(\omega)$ are used for modification of yield surface and flow rule, respectively.

Also b defines effects of structure on the mechanical behavior of soil which also is known as a destructuring index (Liu and Carter, 2002). This parameter represents destructuring rate during yielding of soil and can be derived from isothermal compression test data on the structured soil.

Due to the lack of experimental data on thermal characteristics of structured clays, separate laboratory test results conducted on undisturbed samples of soft Bangkok clay and Boom Clay have been used for verification of model performance. Therefore, triaxial test results on artificially structured clay at ambient temperature in addition to the isothermal compression test results on naturally structured (undisturbed) and destructured samples of Boom Clay have been considered. Information of these data is depicted in Table 1. Modeling parameters of both types of soils are presented in Table 2.

4. Model calibration by test results on Bangkok clay

Due to the lack of experimental data on natural Bangkok clay, results of thermal triaxial tests on artificially cemented samples have been used for model verification (Uddin, 1995). Also test results reported by Abuel-Naga (2005) and Abuel-Naga et al. (2007) were used for evaluation of model predictions for remolded (destructured) samples of Bangkok clay.

4.1. Thermal behavior of Bangkok clay (destructured state)

Abuel-Naga (2005) conducted a series of drained triaxial tests on soft Bangkok clay at 25 °C, 70 °C and 90 °C at which the ambient temperature was considered to be 25 °C.

Fig. 4 shows the comparison of model predictions and experimental data for drained triaxial tests on normally consolidated samples of Bangkok clay at different temperatures. Fig. 4a shows variations of deviatoric stress with axial strain while Fig. 4b depicts volumetric strain changes with axial strain. As can be observed from the figure, model predictions of the drained behavior of soft Bangkok clay at different temperatures have a proper correspondence with experimental data. It also displays that the increases in deviatoric stress and compressive volumetric strain with temperature have been successfully predicted by the model.

Fig. 5 shows model predictions for the undrained behaviors of normally consolidated samples of Bangkok clay at the same confining pressure of 300 kPa and cell temperatures of 25 °C, 70 °C, and 90 °C. Fig. 5a shows the variations of normalized deviatoric stress with axial strain and Fig. 5b indicates normalized pore pressure changes with axial strain at different temperatures. In this figure, model predictions are in good agreement with experimental data and increase in pore pressure with cell temperature was accurately modeled during triaxial tests.

Table 1
Information of soils which have been used in simulation.

Test	Soil type	Reference
Isothermal triaxial tests	Soft Bangkok clay	Abuel-Naga (2005) and Abuel-Naga et al. (2007)
Triaxial tests on cemented clay	Soft Bangkok clay	Uddin (1995)
Triaxial tests on structured clay	Boom Clay	Sultan et al. (2010)

Table 2
List of model parameters for two considered soils.

Clay	b	M	λ	κ	G_A/p'_0	χ	n	C	f	c	d
Soft Bangkok clay	0.2	0.8	0.5	0.1	40	0.26	0	0	–	1.5	0.86
Boom Clay	0.01	0.87	0.18	0.05	20	0.18	0	0	–	–	–

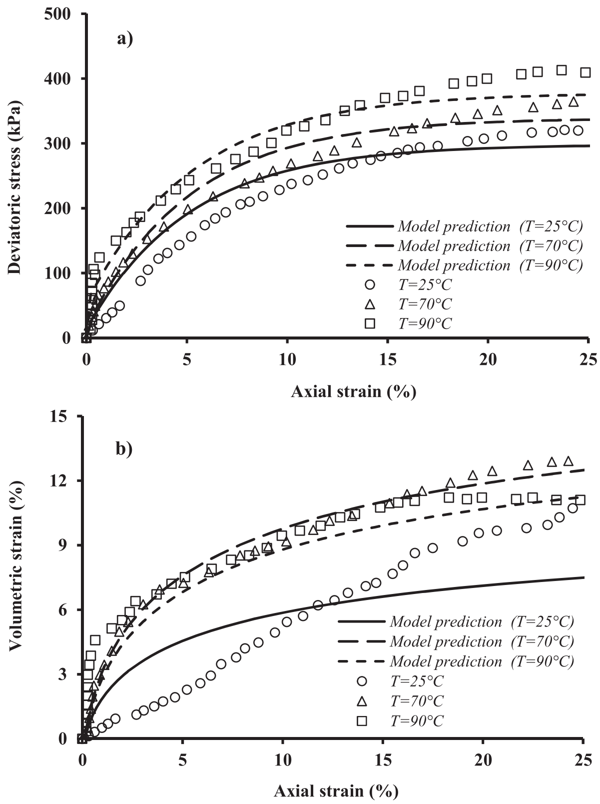

Fig. 4. Model verification for NC Bangkok clay in drained triaxial tests ($p'_c = p'_0 = 300$ kPa): (a) Deviatoric stress-axial strain, and (b) Volumetric strain-axial strain.

