VERIFICATION OF THE GENERALIZED ELASTOPLASTIC MODEL FOR UNSATURATED SOILS

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ABSTRACT

We have already examined about the theoretical aspects of constitutive models for unsaturated soils. From the results, it was recognized that the consideration of two suction effects was necessary to explain the mechanical behavior of unsaturated soils. Here a more general version elastoplastic model with these two suction effects is proposed. The schematic estimations of volume change behavior of unsaturated soils were performed using this model. The verification of the model was also examined simulating typical element test series. The model could well express them.

Key words: capillary phenomena, constitutive equation of soil, effective stress, plasticity, unsaturated soil (IGC: D5/D6/E13)

INTRODUCTION

In our paper (Kohgo et al., 1993), the mechanical behavior of unsaturated soils was interpreted from micro and macro view points. As the results, we recognized that the consideration of two suction effects was necessary to explain the mechanical behavior of unsaturated soils. The two suction effects are as follows. One is that an increase in suction induces an increase in effective stress. The other is that an increase in suction increases both yield stress and the stiffness of soil skeleton against plastic deformations. A simple version of elastoplastic model with the two suction effects has been proposed.

In this paper, we will propose a more general elastoplastic model. The model with

subloading surface includes the term of the third deviator stress invariant. First, typical volume change behavior of unsaturated soils is interpreted using this model to understand schematically the performance. Next, the verification of the model is examined simulating typical element test series.

GENERALIZED ELASTOPLASTIC MODEL

Review of Suction Effects

Before we review about suction effects, suction is necessary to be defined. Here, Matrix suction is only called suction. Suction s is defined as

$$s = u_a - u_w \quad (1)$$

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where u_a is pore air pressure and u_w pore water pressure.

It is also necessary to define the effective suction s^* , since suction effects are postulated as the functions of s^* . The effective suction is

$$s^* = \langle s - s_e \rangle \quad (2)$$

where s_e is air entry suction and the brackets $\langle \rangle$ denote the operation $\langle z \rangle = z$ at $z > 0$ and $\langle z \rangle = 0$ at $z \leq 0$.

According to our classification (Kohgo et al., 1993), three saturation conditions; insular air saturation, fuzzy saturation and pendular saturation are necessary to be taken account. In insular air saturation, the mechanical soil behavior can be explained in terms of effective stresses, the same as those estimated by Terzaghi's effective stress equation for completely saturated soils. In pendular saturation, an increase in suction only increases capillary force which induces increases of both the stiffness of the soil skeleton against plastic deformations and yield stress. In fuzzy saturation, both suction effects seen in insular air and pendular saturation conditions can occur. The mechanical behavior is namely affected by following 4 points.

- (1) Effective stress increases with an increase in suction in saturated zone.
- (2) The saturation zone decreases with an increase in suction.
- (3) Capillary force increases with an increase in suction in unsaturated zone.
- (4) Internal confinement, that is, the inhibition of overall deformations of the soil caused by the Point (3) becomes greater as suction increases.

Where effective stress means the same as that defined in saturated soils (Kohgo et al., 1993).

If the Point (3) is not superior to the applied external forces, the uniform soil behavior may occur and it may express using stresses defined in the continuum mechanics. Comparing shear strength between saturated and unsaturated soils in the uniform shear behavior, we may consider that the extra confining stresses applies in the unsaturated soil. These extra confining stresses are the effective confining stresses and may be regarded as the effective

stresses. The effective stresses can namely be defined as follows. When the effects concerned with Points (1) ~ (3) can be regarded as the effect of an increase in effective confined stresses, the effective confined stresses will be defined as effective stresses. Effective stress equations are assumed to be formulated using a relationship between shear strength of soil and suction on a lightly overconsolidated region. According to the experimental results (Maswoswe, 1985), it seems to be reasonable to define the following empirical effective stress equations.

$$\sigma' = \sigma - u_{eq} \quad (3)$$

$$u_{eq} = u_a - s \quad (s \leq s_e) \quad (4)$$

$$u_{eq} = u_a - \left(s_e + \frac{s_c - s_e}{s^* + a_e} s^* \right) \quad (s > s_e) \quad (5)$$

where u_{eq} is the so called equivalent pore pressure (Nakase, 1969), a_e a material parameter and s_c the critical suction.

The effect of Point (4) expands the yield surface and increases the stiffness of soil skeleton at the plastic range. An increase in suction namely increases both the stiffness of the soil skeleton against plastic deformations and yield stress. Such an effect can be estimated formulating the state surface which defines the state boundary of volume change behavior in unsaturated soils. The state surface can be plotted in the space with the axes, effective mean stress $\log p'$, effective suction s^* , and void ratio e (see Fig. 1). If the relationships between e and $\log p'$ are linear and the $s^* \sim e$ curves fit into the family of hyperbolic equations, they will be expressed as follows.

$$e = -\lambda^* \log p' + \Gamma^* \quad (6)$$

$$\lambda^* = \frac{\lambda}{1 + y} \quad (7)$$

$$\Gamma^* = \frac{\Gamma + e_0^0 y}{1 + y} \quad (8)$$

$$y = \left(\frac{s^*}{a_s} \right)^{n_s} \quad (9)$$

$$p' = \frac{1}{3} \sigma'_{ii} \quad (10)$$

where λ and Γ are respectively λ^* and Γ^* at

Fig. 1. The generalized state surface

saturation, a_s and n_s the material parameters, σ'_{ij} is current effective stress tensor and repeated indices indicate summation.

