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Elasto-viscoplastic modeling of geomaterials

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ABSTRACT
It is well known that constitutive models of geomaterials play a very important role in the Computational Geotechnics.
After Cam-clay model was extablished, many constitutive models have been developed. Since the behavior of geomaterials is rate dependent, it is preferable to use rate dependent models such as viscoplastic model to the bahvior of geomateials. The time-dependent behavior of soil, manifested as creep, relaxation, and rate sensitivity, comprises indispensable factors for predicting the long-term settlement behavior of soft clay deposits, slope stability, and landslides. In the present report, rate-dependent models, such as viscoelatic and viscoplasticity models are overviewed.

1 VISCOELASTIC CONSTITUTIVE MODELING

The modeling of many kinds of materials has been carried out within the framework of viscoelasticity for polymers, metals, concrete, soil, and rock. The well-known linear models are the Maxwell model, the Voigt-Kelvin model, and the spring-Voigt three-parameter model. It has been reported that the linear spring-Voigt model can describe the dynamic nature of soil (Kondner and Ho, 1965; Hori, 1974). By introducing the concept of the distribution of relaxation time into the linear model, it is possible to model the wide range of time-dependent behavior of soil. Murayama and Shibata (1964) have proven the time dependency of clay in high-frequency regions by considering the distribution of relaxation time. Murayama (1983) proposed non-linear viscoelastic and viscoplastic models based on the original model (Murayama and Shibata, 1964; Sekiguchi, 1977).

Di Benedetto et al. (1997) proposed a simple asymptotic body (SAB) for the simplification of the viscoelastic model for soil which can be classified into three-parameter models. In the range of small strain, the linear viscoelastic approach is valid. However, in the range of large strain, the features include both viscoelasticity and visoplasticity. Oka, Kodaka and Kim (2004e) have succeeded in describing the behavior by a viscoelastic-viscoplastic model for clay which can explain the dynamic behavior of clay for a wide range of strain levels.

1.1 *Microrheology models for clay*

The viscous behavior of clay has been analyzed in the field of Microrheology. Murayama and Shibata (1964) applied the rate process theory by Eyring (1936) to clay and derived a rheological model. Then, Singh and Mitchell (1968, 1969), and Mitchell, Singh and Campanella (1968) successfully described the creep behavior of clay based on the rate process theory. Using the rate process theory, an exponential type of non-linear flow law, between the shear force acting on each flow unit and the strain rate when the shear force is found to be larger than the thermal energy, was created.

2 ELASTO-VISCOPLASTIC CONSTITUTIVE MODELING

2.1 *Overstress models*

To describe both the viscous nature and the plastic nature of soil, viscoplastic modeling is necessary. Perzyna (1963) proposed a viscoplastic theory which generalizes the linear theory for the viscoplasticity theory by Hohenemser and Prager (1932). Hohenemser and Prager(1932)'s model, a linear extended viscoplastic model, is based on Bingham's fluid and plasticity model.

Yong and Yapp (1969) indicated the possibility of applying the viscoplasticity theory to the dynamic behavior of clay. Then, Adachi and Okano (1974) first proposed an elasto-viscoplastic theory for clay based on Perzyna's theory and the original Cam-clay model (1963). They assumed that the hardening parameter is an axial strain. They showed that viscoplasticity is an applicable theory to the rate-dependent behavior of water-saturated clay. However, a quantitative description had not yet been successively given with the model. Oka (1981), Adachi and Oka (1982a)'s newly proposed elasto-viscoplastic model is based on both Perzyna's model and Cam-clay model. It incorporates the assumption that in the stress state, after the completion of consolidation, the soil has not yet reached the equilibrium state, but has still been in a non-equilibrium state with the strain-hardening parameter of the inelastic volumetric strain, although the inelastic void ratio has been taken as a hardening parameter in Cam-clay model. The model is capable of describing the rate sensitivity, the creep, and the relaxation of cohesive soil, in particular, the volumetric relaxation behavior reported by Arulanandan et al. (1971).