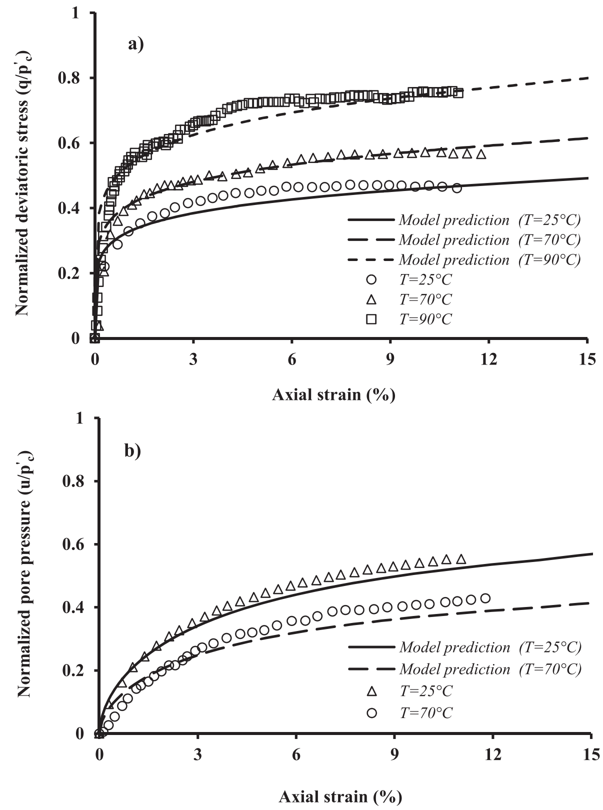

Fig. 5. Model verification for NC Bangkok clay in udrained triaxial tests ($p'_c = p'_0 = 300$ kPa): (a) Normalized deviatoric stress-axial strain, and (b) Excess pore pressure (u) per preconsolidation pressure-axial strain.

Fig. 6 shows model predictions for the behavior of Bangkok clay during drained triaxial tests under a confining pressure of 300 kPa at 70 °C in different overconsolidation ratios. Fig. 6a displays the deviatoric stress and Fig. 6b shows the volumetric strain changes with axial strain.

As can be seen in Fig. 6a, the model-predicted test results under different overconsolidation ratios are of a good match. The failure stresses decrease with the increase of overconsolidation ratio. Also, decrease in compressive volumetric strains due to the increase in overconsolidation ratio was predicted by the model. Although it does not match completely with associated experimental data, it is generally successful in prediction of expected behavior that an overconsolidated cohesive soil shows less compressive behavior due to the increase in overconsolidation ratio.

4.2. Thermal behavior of Bangkok clay (artificially structured samples)

Uddin (1995) reported triaxial test results on artificially cemented samples of Bangkok clay in different cement contents (CC). Simulation of the drained triaxial tests on artificially structured clays with 5%, 10% and 15% Portlend cement under a confining pressure of 600 kPa at 25 °C is showed in Fig. 7. As can be seen,

