Extended Critical State Model for Cohesive Clay with Initial Induced Anisotropy

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Abstract

In order to evaluate the stress-strain behaviour of isotropically and/or anisotropically consolidated cohesive clay, two different types of isotropic hardening critical state models and an anisotropic hardening critical state model were developed based on the critical state concept and the new assumptions of the internal work dissipated per unit volume of soils. The model proposed is based on the associated flow rule, in which a set of yield function and hardening modulus are contained. It was shown that the models had the possibility to be able to reasonably represent the cohesive and anisotropic properties in stress-strain curves for normally consolidated clay.

1. Introduction

Clayer soils which were naturally deposited are widely recognized to possess some cohesive properties characterized by cementation and bonding effects in the mechanical behaviour and an anisotropy induced by progressive deformation due to initial consolidation and subsequent loading. These properties tend to complicate the stress-strain behaviour in natural clay. There is therefore a need to develop a constitutive model for later use in evaluating the complicated behaviour in natural clay.

The aim of this paper is to present two kinds of simplified isotropic hardening elasto- plastic constitutive models and an anisotropic hardening one for normally consolidated cohesive clay under static loading condition. This will be done by extending the Modified Cam-Clay model well-known as the critical state model, which are developed by Roscoe and his colleagues[1)-3)]. The key assumption of the critical state model presented is in the assumption of the internal work dissipated per unit volume of the materials.

The model presented consists of a yield function and hardening modulus which are formulated by introducing the critical state concept and some new assumptions for the internal dissipated energy. The model contains four or five soil constants which are not only easily determined by a few conventional triaxial tests but also have a clear physical meaning. The predicted behaviour for undrained triaxial tests are compared with available experimental data. It is shown that the models accurately represent the

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changes in stress-strain behaviour for an isotropically or anisotropically consolidated clay. In this study, the compressive stresses and strains are taken as positive.

2. General stress strain increment

2.1 Stress-strain increment parameter

To specify the variation of either a yield or plastic potential surface with the state of stress, the following stress and strain-increment parameters are used in this study,

$$p=\frac{1}{3}\sigma_{ij}\,\delta_{ij} \quad ; \qquad q=\sqrt{\frac{3}{2}s_{ij}\,s_{ij}} \tag{1}$$

$$dv=d\varepsilon_{ij}\,\delta_{ij} \quad ; \qquad d\varepsilon=\sqrt{\frac{2}{3}de_{ij}\,de_{ij}} \tag{2}$$

where, p and q mean the mean principal effective stress and deviatoric stress, whose parameters associate with the first order stress and second order deviatoric stress invariant, respectively, dv and $d\varepsilon$ mean the incremental volumetric and deviatoric strain which are also related to the first order strain and second order deviatoric strain invariant, σ_{ij} and s_{ij} are stress and deviatoric stress tensor, respectively, in which $s_{ij}=\sigma_{ij}-p\delta_{ij}$, $d\varepsilon_{ij}$ and de_{ij}, which are defined as $de_{ij}=d\varepsilon_{ij}-(dv/3)\delta_{ij}$, are incremental strain and incremental deviatoric strain tensor, respectively, and δ_{ij} is Kronecker delta.

2.2 Stress-strain increment

Based on the linear incremental approach in plasticity, it is assumed that the total strain increment $d\varepsilon_{ij}$ due to a stress increment $d\sigma_{ij}$ can be divided into elastic and plastic parts as follows:

$$d\varepsilon_{ij}=d\varepsilon^{e}_{ij}+d\varepsilon^{p}_{ij} \tag{3}$$

For the isotropic case, the elastic strain increment is easily related to the stress increment as follows:

$$d\varepsilon^{e}_{ij}=\frac{1+\nu}{E}d\sigma_{ij}-\frac{\nu}{E}d\sigma_{kk}\,\delta_{ij} \tag{4}$$

where, E and ν are constants, Young's modulus and Poisson's ratio, respectively, which are also associated with the bulk modulus K and shear modulus G, such that:

$$K=\frac{E}{3(1-2\nu)} \quad ; \qquad G=\frac{E}{2(1+\nu)} \tag{5}$$

Based on the associated flow rule, the plastic strain increment $d\varepsilon^{p}_{ij}$ can be given by

$$d\varepsilon^{p}_{ij}=\Lambda\frac{\partial f}{\partial\sigma_{ij}} \tag{6}$$

where, Λ is a proportional factor and f is a yield function which is equivalent to plastic potential in the associated flow rule. When the expressions of f and Λ are precisely formulated by the function of the state of stress , the plastic strain increment is then clearly computed by using Eq. (6).

