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A Constitutive Model for Cemented Clay in a Critical State Framework

한계상태이론을 이용한 시멘트 고화처리 점토에 대한 구성 모델

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요 지

연약지반 개량을 위한 시멘트의 사용은 깊은 심도의 점토 지반을 개량하는데 일반적으로 사용되는 기술이 되었다. 시멘트는 지반의 강도를 증가시키고 압축성을 감소시키는 역할을 한다. 시멘트-흙 혼합물의 강도 증가에는 여러 가지 요소가 있는데 이 중 대표적인 것은 시멘트량, 흙의 종류, 함수비, 양생시간등을 들 수 있다. 시멘트 첨가량이 적은 경우, 전단 강도증가는 기본적으로 시멘테이션 효과로 인한 점착력의 증가에 의한 입자들간의 마찰력으로부터 발생한다. 이러한 거동은 과압밀된 흙의 거동과 유사함을 볼 수 있다. 시멘트량이 많은 경우, 강도 증가의 주원인은 입자간의 물리적 결합에 기인하는데 이는 연약한 암석과 비슷한 거동을 한다. 시멘트 고화처리 흙의 응력-변형 거동을 분석하기 위해 한계상태 이론을 적용하였다. 그리고, 토립자간의 시멘테이션 효과를 반영하기 위해 새로운 한계상태 파라메타를 도입하였으며 시멘트 고화처리 점토의 거동을 분석하기 위한 새로운 한계상태 모델을 제시하였다.

Abstract

The addition of cement to soft soil has become a popular technique for improving the mechanical characteristics of deep clay deposits. The addition of cement increases the strength and decreases the compressibility of the material. There are many factors affecting the strength of the soil-cement mixture, which include the amount of cement, the type of soil, the water content in the soil and the curing time. With small amounts of cement, the shear strength of the material is basically derived from frictional contact of the particles with an added cohesion due to cementation. It behaves like an overconsolidated soil. At higher cement content, a major component of the strength is based on physical bonding of the particles and the soil can behave like to a soft rock.

A critical state framework is used to capture the essential stress-strain response of the cement-treated clay. New critical state parameters are introduced to reflect the effect of cementation and a new critical state model for analyzing the behaviour of cement treated clay is introduced.

Keywords : Soil cement, Constitutive model, Critical state ratio, Stress strain relationship, Soil cement, Modified Cam Clay model.

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1. Introduction

Many geotechnical projects involved usage of poor soils, such as dredged soil or marine deposits in the harbor, which necessitates their improvement by physical or chemical methods depending on the types of soil and the nature of the projects. Specifically, deep stabilization of soft soils is being used more often in many countries around the world.

The addition of cementing agents, especially Portland cement, is often used to improve the mechanical properties of soils, and this method has been extensively employed to solve such geotechnical problems. Depending on the intended function of the stabilized soil, either to decrease settlements or to improve stability, a desired increase in the stiffness and strength can in most cases be achieved, provided that the right amount and type of stabilizing agent could be used.

The behaviour of artificially cemented soil has been a subject of considerable interest over the last decade because of increasing development of large offshore structures founded on these materials. Therefore, the evaluation of the yielding behaviour of cement-treated clay is particularly important with regard to the design of structures founded on these materials. As the clay changes from a natural state to a reconstitute cemented state, there is a dramatic change in stiffness and other properties.

An important consideration of cement-treated soils is that bonding effects of soil particles control their yield behaviour, which can be independent of the previous stress history. There are many approaches for describing the behaviour of cement-stabilized soils. Constitutive soil models are usually based on the classical principles of soil mechanics, in which the current state of the material is expressed in terms of the effective stresses and the void ratio, and its stress history is usually expressed in terms of the maximum pre-consolidation pressure. Artificially cement-treated soils have components of stiffness and strength, which cannot be described by the above principles and this usually stems from the influence of structure caused by cementation(e.g. Leroueil; Vaughan, 1990; Gens and Nova, 1993).

Many researchers have made important contributions to the understanding of cemented soils behaviour or structured soils caused by various adhesion-inducing processes such as cementation, thixotrophy (aging), leaching or even over-con -solidation (e.g. Tavenas and Leroueil, 1990; Leroueil and Vaughan, 1990). Other research has been carried out on a variety of structured soils such as residual soils, cemented sands, marls, soft clays, lignite and clay shales (Malandraki and Toll, 1996). However, the cement-treated clay has not been sufficiently investigated.