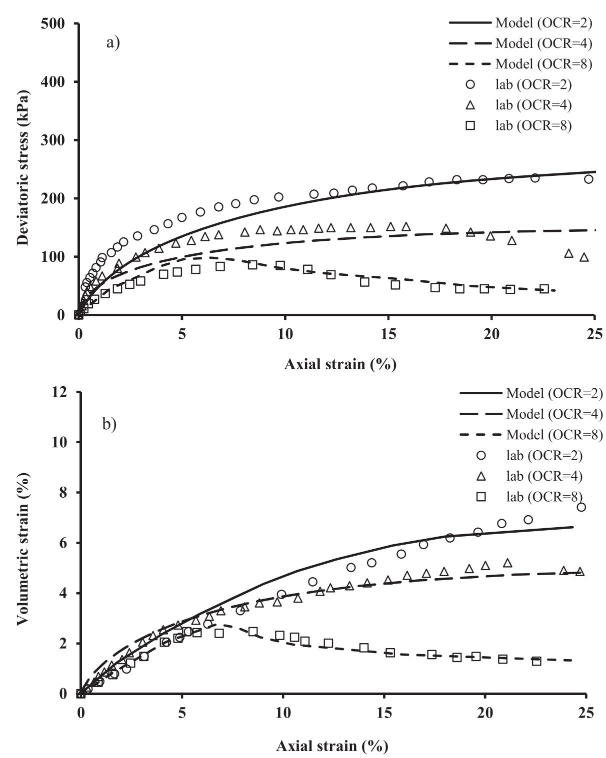

Fig. 6. Model verification for overconsolidated samples of Bangkok clay in drained triaxial tests at 70 °C ($p'_c = p'_0 = 300$ kPa): (a) Deviatoric stress-axial strain, and (b) Volumetric strain-axial strain.

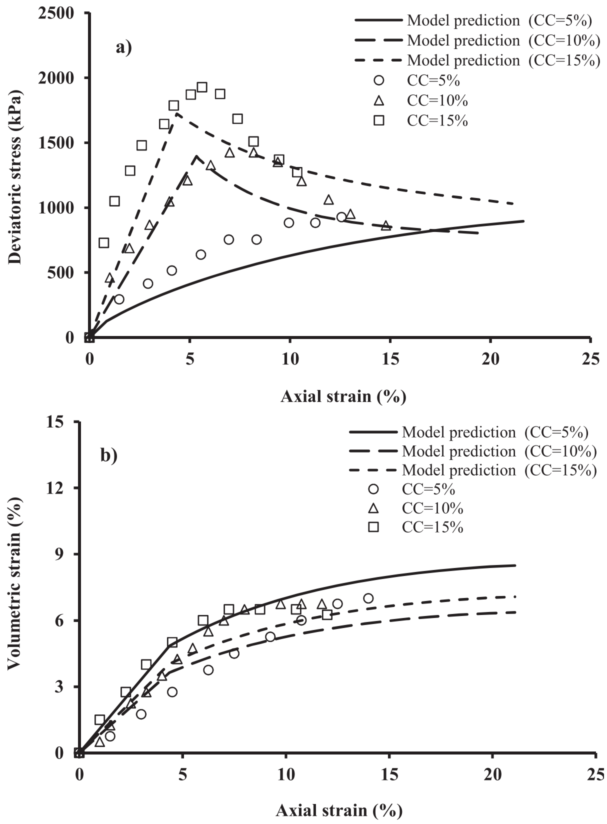

Fig. 7. Model verification for samples of artificially structured soft Bangkok clay with 5%, 10% and 15% cement contents in drained triaxial tests at 25 °C ($p'_0 = p'_c = 600$ kPa): (a) Deviatoric stress-axial strain, and (b) Volumetric strain-axial strain.

model predictions are in a good agreement with experimental data, because it can cover the softening behavior of soil sufficiently. The peak and ultimate strengths have been well simulated.

Numerical conformity of model predictions and experimental data shows that the model has covered soil destructuring and bond degradation effects on the stress–strain behavior during thermo-mechanical loading. Also, a decrease in compressive volumetric strains and tendency for dilation with an increase in cement content has been successfully simulated by a model. This is an important aspect of the behavior of cemented soils.

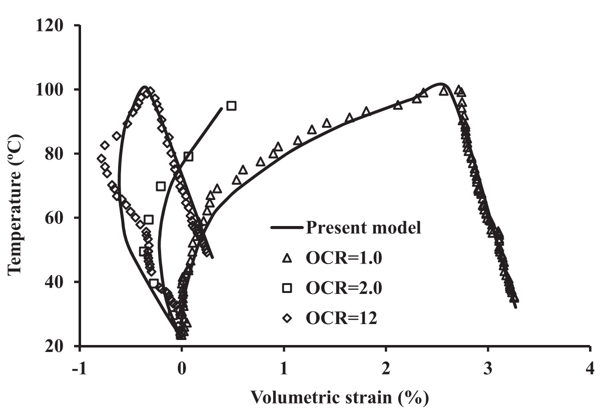

Fig. 8. Model prediction for thermal volume change behaviors of Boom Clay samples with different OCR values (experimental data after Sultan et al., 2002).