3. Formulation of each critical state model presented

3.1 Critical state condition for cohesive clay

Roscoe, Schofield and Wroth[1] and Schofield and Wroth[2] suggested that an element of soil during shear eventually reaches a critical state condition in which it can continue to deform without further change of void ratio,e , and of the effective stress q and p. Such critical state condition in q-p space is expressed by

$$q=Mp \tag{7}$$

and also that, based on the observed straight line of the consolidation curve in e-ln p plane, it is given by

$$e=\Gamma-\lambda \ln p \tag{8}$$

where, M is the slope of critical state line in q-p plane, Γ is the critical void ratio when p equals to unit pressure and λ is a soil constant defined as the slope of e-ln p virgin straight line.

Here, in order to prescribe the critical state condition of clay with an inherent cohesion such as may be mobilized by any cementation effects, the critical state line in Eq.7 is extended as follows:

$$q=M(p+p_r)=Mp^* \;; \qquad p^*=p+p_r \tag{9-a}$$

$$e=\Gamma-\lambda \ln(p+p_r)=\Gamma-\lambda \ln p^* \tag{9-b}$$

where, p_r is a soil constant which is defined by the value of p when $q=0$ as shown in Fig. 1 , and M means the value of η^* in the critical state, η^* is stress ratio defined by

$$\eta^*=\frac{q}{p+p_r} \tag{10}$$

where, it should be noted that, when $p_r=0$, Eq.9 reduces to Eq.7. The schematic view for the critical state line assumed in q-p and e-ln p plane are shown in Fig. 1 .

3.2 Work dissipation assumed and associated yield function

3.2.1 Modified Cam-Clay type

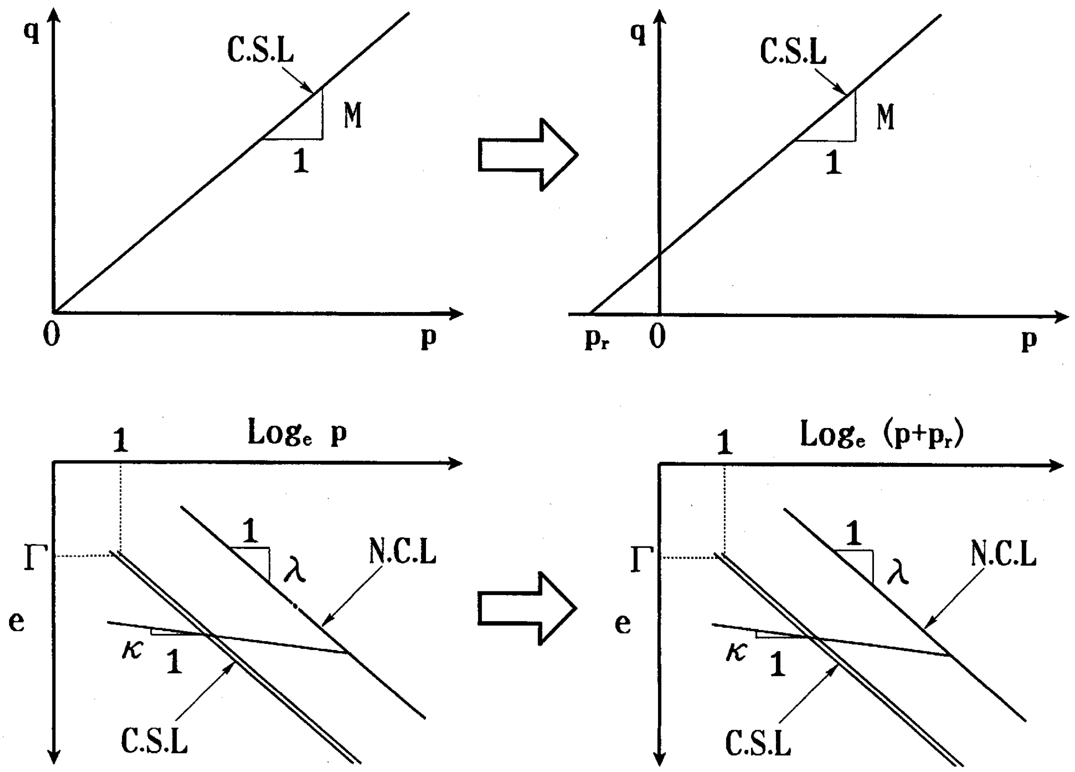

Fig. 1 Schematic diagram of critical state line assumed

A plastic work dissipation per unit volume of the material must be assumed for the specification of the yield surface in the critical state model. In order to introduce the cohesion component p_r to the yield function, the following internal dissipated work dW_{in} is presented such that:

$$dW_{in}=p^*\sqrt{(dv^p)^2+(Md\varepsilon^p)^2}-p_r dv^p \tag{11}$$

where $p^*=p+p_r$, it is important to note that when $p_r=0$ Eq.11 reduces to the dissipated work equation assumed in the Modified Cam-Clay model.