In this paper, the behaviour of an artificially cement-treated clayey soil when subjected to drained triaxial tests was studied. We assumed that the increase strength in the cement-treated soil is mainly due to the bonding of soil particles and that the soil state finally goes to the critical stress state by reduction of bonding effects between soil particles($m \to M_{cs}$). This concept is similar to the assumption that was made by Adachi et al (1993) to describe an elasto plastic constitutive model for soft rock. The term bonding is used in general sense and includes all type of cohesive forces (bonds) at the inter-particle contacts by cement agents. The proposed model assumed that the value of the constant λ values of Modified Cam Clay model changes during the shear strain where the parameter λ is the slope of the normal consolidation line. The yield and plastic potential functions are affected by the variation of λ'. Similar to the original family of critical state models, the proposed constitutive model requires only a minimum number of material parameters, which can be obtained using standard laboratory tests.

2. The proposed model

The proposed model is based on the critical state concept for soil state proposed by Schofield and Wroth(1968). The major features are the bonding stress ratio m and variable λ'. The former considered the cementation effects and the effect of breaking of bonding with the shear strain. The latter considered the plastic deformation at the beginning of shear.

Yield function in terms of cohesion and friction angle and associated flow rule are presented and the proposed model simulates the breaking of bonding using new concepts. During the development of the model, efforts were made to retain simplicity of the model, while attempting to accurately describe the characteristics of cement treated clayey soil.

Specifically, the proposed model uses the strain incremental method for the behaviours of strength and volume changes instead of stress path method as in the Modified Cam Clay model.

2.1 Basic concepts of the constitutive model for cement-treated soils

It is assumed that the material strength is composed of frictional strength and cohesion. In addition, it is also assumed that at the early stage of the straining processes the frictional component of strength is relatively small. However, the strength due to cohesion of the material decreases with the increase in deformation and finally the frictional strength controls the strength of the materials. In this study, the cementation effect is accounted for through the bonding stress ratio m.

As shown in Fig. 1, it is assumed that the increase in the strength of cement-treated soils is mainly due to bonding effects between the soil particles. The increases in strength and volume change characteristics of cement-treated soils are related to the bonding stress ratio m and the variable λ' is a function of mean normal effective stress p' and void ratio e. With the advance of deformation, cementation effects of particles decrease because of breaking of bonds and the soil characteristics finally approach to those of the critical state. The bonding stress ratio m considers the bonding effect and decreases with shear strain. In order to preserve the generality of formulation, we assumed that the decrease of bonding effect is related to the shear strain.

In the proposed model, m is defined as:

$$m = M_{cs} + M'(\varepsilon^p) \tag{1}$$

On rearrangement it gives:

$$m = M_{cs} + \frac{c}{p'_c} e^{-k_p \varepsilon^p_q} \tag{2}$$

in which M_{cs} is the critical stress ratio in the Modified Cam Clay model, c is the cohesion intercept, e is the exponential function basis of natural logarithm, p'_c is the midpoint along the mean normal effective stress axis p', k_p is a control parameter that describes bond degradation ratio between soil particles and ε^q_p is the total plastic shear strain. In Eq.(2), $k_p = f(P'_0)$ is a parameter to control the bond degradation rate, which is a function of the initial confining pressure and k_p is given by

$$k_p = f(P'_0) \tag{3}$$

In general, experimental results show that the reduction rates of bonding to the initial value of m under low confining pressure are faster than high confining pressure. The first term of Equation takes into account the frictional strength of unbonded soils and the second term accounts for the amount of bonding effect of cement treated soils. Obviously, the amount of bonding will gradually decrease with the development of irreversible plastic shear strains. Eq.(2) implies that bonding stress ratio m reduces to the

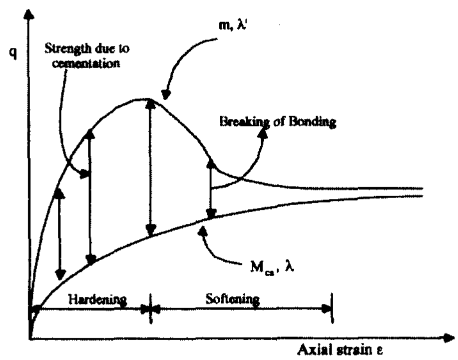


Fig. 1 Stress-strain relationship with the breaking of bonding effects

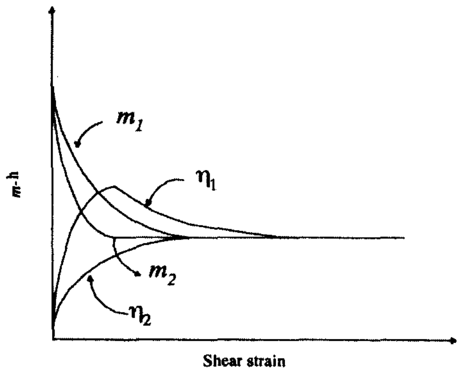


Fig. 2 Relationship between m and $\eta(q/p')$ with increasing plastic shear strain

critical stress ratio M_{cs} of the Modified Cam Clay model if the cohesion intercept c vanishes due to the elimination of all bonding effects by shearing.