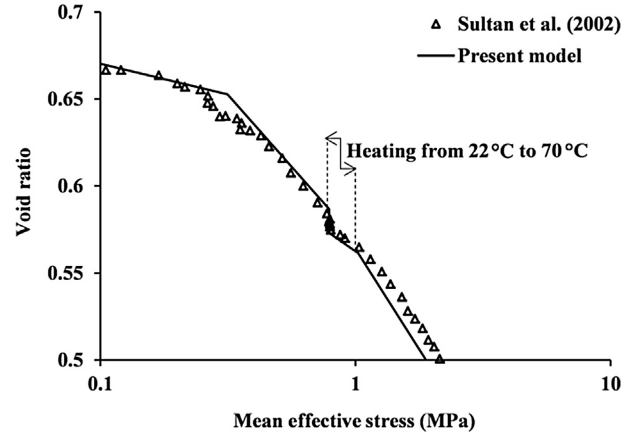

Fig. 9. Comparison between predicted and observed results for thermal consolidation of Boom Clay (Experimental data after Sultan et al., 2002).

4.3. Thermal consolidation tests on intact samples of Boom Clay

In order to illustrate model predictions on thermal consolidation of structured clay, in the following, a number of tests reported by Sultan et al. (2002) are considered. The tests have been performed on samples extracted at a depth of 223 m in the Boom Clay deposit (Decleer et al., 1983), in the underground facilities of SCK·CEN in Mol (Belgium). The Boom Clay is a stiff clay, with a plasticity index of about 50, a natural porosity around 40% and a water content varying between 24% and 30%.

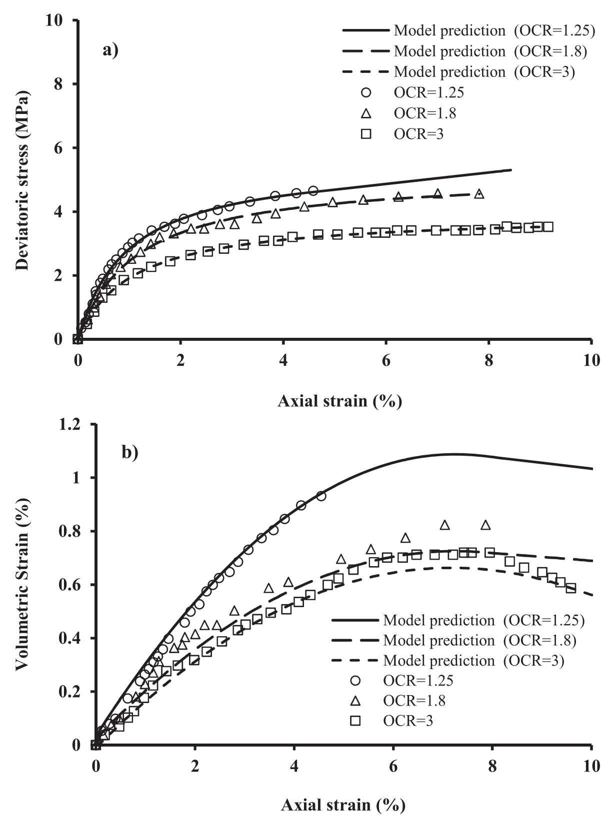

Fig. 10. Model verification for lightly OC samples of structured Boom Clay in drained triaxial tests at 25 °C ($p'_c = 9$ MPa): (a) Deviatoric stress-axial strain, and (b) Volumetric strain-axial strain.

The tests were carried out in an isotropic compression cell designed to support high pressures and temperatures. The Cam–Clay parameters have been reported by Sultan et al. (2002) using several compressibility curves, for example, $\kappa = 0.05$ and $\lambda = 0.18$. Also, the value of thermal coefficient expansion for the drained state is obtained by averaging the values obtained from the cooling phases, giving $\alpha = -6 \times 10^{-5}$ $(°C)^{-1}$.

The volumetric response of model is compared to the experimental data, as shown in Fig. 8. In this figure, the dilation phase is progressively vanishing with decrease in OCR and the amplitude of the contracting phase is increased. A satisfactory reproduction of the thermal volumetric strains tests can be observed in different OCR values.