The model is a rigorous combination of two theories, namely, Cam-clay model and Perzyna's model. However, the model has a shortcoming, namely, it cannot describe conventional accelerated creep behavior, i.e., creep failure. Professor S.Sture of the University of Colorado pointed out this shortcoming at the Int. workshop in Grenoble (Oka, 1982). Aubry (1985) experimentally showed that the critical state line is not rate sensitive. It can be understood that the rate dependency fades out at the critical state. Giving consideration to the rate independency at the critical state leads to the fact that the viscosity asymptotically becomes zero when approaching the critical state. Following the above point, Adachi, Oka and Mimura (1987) constructed an improved viscoplastic model by considering the variations in viscosity. The derived model is very capable of describing creep failure, i.e., accelerating creep behavior. The prediction obtained through this model indicates that the drop in stress is rather small in comparison to the experimental evidence on sensitive clay and natural soil during strain softening. During the strain-softening behavior of natural clay, it is observed that strain softening follows a rather large decrease in the mean effective stress. This indicates that the soil exhibits both shear and volumetric softening. In order to incorporate these features, a new model has been developed considering the degradation of soil structures and rate dependency by Kimoto (2002) and Kimoto, Oka and Higo (2004). This new viscoplastic model will be introduced in the following section.

Many other models have been proposed to describe the time-dependent behavior of soil. For the overstress models, Dafalias (1982), Katona (1984), Baladi and Rohani (1984), and Zienkiewicz et al. (1975) have proposed elasto-viscoplastic models within the framework of an overstress type of theory. Another type of overstress model has been proposed by Duvaut and Lions (1976). Although their model is linear overstress type of model, Duvaut and Lions's model is advantageous in that the plasticity model can easily be transferred into a viscoplastic one using the projection rule. Phillipes and Wu (1973)'s model is a non-linear viscoplastic model using a similar projection technique to obtain the overstress. Sawada et al. (2001) proposed a Cosserat viscoplasticity model for clay.

2.1.1 *Time-dependent viscoplastic model*

Sekiguchi (1977) proposed an elastic-viscoplastic model that clearly includes real time. Sekiguchi's model was originally proposed as a creep model which included failure. Nova (1982), Dragon and Mroz (1979), and Matsui and Abe (1985) derived time-dependent models which are called non-stationary models. It should be pointed out that these models include time and explicitly violate the principle of objectivity. Yin and Graham (1999) proposed an elasto-viscoplastic model based on the modified Cam-clay model and the flow surface.

2.1.2 *Viscoplastic model based on the stress history tensor*

Oka (1985) proposed a viscoplastic model with the stress history tensor which is based on the assumption that the state of materials depends on the stress and the stress history. He assumed that the yield function depends on the stress history tensor and not on just the current stress or the internal variables. The stress history tensor is given by the convolution integral of the stress tensor with respect to the generalized time measure which is inherent to the materials. Oka and Adachi (1985) developed an elasto-viscoplastic model using the stress history tensor for the analysis of the strain softening behavior of soft rock, and of frozen sand (Adachi et al., 1990; Oka et al., 1994c), and generalized it as the viscoplastic model (Adachi and Oka, 1995; Adachi et al., 2003, Adachi et al., 2005). This type of model can be applicable to the rate independent behavior adopting a special timemeasure for defining the stress history tensor. The application of the model is discussed in chapter 3.10.3.

2.2 *Adachi and Oka model*

Oka (1981), Adachi and Oka (1982a) developed an elasto-viscoplastic constitutive model for clay based on Cam-clay model and an overstress type of viscoplastic theory (Perzyna, 1963). The important assumption taken in the derivation of the model is that "At the end of consolidation, the state of the clay does not reach the static equilibrium state but is in a non-equilibrium state.

It is assumed that the strain rate tensor consists of elastic strain rate tensor $\dot{\varepsilon}_{ij}^{e}$ and viscoplastic strain rate tensor $\dot{\varepsilon}_{ij}^{vp}$, such that The elastic strain rate is given by a generalized Hooke type of law.