The variations of strength and volume change behaviours of the cemented soil mainly depend on the cementation and breakage of bonds between soil particles. This model considers the breaking of bond between soil particles to the shear strain through the bonding stress ratio m. The bonding stress ratio m decreases constantly with shear strain and finally approaches the critical state stress ratio M_{cs} (See Fig.2). When the bonding stress ratio m is larger than the current stress ratio $\eta(q/p')$ as shown in Fig.2, the yield locus expands and the strength increases. In this case, volumetric contraction takes place. However, when $m < \eta$, the size of yield locus decreases and a softening effect accompanied by volumetric dilation will be predicted.

Fig. 2 shows that the bonding stress ratio m decreases with shear strain and finally reaches the critical stress ratio M_{cs}.

Concurrently, current stress ratio η increases and goes to the point at $m=\eta_2$ if cohesion intercept is zero ($c=0$). However, if the cohesion intercept is larger than zero ($c>0$) by the bonding effects, then the current stress ratio η is larger than m. The proposed model assumes that the bonding stress ratio m is not constant but is a function of cohesion, initial confining pressure, etc.

2.2 Description of the proposed model

One of the important aspects of the behaviour of structured soils, such as naturally or artificially cemented soils, is that the size of the yield curve is not only related to the void ratio, it is predominantly related to the strength of bonds between the soil particles. It has also been recognized that even in the overconsolidated range, soils are not perfectly elastic and the stress area in which the soil is perfectly elastic is rather limited. In these structured soils, large strains are generally necessary to reach a complete destruction of the soil structure, and yielding corresponding to this process of destruction happens progressively (Pellegrino (1970), Leroueil et al.(1979), Maccarini(1987)).

It is assumed that the bonding effect of the cement-treated soil gives rise to an elliptical cementation yield surface as shown in Fig. 3. Due to cementation, the material exhibits a tensile strength; therefore the yield surface passes through point B. Consequently, the coordinate of point A at which the yield surface crosses the p'-axis is (c', 0).

With the increase of shear strain, the bonding effect between soil particles decreases and therefore, the yield surface shifts, and tensile strength c' is eliminated Three parameters define the new yield locus: p_0, which controls the yielding of cemented soil in isotropic compression, the bonding stress ratio m, and the tensile strength c' which is related to the cohesion of cement-treated soils. The parameters in Eqs.(4) and (2), c' and m are functions of the cumulative plastic shear strain $\varepsilon^q{}_s$ and are defined as follows.

$$c' = \frac{c}{M_{cs}} e^{-k_p \varepsilon^p{}_q} \tag{4}$$

$$m = M_{cs} + \frac{c}{p'_c} e^{-k_p \varepsilon^p{}_q} \quad \text{(Eq.2. bis.)}$$

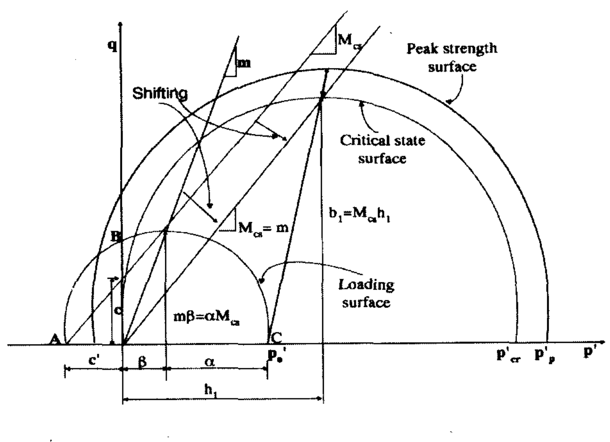


Fig. 3 Yield surface for cement-treated clay

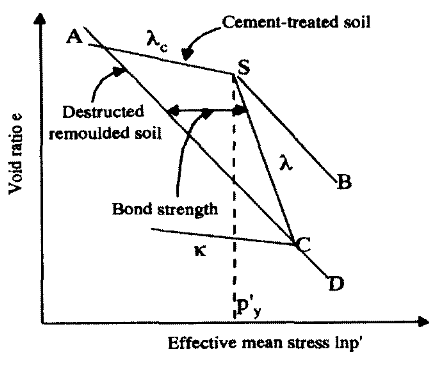


Fig. 4 Schematic behaviour of isotropic compression curve for cement-treated soil (Modified after Konrad, 1997 & Kavvadas et al. 1998)

in which c is the cohesion intercept, M_{cs} is the critical state stress ratio and p_c' is the midpoint along the normal effective stress axis.

A loss of cementation occurs during yielding, therefore, the bonding stress ratio m decreases as the yield surface evolves during loading. There is no exact method for determining the purely elastic domain by laboratory or field test.