This expression may be combined with the outer work done, dW_{out}, $dW_{out}=pdv^p+qd\varepsilon^p$, to show that

$$\frac{dv^p}{d\varepsilon^p}=\frac{M^2-\eta^{*2}}{2\eta^*} \tag{12}$$

Further, applying the normality rule, namely, $dv^p/d\varepsilon^p=-dq/dp$, to Eq.12, the yield function of cohesive clay characterized by Eq.11 is easily derived as

$$f=p^{*2}-p^*_0p^*+\frac{1}{M^2}q^2=0 \tag{13-a}$$

$$\frac{\partial f}{\partial p}=p^*\left[1-\frac{1}{M^2}\left(\frac{q}{p^*}\right)^2\right]\ ;\quad \frac{\partial f}{\partial q}=\frac{2q}{M^2}\ ;\quad \frac{\partial f}{\partial p_o{}^*}=-p^* \tag{13-b}$$

where, $p_0{}^*$ is defined by $p_0{}^*=p_0+p_r$ in which p_0 is the value of p at $\eta^*=0$. The typical shape of the yield function is shown in Fig. 2, from which it is obvious that the yield function is one of the isotropic hardening type.

3.2.2 General Cam-Clay type

Here, let us consider the more general equation for the internal dissipated work based on the assumption of an isotropic hardening. Considering the cohesion for p_r in the energy equation assumed, after all, the general expression for the internal dissipated work may be presented as follows:

$$dW_{in}=p^*\sqrt{(dv^p)^2+(2-c)\eta^*dv^pd\varepsilon^p+(Md\varepsilon^p)^2}-p_rdv^p \tag{14}$$

where c is a soil constant which characterizes the $\eta^*-dv^p/d\varepsilon^p$ relationship. Based on this equation, the following $\eta^*-dv^p/d\varepsilon^p$ relationship can be derived as follows:

$$\frac{dv^p}{d\varepsilon^p}=\frac{M^2-\eta^{*2}}{c\eta^*} \tag{15}$$

Further, applying the same manner mentioned above to Eq.15, the yield functions are newly derived as

For $c\neq1$:

$$f=p^{*2}-p^{*2}\left(\frac{p^*_0}{p^*}\right)^{\frac{2(c-1)}{c}}+\frac{(c-1)}{M^2}q^2=0 \tag{16-a}$$

$$\frac{\partial f}{\partial p}=\frac{2(c-1)p^*}{c}\left[1-\frac{1}{M^2}\left(\frac{q}{p^*}\right)^2\right]\ ;\quad \frac{\partial f}{\partial q}=\frac{2(c-1)q}{M^2} \tag{16-b}$$

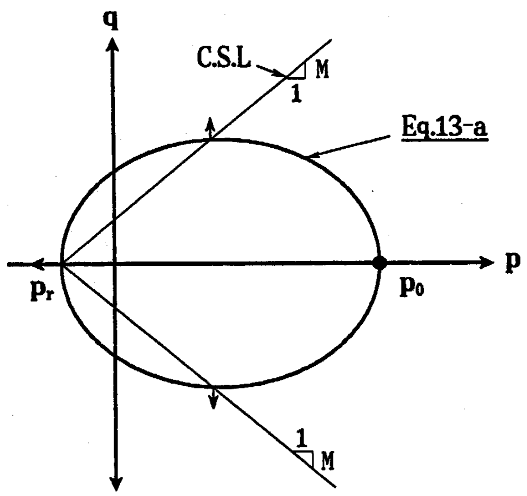

Fig. 2 Typical shape of yield curve in Modified Cam-Clay type model (MCC-Model)

$$\frac{\partial f}{\partial p_0^*}=\frac{2(c-1)p^*}{c}\left(\frac{p_0^*}{p^*}\right)^{\frac{2-c}{c}}$$

For c=1 :

$$f=q^2+2M^2p^{*2}ln\left(\frac{p^*}{p_0^*}\right)=0 \quad (17\text{-a})$$

$$\frac{\partial f}{\partial p}=2p^*\left(M^2-\left(\frac{q}{p^*}\right)^2\right) ; \quad \frac{\partial f}{\partial q}=2q \quad (17\text{-b})$$

Here, it is important to emphasize that when c=2, the dissipated work of Eq.14 reduces to that of Eq.11 (Modified Cam-Clay type), thus, Eq.14 can be considered as the expression of more general dissipated work for the critical state model of the isotropic hardening type. The effects of constant c on the shape of yield curves are shown in Fig. 3, which can be seen that the shape is remarkably different from each other.