The viscoplastic flow rule is given by

$$\dot{\varepsilon}_{ij}^{vp} = \gamma \left\langle \Phi_1(F) \right\rangle \frac{\partial f}{\partial \sigma'_{ij}}, \qquad F = \frac{f - \kappa_s}{\kappa_s} \tag{2.4}$$

$$\left\langle \Phi_1(F) \right\rangle = \begin{cases} \Phi_1(F) & : F > 0 \\ 0 & : F \le 0 \end{cases} \tag{2.5}$$

$\dot{\varepsilon}_{ij}^{vp}$ is the viscoplastic strain rate tensor, γ is the viscosity parameter, σ_{ij} is the total stress tensor, σ'_{ij} is Terzaghi's effective stress tensor, f is the dynamic yield function, δ_{ij} is Kronecker's delta, Φ_1 is a material function which accounts for the strain rate sensitivity, $\langle \ \rangle$ is Macaulay's bracket, $F = 0$ denotes the static yield function, and κ_s is the hardening parameter.

2.3 *The extended viscoplastic model considering stress ratio dependent softening*

As mentioned above, Adachi, Oka and Mimura (1987) extended the original model to describe the acceleration creep behavior of clay by introducing a second material function into the model. Oka et al. (1994d, 1995b) studied the instability of the extended model during the undrained conventional creep process and strain localization analysis.

The second material function is introduced to explain that the rate dependency of clay vanishes at the failure state. In other words, the stress ratio at the failure state does not depend on the strain rate.

2.4 *Elasto-viscoplastic model for cohesive soil considering degradation*

The prediction by the extended model with the second material function (Adachi et al., 1987) indicates that the drop in stress is rather small compared with the experimental evidence on sensitive clay and natural soil during strain softening. During the strain-softening behavior of natural clay, it is observed that strain softening follows a rather large decrease in mean effective stress. This indicates that the soil exhibits both shear and volumetric softening. In order to incorporate these features, a new model has been developed considering the degradation of soil structures and the rate dependency by Kimoto (2002), Kimoto and Oka (2004), and Kimoto and Oka (2005). The model was applied to Osaka Pleistocene clay, namely, Kyuhoji clay whcih was sampled from the upper Pleistocene layer called Ma12. It was confirms that the proposed model can describe the difference in behavior between the highly structured and the lowly structured soil.

2.5 *Cyclic elasto-visoplastic model*

We need a cyclic plasticity model for the dynamic analysis of clay. For that purpose, we have developed a cyclic viscoplasticity model with a non-linear kinematic hardening model (Oka, 1992) and a cyclic viscoelastic-viscoplastic model with a kinematic hardening model (Oka, Kodaka and Kim, 2003). The models have been successively applied to the dynamic analysis of the ground during earthquakes, considering liquefaction, and shown in the last chapter (Oka et al., 2003b). The cyclic elasto-viscoplastic model was applied to undrained triaxial tests with step-changed strain rates. It was found that the model can describe the isotaches characteristics well (Oka et al., 2003a).

2.6 *Elasto-viscoplastic model for unsaturated soil*

For unsaturated soil, we have applied the viscoplastic model considering the effect of suction and extended viscoplastic parameter (Kim, Kimoto, Oka and Kodaka, 2005) to the behavior of unsaturated silt by Cui and Delage(1996).

2.6.1 *Application to Consolidation analysis*

It is well known that there are two types of time-dependent behavior for soil. One is consolidation and the other is brought about by the inherent viscous nature of the soil skeleton. The interaction between the pore water and the soil skeleton results in consolidation. The viscous properties of the soil skeleton are related to the microstructure of the soil particles. Although many problems due to the consolidation of various types of soil have been solved, some problems still exist. One of them is the interaction between the viscosity and the changes in the soil structure. In the following, two problems will be discussed. One is the influence of the soil specimen thickness on consolidation and the other is the interaction between the viscoplastic properties and the strain softening due to structural changes.