Compression of granular materials due to high confining pressure and subsequent shearing induce grain crushing (Lee & Seed, 1967; Vesic & Clough, 1968).

The effect of crushing of grains on the position of the critical state line has been identified by Been et al (1991) and schematically shown by Konrad (1997). The slope of the critical state line before yielding can be characterized by λ_c, and at stresses larger than yield the slope is λ. In the isotropic consolidation curves of bonded soil, after reaching the yield point (S), the material state follows a compression curve such as (SB) or (SC) depending on the post-peak rate of bonding degradation. These curve towards the (AD) line, but do not necessarily reach it, thus showing that the bonding effect between soil particles may not be eliminated completely even after appreciable straining beyond the yielding point, a fact widely accepted in literature (e.g. Leroueil and Vaughan, 1990; Burland, 1990). These concepts apply to the proposed model. However, the proposed model assumes that the slope of λ_c and λ continually change depending on the mean normal effective and concurrent void ratio e. The variable $\lambda'(p', e)$ is obtained from the regression method fitting the $e-\ln p'$ isotropic consolidation data points and obtaining the position of the normal consolidation line as shown in Fig.5.

For the variable λ' in the $e-\ln p'$ isotropic consolidation curve, the model uses a 3rd order polynomial equation to fit the experimental data using the regression method. It is also assumed that the normality rule applies to cement-treated soil, so that it will have a family of yield loci the same as the Modified Cam Clay model. Therefore the plastic potential will be given by the same family of curves in the $p'-q$ plane. According to studies conducted on cement-treated soils, consolidation yield stress is nearly equal to or a slightly larger than its unconfined compression strength q_u and the stabilization process leads to apparent heavily overconsolidated characteristics as compared with the original soil (Kawasaki et al, 1984).

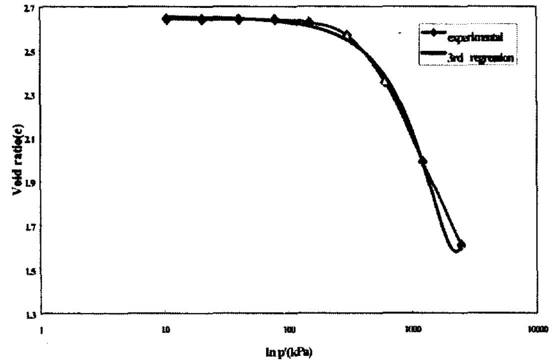


Fig. 5 Regression method for the variable λ' in the isotropic compression curve

If the material behaviour follows the Modified Cam Clay model, as is the case for low confining pressure below the apparent over consolidation ratio enclosed by the elastic domain, this may lead to overestimating the elastic domain of cement-treated soils. To avoid the large elastic domain of the Modified Cam Clay model, the proposed model assumes that the elasto plastic behaviour occurs from the beginning of shear in the cement-treated clay soil and the variable λ' applies instead of constant κ in the swelling line of isotropic consolidation curves. In reality, the size of elastic domain is very small compared to the size of elasto plastic behaviour domain and most soils behave elastically over a very limited range. Therefore, in the proposed model, the purely elastic domain contrary to classical critical state model is not considered. The variable λ' in the $e-\ln p'$ consolidation curve is,

$$\lambda' = \frac{de}{d\ln p'} = \frac{de}{dp'} \cdot \frac{dp'}{d\ln p'} = \frac{de}{dp'} \cdot p' \tag{5}$$

It can be seen from Eq. (5) that λ' is a function of p', e, i.e.

$$\lambda' = \lambda(p', e) \tag{6}$$

where $p' = (\sigma_1' + 2\sigma_3')/3$ is the mean effective normal stress in triaxial testing conditions, σ_1', σ_3' are major and minor effective stresses respectively and e is the corresponding void ratio.

2.3 Formulation of the constitutive model

A specific mathematical formulation for cemented soils consistent with the requirements outlined above will be presented. The main purpose of this particular model is to illustrate the strength and volumetric behaviour of cement-treated soils. The proposed model follows the associated flow rule and considers the reduction of bonding with strain increments using the bonding stress ratio m. In particular, the model follows the strain control and the plastic potential and yield loci change with shear strain. For triaxial conditions, the expression for the yielding function of Modified Cam Clay model, f can be written as follows:

$$f = q^2 - M_{cs}^2[p'(p_0' - p')] = 0 \tag{7}$$

To account for the effect of bonding, the above expression for the yield function takes the following form:

$$f = (p' - \beta)^2 \cdot (M_{cs}\alpha)^2 + q^2\alpha^2 - (M_{cs}^2\alpha^4) = 0 \tag{8}$$

in which M_{cs} is the critical stress ratio, $q = \sigma_1' - \sigma_3'$ is the deviatoric stress, p' is mean normal stress and α, β and c' are defined below:

$$\alpha = \frac{p_0' - c'}{2} \tag{9}$$

$$\beta = \frac{p_0' + c'}{2} \tag{10}$$

$$c' = \frac{c}{M_{cs}} e^{-k_p \varepsilon^p_q} \qquad \text{(Eq. 4 bis)}$$

The shapes of the yield surfaces are shown in Fig. 3. The two parameters that control the enlargement of the yield surface due to bonds between soil particles are p_0' and the cohesion intercept c. Eq.(8) reduces to the yield function of the Modified Cam Clay model if the cohesion intercept c is equal to zero. It is assumed that the total strain rate tensor $d\varepsilon_{ij}$ is composed of two parts, namely, an elastic component $d\varepsilon^e_{ij}$ and a plastic component $d\varepsilon^p_{ij}$.

$$d\varepsilon_{ij} = d\varepsilon^e_{ij} + d\varepsilon^p_{ij} \tag{11}$$

The elastic component in the proposed model is calculated using the Eqs.(12) and (14) and shear modulus G' is given by Eq.(13). The incremental elastic shear strain can be obtained from:

$$d\varepsilon^e_s = \frac{dq}{3G'} \tag{12}$$

in which

$$G' = \frac{3(1-2\nu')}{2(1+\nu')} \cdot \frac{\nu p'}{\chi} \tag{13}$$

and $d\varepsilon^e_s$ is the increment of elastic shear strain, dq is the incremental change in deviatoric stress, ν is the Poisson's ratio and v is the specific volume. The increment of elastic volumetric strain is calculated from Eq.(14), which is derived from the definition of the swelling line χ in the isotropic consolidation curve.

$$d\varepsilon^e_\nu = \frac{\chi}{p'(1+e_0)} \cdot dp' \tag{14}$$

where $d\varepsilon^e_\nu$ is the incremental elastic volumetric strain, χ is the slope of the swelling line in the $e - \ln p'$ plane, e_0 is the void ratio at the beginning of loading and p' is the mean normal effective stress. For the constitutive equation, the proposed model follows the general plastic stress strain relationship proposed by Schofield et al(1968).

$$\begin{bmatrix} \delta\varepsilon^p_p \\ \delta\varepsilon^p_q \end{bmatrix} = \frac{-1}{\left[\frac{\partial f}{\partial p_0}\left[\frac{\partial p_0'}{\partial \varepsilon^p_p} \cdot \frac{\partial g}{\partial p'} + \frac{\partial p_0}{\partial \varepsilon^p_q} \cdot \frac{\partial g}{\partial q}\right]\right]} \cdot \begin{bmatrix} \frac{\partial f}{\partial p'}\frac{\partial g}{\partial p'} & \frac{\partial f}{\partial q}\frac{\partial g}{\partial p'} \\ \frac{\partial f}{\partial p'}\frac{\partial g}{\partial q} & \frac{\partial f}{\partial q}\frac{\partial g}{\partial q} \end{bmatrix} \begin{bmatrix} \delta p' \\ \delta q \end{bmatrix} \tag{15}$$

The plastic stress strain response in matrix equation using the relationship described in Eqs.(14) and (17~21) can be summarized as follows. The hardening rule of the Modified Cam Clay model :

$$\begin{bmatrix} \delta\varepsilon^p_p \\ \delta\varepsilon^p_q \end{bmatrix} = \frac{-1}{A[BD]} \begin{bmatrix} B^2 & BC \\ BC & C^2 \end{bmatrix} \begin{bmatrix} \delta p' \\ \delta q \end{bmatrix} = \begin{bmatrix} F_1 & F_2 \\ F_3 & F_4 \end{bmatrix} \begin{bmatrix} \delta q/3 \\ \delta q \end{bmatrix} \tag{16}$$

In Eq.(8), the parameters α and β are functions of p_0', Therefore, we expanded the yield function using the chain rule as follows.

$$\frac{\partial f}{\partial p_0}=\frac{\partial f}{\partial \alpha}\frac{\partial \alpha}{\partial p_0}+\frac{\partial f}{\partial \beta}\frac{\partial \beta}{\partial p_0} \tag{17}$$

$$A=\frac{\partial f}{\partial p_0}=-(p'-\beta)M_{cs}^2-\alpha M_{cs}^2 \tag{18}$$

$$B=\frac{\partial f}{\partial p'}=2(p'-\beta)M_{cs}^2 \tag{19}$$

$$C=\frac{\partial f}{\partial q}=\frac{2q}{\beta^2} \tag{20}$$

$$D=\frac{\partial p_0'}{\partial \varepsilon_p^p}=\frac{\nu p_0'}{\lambda'-x} \tag{21}$$

Eq.(15) is applicable only if plastic strains are occurring. The compliance matrix is symmetric because the model follows the normality rule. Since the determinant of the matrix in Eq.(15) is zero, the inverse of the matrix cannot be calculated and therefore it cannot be used to calculate the stress increment from the strain increments. To avoid this shortcoming, the authors assume that the initial value of η is not zero but close to zero for the calculation of the model. According to the predicted values of strength and volumetric strain of cement treated clayey soil, the effects of assumed initial values can be ignored.