3.2.3 Anisotropic Modified Cam-Clay type

Instead of Eq.11, the expression for the internal dissipated work of cohesive clay with triaxial anisotropy is proposed under the triaxial stress condition, such that:

$$dW_{in}=p^*\sqrt{(dv^p)^2+2\alpha\eta^*dv^pd\varepsilon^p+(Md\varepsilon^p)^2}-p_rdv^p \quad (18)$$

where, α means the triaxial non-dimentional anisotropic parameter, using tensor quantity α_{ij}, in general, α is defined by

$$\alpha=\sqrt{\frac{3}{2}\alpha_{ij}\ \alpha_{ij}} \quad (19)$$

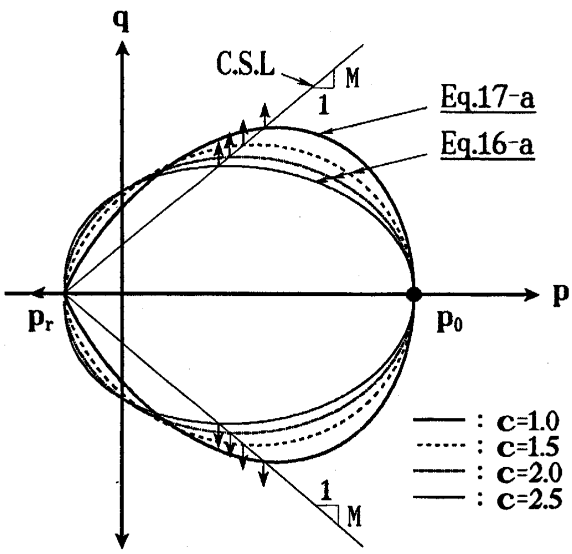

Fig. 3 Effects of constant c on the shape of yield curve in General Cam-Clay type model (GCC-Model)

where, α_{ij} is a second order dimensionless deviatoric tensor. In the case of triaxial anisotropy, $\alpha=\alpha_1-\alpha_3$, since triaxial anisotropy is transverse isotropy, namely, $\alpha_2=\alpha_3$. Noted that when $p_r=0$, Eq.18 reduces to the equation represented by Dafalias[4].

Based on Eq.18, after all, the following $\eta^*-dv^p/d\varepsilon^p$ relationship can be derived as follows:

$$\frac{dv^p}{d\varepsilon^p}=\frac{M^2-\eta^{*2}}{2(\eta^*-\alpha)} \tag{20}$$

Further, applying the same manner mentioned above to Eq.20 (see section 3.2.1), an anisotropic yield function in triaxial stress space can be expressed by

$$f=p^{*2}-p_0{}^*p^*+\frac{1}{M^2}(q^2-2\alpha p^*q+\alpha^2p^*p_0{}^*)=0 \tag{21-a}$$

$$\frac{\partial f}{\partial p}=p^*\left(1-\frac{1}{M^2}\left(\frac{q}{p^*}\right)^2\right)\ ;\ \frac{\partial f}{\partial q}=\frac{2(q-p^*\alpha)}{M^2}\ ;\ \frac{\partial f}{\partial p_0{}^*}=-p^*\left(1-\left(\frac{\alpha}{M}\right)^2\right) \tag{21-b}$$

The shape of this yield function in q-p plane is shown in Fig.4, where it should be noted that p^*_0 is chosen as the value of p at $\eta^*=\alpha$.

When an anisotropy is considered in the elasto-plastic model, the model should involve at least an internal anisotropic parameter represented by a tensor quantity to show the anisotropic stress-strain behaviour in the general stress space. Thus, it seems to be very important to generalize the expression of anisotropic yield function in triaxial space. Then, Eq.21 can be generalized by