Fig. 9 shows consolidation curve of Boom Clay in ambient temperature (p'_{c0}= 0.32 MPa) which is well produced by the present model. The thermal hardening phenomenon is also correctly reproduced in this figure where heating from 22 °C to 70 °C increased the mean effective pressure from 0.8 MPa to 0.98 MPa.

4.4. Model verification by test results on intact samples of Boom Clay

Triaxial tests on intact samples of Boom Clay reported by Sultan et al. (2010) were used in this text. Fig. 10 shows the results of simulation for drained behavior in a triaxial test at a confining stress of 9 MPa for different OCR values at 25 °C. Also, Fig. 11 shows the model predictions compared with experimental results of Sultan et al. (2010) for highly overconsolidated samples of Boom Clay.

As shown in Fig. 10, model predictions are quite accurate at low OCR values. In Fig. 10a, it is evident that increase in OCR decreases

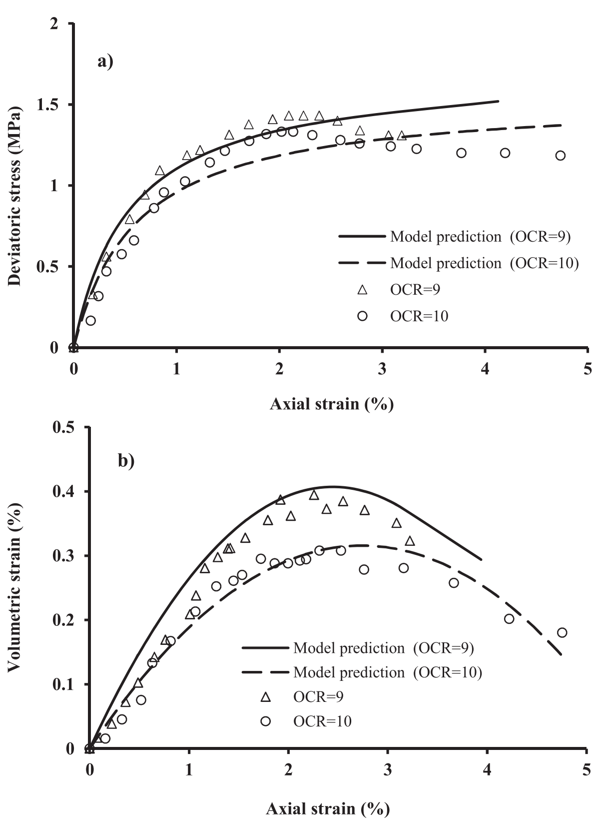

Fig. 11. Model verification for highly OC samples of structured Boom Clay in drained triaxial tests at 25 °C (p'_c = 9 MPa): (a) Deviatoric stress-axial strain, and (b) Volumetric strain-axial strain.

the deviatoric stress, which is compatible with experimental data. Also, as indicated in Fig. 10b, compressive volumetric strains decrease with increase in OCR, which is in agreement with the general framework of the behavior of cohesive soils.

Fig. 11 shows a comparison of model predictions with experimental data for higher OCR values. In this figure, simulation of soil behavior shows a fair compatibility to real data. According to Fig. 11b, a decrease in compressive volumetric strains with an increase of OCR and tendency of samples to dilate in higher axial strains have been simulated by the model. It confirms model's applicability in numerical schemes for the solution of boundary value problems of geotechnical engineering.

5. Conclusions

In this paper, a constitutive model is developed for simulation of the thermo-mechanical behavior of structured clays at elevated temperatures from ambient temperature up to 100 °C, including deviatoric stress-axial strain, volumetric strain-axial strain, and pore pressure-strain diagrams. This model is applicable to the isothermal condition in triaxial tests conducted on naturally and artificially structured clays. The model is based on modified Cam–Clay model and critical state soil mechanics. Model parameters can all be easily acquired from triaxial tests. Considering the various additional model abilities in comparison with the basic model, it was proved to be adequate for predicting the effect of coupled thermo-mechanical conditions on soil behaviors. In order to verify model predictions, simulation results were compared with experimental data for two types of clays.

For the further studies, extending the model for prediction of unsaturated structured clays is desirable. Also, the lack of experimental data for structured clays revealed the need for comprehensive studies on the thermo-mechanical behaviors of both naturally structured and artificially cemented clays.

Conflicts of interest

The authors wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

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