For the calculation of deviator stress increment δq, the proposed model uses the strain path method. Eq.(16) leads to the following relationship for the deviator stress increment:

$$\delta q=\frac{3\delta\varepsilon_p^q}{F_3+3F_4} \tag{22}$$

in which

$$F_3=\frac{-1}{A[BD]}\cdot BC \tag{23}$$

$$F_4=\frac{-1}{A[BD]}\cdot C^2 \tag{24}$$

in which B and C are obtained from Eqs.(19) and (20).

2.3.1 Flow rule

In metal plasticity, it is customary to assume that the plastic potential is identical to the yield surface. Experimental as well as theoretical considerations suggest, however, that in general the plastic potential is not the same as the yield surface for sands and clays, and that in the latter case the deviation from the normality rule is not as large as that in the former. Further research is needed to assess whether the flow rule should be associated or not for cement treated clay. The flow rule determines the relationship between the plastic volumetric strain increment ε_p^p and the shear strain increment $d\varepsilon_q^p$. The proposed model uses an associated flow rule, i.e., the normality and the plastic strain increments followed from a mechanism of plastic deformation relating the normal to the plastic potential at the current effective stress state.

$$\delta\varepsilon_p^p=\chi\frac{\partial g}{\partial p'} \tag{25}$$

$$\delta\varepsilon_p^p=\chi\frac{\partial g}{\partial q} \tag{26}$$

Therefore, the vector of plastic strain increment $\delta\varepsilon^p$ is in the direction of the outward normal to the yield locus. This implies from Eqs.(25) and (26) that when plastic deformations are occurring, it can be seen that the flow law is a revised form of the Modified Cam Clay model.

$$\frac{\delta\varepsilon_p^p}{\delta\varepsilon_q^p}=\frac{\partial f/\partial p'}{\partial f/\partial q}=\frac{2(p'-\beta)}{\alpha^2} \tag{27}$$

here p' is the mean normal effective stress and α and β are defined in Eqs.(9) and (10). In order to obtain total strain rates, elastic strain rates are added to the plastic ones.

2.3.2 Hardening Function

The proposed model possesses the isotropic hardening rule. The isotropic hardening rule controls the size of the yield surface and describes the shifting of the yield surface. The change of the size of the yield surface due to the plastic volumetric strain increment can be written as:

$$\delta p_0'=\frac{1+e}{\lambda'-x}\cdot\frac{q^2/M_{cs}^2+(p'^2+p'c')}{(p'+c')}\cdot d\varepsilon_p^p \tag{28}$$

in which $d\varepsilon_p^p$ is the plastic volumetric strain increment, e is the void ratio, λ, x are the compressibility parameters during compression and rebound and M_{cs} is the critical stress ratio in the Modified Cam Clay model. The deviator stress is q, p' is the mean normal effective stress, tensile

Table 1. Physical properties of the clay.

Soil	$\rho[t/m^3]$	$w_N(\%)$	$w_L(\%)$	$q_u(t/m^2)$	S_t	O_c	pH
Clay	1.55	78	70	15	18	1.3	7.4

Notes ; ρ:Density, w_N:Water Content, w_L:Liquid Limit, S_t:Sensitivity, O_c:Organic content.

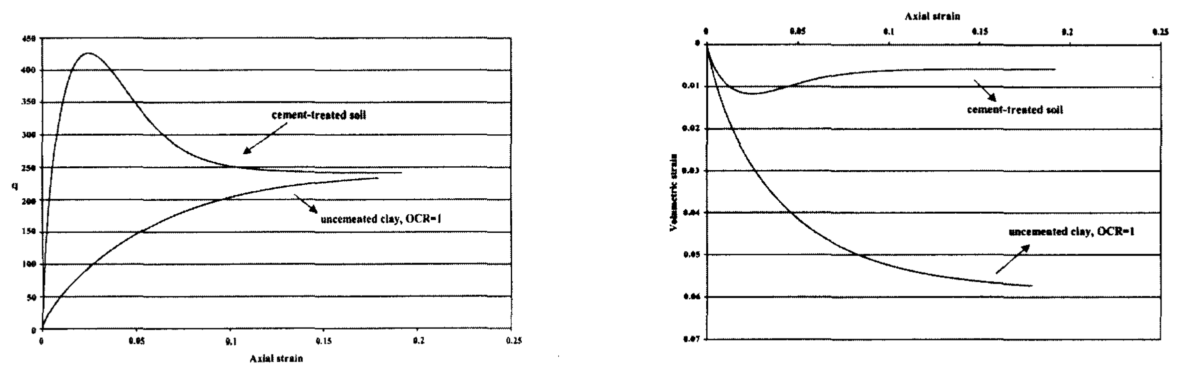
strength c' is the same as the Eq.(4). Eq.(28) reduces to the isotropic hardening rule of the Modified Cam Clay model if the tensile strength c' is going to zero.