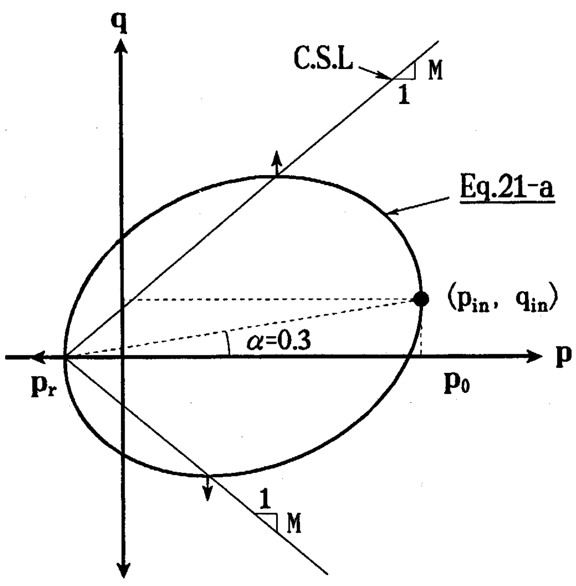

Fig. 4 Typical shape of yield curve in Anisotropic Modified Cam-Clay type model (AMCC-Model)

$$f=p^{*2}-p_0{}^*p^*+\frac{3}{2M^2}[(s_{ij}-p^*\alpha_{ij})(s_{ij}-p^*\alpha_{ij})+(p_0{}^*-p^*)p^*\alpha_{ij}\alpha_{ij}]=0 \quad (22\text{-a})$$

$$\frac{\partial f}{\partial p}=p^*\left(1-\frac{1}{M^2}\left(\frac{q}{p^*}\right)^2\right)\ ; \qquad \frac{\partial f}{\partial s_{ij}}=\frac{3}{2M^2}[2(s_{ij}-p^*\alpha_{ij})] \quad (22\text{-b})$$

$$\frac{\partial f}{\partial p_0{}^*}=p^*\left(\left(\frac{\alpha}{M}\right)^2-1\right)$$

When, in Eq.(22-a), q^* is defined as

$$q^*=\sqrt{\frac{3}{2}(s_{ij}-p^*\alpha_{ij})(s_{ij}-p^*\alpha_{ij})} \quad (23)$$

which is associated with the second order deviatoric invariant, then Eq.(22-a) is rewritten as

$$f=p^{*2}-p_0{}^*p^*+\frac{3}{2M^2}[q^{*2}+(p_0{}^*-p^*)p^*\alpha_{ij}\alpha_{ij}]=0 \quad (24\text{-a})$$

$$\frac{\partial f}{\partial q^*}=\frac{2q^*}{M^2}\ ; \qquad \frac{\partial f}{\partial p_0{}^*}=p^*\left(\left(\frac{\alpha}{M}\right)^2-1\right) \quad (24\text{-b})$$

Using the above anisotropic yield function, the yield behaviour for cohesive clay with initial induced anisotropy may be reasonably evaluated in the genaral stress space when soil constant, α_{ij}, is properly chosen.

3.3. Derivations of flow vector and hardening modulus

As mentioned in section 2.2, the plastic strain increment can be represented by Eq. 6. In Eq.6, the yield function has the following general form

$$f(\sigma_{ij},\ \kappa^p)=0 \quad (25)$$

where, k^p is the hardening parameter which represents a change in size of the yield curve.

Now, applying the consistency condition to Eq.25, the yield condition gives

$$df=\frac{\partial f}{\partial \sigma_{ij}}d\sigma_{ij}+\frac{\partial f}{\partial k^p}dk^p=0 \quad (26)$$

In the critical state model, the irrecoverable change in void ratio, namely, plastic volumetric strain is generally used as the hardening parameter. In this case, Eq.25 can be rewritten as $f=f(\sigma_{ij},\ v^p)$, therefore, for Eq.26, one has

$$df=\frac{\partial f}{\partial \sigma_{ij}}d\sigma_{ij}+\frac{\partial f}{\partial v^p}dv^p=0 \quad (27)$$

Here, from Eq.6, $dv^p = \Lambda(\partial f/\partial p)$, and substituting this relationship into Eq.27, Λ is expressed by

$$\Lambda = -\frac{\frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij}}{\frac{\partial f}{\partial v^p}\frac{\partial f}{\partial p}} = \frac{1}{H}\frac{\partial f}{\partial \sigma_{ij}} d\sigma_{ij} \tag{28}$$

and therefore, the hardening modulus H is given by:

$$H = -\frac{\partial f}{\partial v^p}\frac{\partial f}{\partial p} = -\left(\frac{\partial f}{\partial p_0^*}\frac{\partial p_0^*}{\partial e^p}\frac{\partial e^p}{\partial v^p}\right)\frac{\partial f}{\partial p} \tag{29}$$

In addition, substituting Eq.(28) into Eq.(6), consequently, the plastic strain increment $d\varepsilon_{ij}{}^p$ can be rewritten as follows:

$$d\varepsilon_{ij}{}^p = \frac{1}{H}\frac{\partial f}{\partial \sigma_{ij}}\left(\frac{\partial f}{\partial \sigma_{ij}}\right) d\sigma_{ij} \tag{30}$$

Therefore, in order to calculate the plastic strain increment based on Eq.30, the flow vector $\partial f/\partial \sigma_{ij}$ and hardening modulus H must be expressly determined as the function of state of stress.