2.6.2 *Effect of sample thickness*

It has been reported that the influence of the specimen thickness on consolidation plays an important role in the prediction of the actual settlement due to the consolidation (e.g. Ladd et al., 1977; Aboshi, 1973; Aboshi and Matsuda, 1981; Oka et al. 1986; Leroueil, 1995; Mesri et al., 1995; Tang and Imai, 1995, etc.). As is well known, in the general report for the 9th ICSMFE, Ladd et al. (1977) showed two hypotheses for consolidation behavior by compiling the previous results (Fig. 1). Curve A is supported by Ladd et al. (1977) and Mesri and Rokhsar (1974). Curve B is based on the hypothesis that there is a unique strain-time relationship with respect to time-dependent characteristics and that creep deformation occurs from the beginning of the consolidation. Curve C is between Curves A and B, and appears to correspond to the experimental results of Aboshi (1973, 1995, 2004).

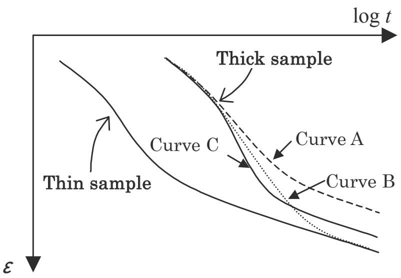

Figure 1 Schematic diagram of the average strain to time.

Many researchers have reported that the behavior which appears to be associated with the collapse of the soil structure can be recognized during the consolidation process. Bishop and Lovenbury (1969) conducted constant stress creep tests under drained conditions on undisturbed clay, and observed a sudden increase in the strain rate. Concerning field cases, the anomalous pore-pressure behavior during the consolidation of soft clay has been reported by many researchers. Mitchell (1986) reported that pore-pressure stagnation or a continuous increase after all the fill placement occurred because of a structural breakdown during compression. Furthermore, Leroueil (1988) and Kabbaj et al. (1988) observed increases in the pore water after the completion of the construction of test embankments, reflecting the fact that the effective stress temporarily diminished in the stress-strain curve. The prediction of these phenomena by numerical methods has been studied since the 1980s. Kabbaj et al. (1985) analyzed one-dimensional creep tests by the finite difference method using an elasto-viscoplastic constitutive model (Oka, 1981). They showed that the strain rate remained momentarily constant during creep simulations around the preconsolidation pressure.

Aboshi (1973) experimentally observed that the initial strain rate for the thick sample is lower than that for the thin one. Fig. 2.5 illustrates the consolidation curves obtained by loads (19.6 - 78.4 kPa) for samples with different thicknesses (Aboshi 1973).

As shown in the above, the initial strain rate of the thick sample just before consolidation is smaller than that of the thin sample, although the strain just after the preparatory consolidation is almost equal. The reason for the difference in strain rates is that the periods of preconsolidation are different, namely, from 1 day for the thinner sample to 4 months for the thicker one. Oka and Kimoto(2004) and Kimoto and Oka(2005) found that the effect of the sample thickness on the time-settlement curve is mainly due to the difference in strain rates before consolidation. This explanation was first pointed by Oka, Adachi and Okano (1986) and was confirmed by Tang and Imai (1995). Of course there might be other reasons such as sample disturbance and the natural variability (Leroueil, 1995; Mesri and Choi, 1985).

2.6.3 *Effect of degradation*

Stagnation and/or an increase in the pore-water pressure after loading and during the consolidation of soft clay is called "anomalous pore pressure" and it has been observed after loading and during consolidation by Professor J.K. Mitchell in his 20th Terzaghi Lecture (1986). This problem has hitherto been studied, but it has not yet been fully solved. Asaoka (2003) tried to analyze a similar problem by an elasto-plastic approach considering the degradation of materials. We would say that the reason is the rate-dependent structural degradation of soft clay, as discussed above. Oka, Leroueil and Tavenas (1991) numerically analyzed such a phenomenon observed in the clay foundation at St. Alban's test embankment D using an elasto-viscoplastic model. They used a model with volumetric strain softening and a comparatively better solution than the conventional model. However, the stagnation or the temporary increase in pore-water pressure after the construction of the embankment could not be reproduced. In this section, we have analyzed the same problem using an elasto-viscoplastic model with the above-mentioned structural degradation (Karim et al., 2005).

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