3. Evaluation of the constitutive model

Compared to the uncemented soils, the addition of small amounts of cement to the soil significantly increases the peak strength and stiffness of the soil. Previous studies have shown that the shear strengths of natural and artificially cemented soil could be generally represented by a linear Mohr Coulomb failure criteria defined by the cohesion intercept c and the friction angle ϕ'. Test results taken from the artificially cemented soil by Ahnberg (1996) show that the cohesion intercept c is mainly a function of the cement content and the friction angle ϕ' is not affected by the cementation at large strains. Also, the highly brittle behaviour observed for low confining stresses changes to a ductile behaviour as the confining stress increases.

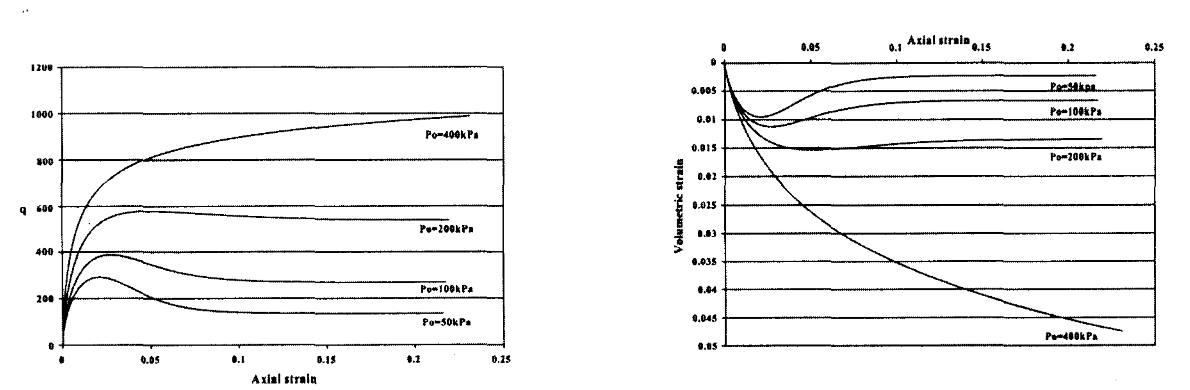
The proposed model followed the concepts described above. First, we will explain the general behaviour of the proposed model before examining the simulated results of laboratory experiments using the proposed model.

Fig. 6 presents a schematic comparison between the S_t calculated stress strain relationships of cement treated clayey soil and normally consolidated natural clay. As shown in Fig. 6, when a cohesion intercept of $c=0$ is assumed, the stress strain relationship of the normally



(a) Relationship between deviator stress and axial strain (b) Relationship between volumetric strain and axial strain

Fig. 6 Schematic diagram of stress-strain relations predicted by proposed model



(a) Relationship between deviator stress and axial strain (b) Relationship between volumetric strain and axial strain

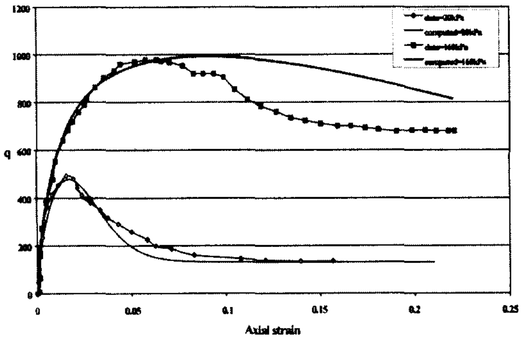
Fig. 7 Computed drained triaxial compression tests consolidated clay, loose sand and the behaviour of soft rock at high confining pressure can be simulated.

On the other hand, when the condition $c \neq 0$ is applied, the behaviour of cement treated soils, such as an over-consolidated clay, dense sand and soft rock at low confining pressure can be predicted. Secondly, the model has been applied to the simulation of drained triaxial compression tests starting from different values of initial confining pressure. All tests have the same parameter values. The results are plotted in Fig. 7. It can be noted that the reduction of bonding effects with confining pressure referred to previously is well reproduced, i.e. the highly brittle or large softening behaviour for low confining pressure changes to a ductile behaviour as the confining pressure increases and more dilatant low confining stresses than at high confining stresses. These are similar to the behaviour of soft rock and overconsolidated clay observed by Rocco et al (1993). According to their observation, the behaviour of soft rock at low confining pressure shows that failure occurs abruptly and is followed by a large reduction in strength. However, at high confining pressure, the behaviour is fairly ductile and failure occurs at large strains.