Now, for simplicity, in this study, we consider the Drucker-Prager failure/yield surface as the useful failure or yield criterion in π plane, namely, it means that both crirical state condition and yield function are independent on the third deviatoric stress invariant.Then, each yield function for the case of an isotropic hardening (Eqs.13, 16 and 17) and for the case of an anisotropic hardening (Eq.22 or 24) has the following general form, respectively,

For case of isotropic hardening (Eqs.13, 16 and 17):

$$f(\sigma_{ij}, \varkappa^p) = f(p^*, q, \varkappa^p) = 0 \tag{31-a}$$

For case of anisotropic hardening (Eq.23):

$$f(\sigma_{ij}, \varkappa^p) = f(p^*, q^*, \varkappa^p) = 0 \tag{31-b}$$

Using the above relationships, the flow vector can be represented as

For case of isotropic hardening (Eqs.13, 16 and 17):

$$\frac{\partial f}{\partial \sigma_{ij}} = A_1\frac{\partial p^*}{\partial \sigma_{ij}} + A_2\frac{\partial q}{\partial \sigma_{ij}} \tag{32-a}$$

$$A_1 = \frac{\partial f}{\partial p^*}; \qquad A_2 = \frac{\partial f}{\partial q}$$

For case of anisotropic hardening (Eq.23):

$$\frac{\partial f}{\partial \sigma_{ij}} = B_1 \frac{\partial p^*}{\partial \sigma_{ij}} + B_2 \frac{\partial q^*}{\partial \sigma_{ij}} \tag{32-b}$$

$$B_1 = \frac{\partial f}{\partial p^*}\ ; \qquad B_2 = \frac{\partial f}{\partial q^*}$$

where, the partially derivative $\partial p^*/\partial \sigma_{ij}$, $\partial q/\partial \sigma_{ij}$ and $\partial q^*/\partial \sigma_{ij}$ are defined as

$$\frac{\partial p}{\partial \sigma_{ij}} = \frac{1}{3}\delta_{ij} \tag{33}$$

$$\frac{\partial q}{\partial \sigma_{ij}} = \frac{3}{2q} s_{ij}\ ; \qquad \frac{\partial q^*}{\partial \sigma_{ij}} = \frac{3}{2q^*}\left\{ (s_{ij} - p^* \alpha_{ij}) - \frac{1}{3}(s_{kl} - p^* \alpha_{kl}) \alpha_{kl} \delta_{ij} \right\}$$

Here, it is important to emphasize that when $\alpha_{ij}=0$, $\partial q^*/\partial \sigma_{ij}$ reduces to $\partial q/\partial \sigma_{ij}$ in the isotropic case, and also that the flow vector is remarkably dependent on the yield function assumed, in other words, the assumption of the internal plastic work dissipated.

Now, the rest work to complete the model is to represent the hardening modulus H in Eq.29, namely, is to give the $\partial p_0^*/\partial e^p$ and $\partial e^p/\partial v^p$ in H, concretely. According to the critical state concept, as shown in Eq.8 which presents the e-ln p* linear relationship during the proportional loading, the irrecoverable change in void ratio e, de^p can be expressed as follows:

$$de^p = -(\lambda - \kappa)\frac{dp_0^*}{p_0^*} \tag{34}$$

where λ and κ are soil constants, which are denoted as the slope of the e-ln p* virgin loading and unloading/reloading line, respectively. It is obvious that Eq.34 can be rewritten as follows:

$$\frac{dp_0^*}{de^p} = \frac{\partial p_0^*}{\partial e^p} = -\frac{p_0^*}{\lambda - \kappa} \tag{35}$$

In addition, since the relationship between dv^p and de^p is given by

$$dv^p = -\frac{de^p}{1+e} \tag{36}$$

the term $\partial e^p/\partial v^p$ in Eq.29 becomes

$$\frac{\partial e^p}{\partial v^p} = -(1+e) \tag{37}$$

Thus, substituting Eqs.35 and 37 into Eq.29, hardening modulus H in the critical state model is precisely defined as

$$H=-\left(\frac{1+e}{\lambda-\kappa}p_0^*\right)\frac{\partial f}{\partial p_0^*}\frac{\partial f}{\partial p} \tag{38}$$

where, the terms $\partial f/\partial p_0^*$ and for each yield function are already shown in Eqs.13, 16, 17 and 22, respectively.

In these discussions, the expressions of the elastic and plastic strain increment (Eqs. 4 and 30) can be precisely derived as the function of the state of stress, based on the newly assumed critical state concept and internal dissipated work.

4. Applicability

4.1. Soil constants

Table 1 indicates the soil constants of Itsukaichi clay for each model, which is a typical high plasticity marine clay. The index properties are shown as specific gravity, $G_s=2.532$, plastic limit, $w_p=51.4\%$, liquid limit, $w_L=124.2\%$ and plasticity index, $I_p=72.8\%$[5]. It is well known that λ, κ, M and e_{in} are often called critical state paremeters, and constants p_r, α and c are the critical state paremeters newly added in this study. Among these, λ and κ are defined as the slope of e-ln p* proportional loading and swelling curves, which can be easily obtained from an isotropically consolidated test (see Fig.1), and M is given by the slope of critical state line in q-p* plane obtained from a few undrained triaxial compression tests. Constants p_r and c are also determined from a few triaxial undrained tests, in which p_r is clearly assosiated with the cohesion compornent C' and internal friction angle ϕ', that is, $p_r=C'\tan\phi'$ and a soil constant c is chosen to fit the experimental shape of undrained stress path or stress-dilatancy curve, and then α is, for simplicity, represented by the value of η^* in the initial

Table 1 Soil constants for Itukaichi clay

	M. C. C.	G. C. C.	A. M. C. C.
λ	0.443		
κ	0.052		
M	1.560		
e_{in}	2.000		
p_r	(※)	(※)	(※)
α	———	———	η_{in}
C	———	(※)	———

※ Constants C and p_r are arbitrarily chosen based on the cohesion and stress-dilatancy property of clay, respectivery.

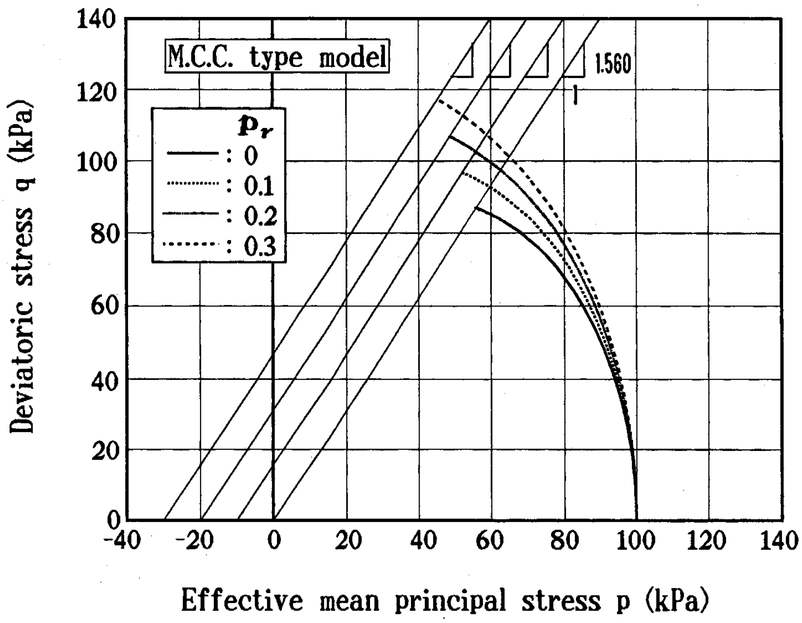

Fig. 5 Effects of constat pr on the undrained stress path in triaxial compression tests based on the MCC-Model

consolidation process. Further, constant e_{In} is defined as the value of e at the unit pressure in the isotropic normally consolidated curve.

4.2. Prediction of undrained triaxial tests

Fig.5 shows the effects of constant p_r on the undrained stress pathfor Itsukaichi clay based on the Modified Cam-Clay type model (MCC), where, except for p_r, the same soil constants in Table 1 are used in the each prediction. It can be seen that the lager p_r is, more stiff undrained stress path tends to become. Then, the effect of constant c on the undrained stress path is investigated using General Cam-Clay type model (GCC). The predicted undrained stress paths are shown in Fig.6, together with the experimental undrained stress path after being isotropically consolidated. It is found that the predicted stress path is strongly dependent on the constant c. The applicability of Anisotropic Modified Cam-Clay type model (AMCC) to anisotropically consolidated Itsukaichi clay is finally investigated by using the results of undrained triaxial tests. Figs.7(a) and (b) show the experimental and predicted undrained stress paths, respectively. These predicted results seem to reasonably describe the anisotropic property of each experimental result.