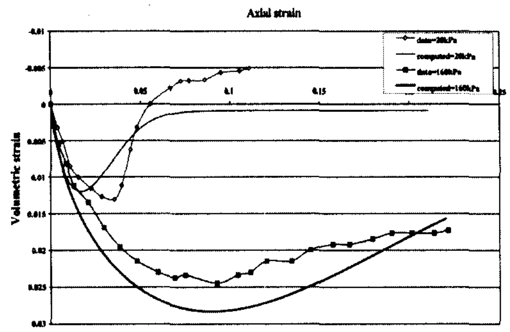
The model is applied to the results of a typical laboratory test on cement-treated clay. The aim is not to model a particular set of results but to demonstrate the capability of the formulation to reproduce the main characteristics of cement treated soils. The proposed model is characterized by only five parameters, which may all be determined from the experimental data obtained from drained triaxial tests.

Samples with 10% cement content were prepared by mixing dry additives and soil at natural water content. The stabilized soil samples were compacted and stored in airtight plastic sample tubes. The cement used in the tests was Portland cement. Samples with the diameter of 50mm and the height of 100mm were used. The dominant clay mineral was illite and the clay content of the tested soil was 65% by the Unified Soil Classification System. The general properties of the clay soil are listed in Table 1.

Results of the drained triaxial tests are shown in Fig. 8, in which the samples behave more dilatantly at low confining stresses than at high confining stresses. The parameters used for the model prediction were easily acquired from the drained traixial tests and standard consolidation tests. The critical stress ratio M_{cs} was obtained from the final strengths reached during triaxial shearing. Poisson's ratio ν was assumed and the elastic parameter $\varkappa$ was determined from the slope of the reloading line during isotropic consolidation. The value of the cohesion intercept c was determined from Mohr Coulomb's failure criteria. The parameter k_p controlling the reduction rate of bonding was evaluated from the experimental result. For the computation analysis of lower confining pressures $\sigma_3 = 20kPa$, the model parameters are Poisson's ratio $\nu = 0.25$, $\varkappa = 0.003$, $c = 250kPa$ and $M_{cs} = 1.9$, $k_p = 50$, $k = 50$. At high confining pressures $\sigma_3 = 160kPa$, the model uses a different critical stress ratio $M_{cs} = 1.7$ because the test results showed that the failure stress ratio did not reach the same values of M_{cs} at large strain. The control



(a) Calculated and experimental data on the deviator stress and axial strain



(b) Calculated and experimental data on the volumetric and axial strain

Fig. 8 Comparison between calculated and experimental data from drained triaxial tests with confining pressure $\sigma_3' = 20kPa$ and $160kPa$ (data from Ahnberg (1996)) parameter of reduction rate of bonding $k_p = 10$ was applied. Comparisons of the predictions and the experimental data are shown in Figs. 8(a) and (b).

For lower confining pressures even the quantitative agreement is good, while for the high confining pressure the agreement is only relatively good. From the experimental test results on the cement-treated clay, it can be seen that the experimental data shows larger strain softening behaviour and lower dilation effects than the model prediction at high confining pressures. Considering the use of a single set of parameters, the overall agreement between the experimental data and model predictions is very satisfactory. The strength and deformation of cement-treated clay and reduction of bonding effects is well described by the model.

4. Conclusions

The paper describes and evaluates a constitutive model for artificially cement-treated clay based on the critical state concepts. Unlike the classical critical state model, the proposed model applied a bonding stress ratio m and $\lambda'(p', e)$ instead of constant M_{cs} and λ, respectively, in order to account for the increase in strength due to cementation which is reduced progressively due to breaking of bonds. The behaviour resembles more or less that of over-consolidated soil and soft rock.

The simulated results of the stress-strain behaviour of a cement treated clayey soil indicate that the model requires only a single set of parameters $M_{cs}, c, \lambda', x, k_p, \nu, e_o, G, K$ to fully define the behaviour of cement treated clayey soil.

From comparisons of the predictions and the test results, it can be concluded that this simple modified model describes relatively well the main features of the behaviour of cement treated clayey soil and provides an improvement in the representation of the cementation effect of stabilized soil compared to the original Modified Cam Clay model. Quantitative agreement between model predictions and experimental results is, however, not always satisfactory. Specifically, volumetric strain of cement treated clay soil under high confining pressures does not exactly follow the associated flow rule assumed in the Modified Cam Clay model. The model has not yet to be applied to field problems and further application is required. However, as it has been shown, the proposed model can be considered as a valuable tool for describing the behaviour of cement treated clay.

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