5. Conclusion

Based on the new assumption of the extended critical state concept and internal works dissipated per unit volume of soils, two types of isotropic hardening critical state madels and an anisotropic hardening critical state model have been presented to evaluate the mechanical behaviour of cohesive clay. The derived process of the model

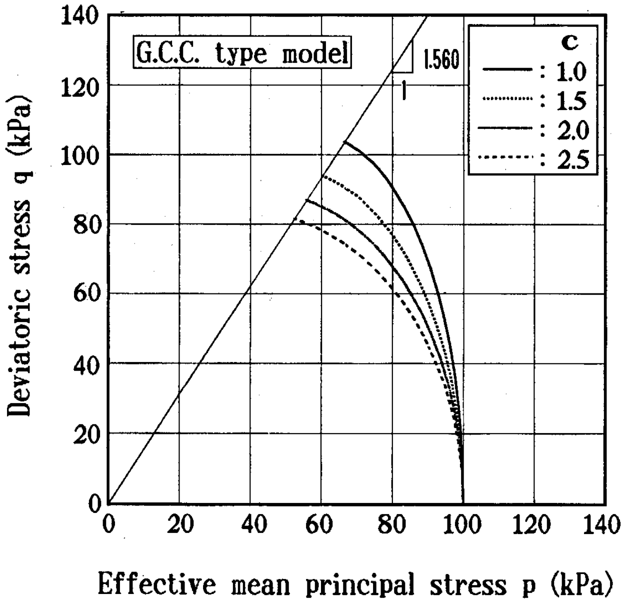

Fig. 6 Effects of constant c on the undrained stress path in triaxial compression tests based on the GCC-Model

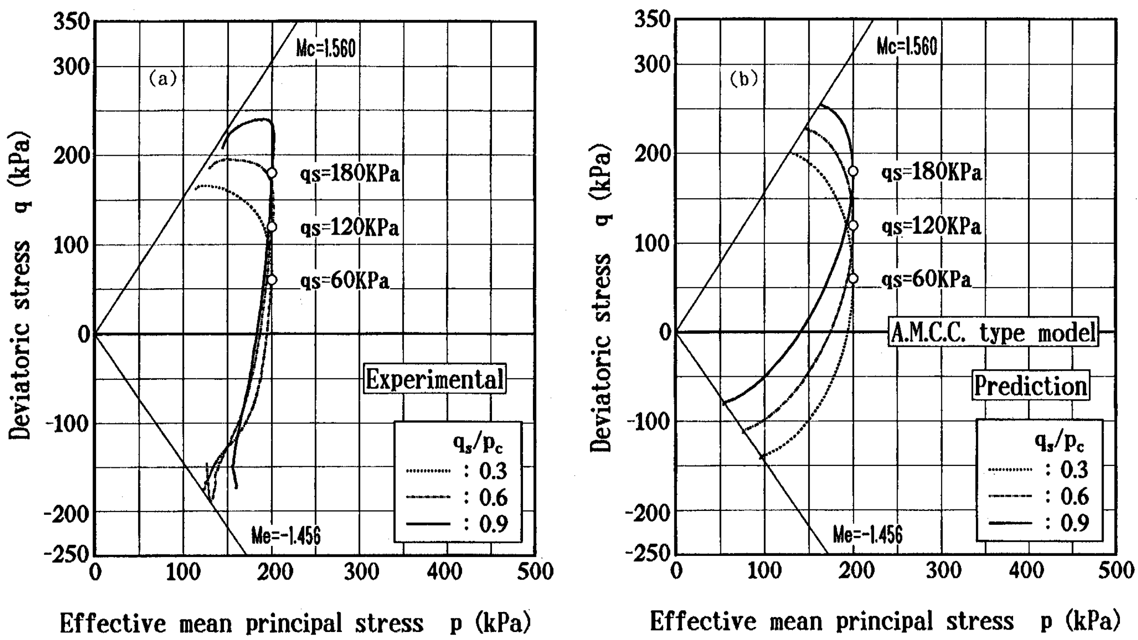

Fig. 7 Undrained stress paths of anisotropically consolidated clay in triaxial compression tests; (a) Experimental results, (b) Predicted results based on the AMCC-Model

is simple and clear, and all the parameters are easily determined from a few triaxial tests. The effects of newly added parameters on the undrained stress path have been clarified for typical high plasticity marine clay.

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