Formulation of Normal Yield Surfaces

In the following, compression stresses are postulated to be negative. The elastoplastic model proposed here, which is generalized taking account of a stress invariant J_3 on the base of the elastoplastic model proposed by Tanaka (1979), has two yield surfaces, as illustrated in Figs. 2 and 3. One is the Mohr-Coulomb type yield condition which defines the failure surface, that is, the Hvorslev surface. The other is the elliptical cap model with corners which defines the Roscoe surface. Both are connected on the critical state locus. The critical state line and these yield surfaces are defined as follows.

$$f = \alpha_{cs}^* I_1 + \frac{\sqrt{J_2}}{g(\theta)} = 0 \quad (11)$$

$$f_1 = \alpha^* I_1 + \frac{\sqrt{J_2}}{g(\theta)} - K^* = 0 \quad (I_0 \leq I_1 \leq P_2) \quad (12)$$

$$f_2 = b^{*2} (I_1 - I_0)^2 + a^2 \frac{J_2}{g(\theta)^2} - a^2 b^{*2} = 0 \quad (I_c \leq I_1 \leq I_0) \quad (13)$$

where θ is the Lode angle, I_1 first stress invariant, J_2 second invariant of deviator stress and J_3 third invariant of deviator stress. They

Fig. 2. Yield surfaces of the proposed elastoplastic model at $-I_1 \sim \sqrt{J_2}$ section

Fig. 3. Yield surfaces of the proposed elastoplastic model in the principal stress space

are defined as,

$$I_1 = \sigma'_{ii} \quad (14)$$

$$J_2 = \frac{1}{2} S_{ij} S_{ij} \quad (15)$$

$$J_3 = \frac{1}{3} S_{ij} S_{jk} S_{ki} \quad (16)$$

$$S_{ij} = \sigma'_{ij} - \frac{1}{3} \delta_{ij} I_1 \quad (17)$$

$$\sin 3\theta = \frac{-3\sqrt{3}J_3}{2J_2^{1.5}} \quad (18)$$

where δ_{ij} is the Kronecker delta.

α_{cs}^* , α^* , a and b^* are defined in Fig. 2. α_{cs}^* , α^* and $g(\theta)$ are expressed as follows.

$$\alpha_{cs}^* = \frac{2 \sin \phi'_{cs}}{\sqrt{3} (3 - \sin \phi'_{cs})} \quad (19)$$

$$\alpha^* = \frac{2 \sin \phi'}{\sqrt{3}(3 - \sin \phi')} \quad (20)$$

$$g(\theta) = \frac{(3 - \sin \phi')}{2(\sqrt{3} \cos \theta - \sin \theta \sin \phi')} \quad (21)$$

where ϕ'_{cs} is the angle of internal friction of the critical state line and ϕ' the angle of internal friction of the failure line.

K^* , a and b^* relate uniquely with I_0 . I_c can be expressed using I_0 as

$$I_c = (1 + R)I_0 \quad (23)$$

where R is a material parameter, like the aspect ratio. I_c is the yield stress of this model. If this is an isotropic hardening model with plastic volumetric strain ε_v^p as a hardening parameter, I_c may be estimated by means of the state surface concept (see Ref. 4).

$$I_c = -3 \exp \left(\frac{B^* - \varepsilon_v^p}{A^*} \right) \quad (24)$$

$$A^* = \frac{(\lambda^* - \kappa)}{2.3(1 + e_0)} \quad (25)$$

$$B^* = \frac{(\lambda_0^* - \kappa)}{2.3(1 + e_0)} \ln(-p'_0) - \frac{(\Gamma_0^* - \Gamma^*)}{(1 + e_0)} \quad (26)$$

Here, κ is the slope of $e \sim \log(-p')$ curve at unloading, e_0 the initial void ratio, p'_0 the initial mean effective stress and λ_0^* and Γ_0^* the initial values of λ^* and Γ^* , respectively.

The following two elliptical plastic potential functions are adopted here.

$$\psi_1 = b^2(I_1 - I_0)^2 + a^{*2}J_2 - a^{*2}b^2 = 0 \quad (I_0 \leq I_1 \leq P_2) \quad (27)$$

$$\psi_2 = b^2(I_1 - I_0)^2 + a^2J_2 - a^2b^2 = 0 \quad (I_c \leq I_1 \leq I_0) \quad (28)$$

where $a^* = P_2 - I_0$, $b = -\alpha_{cs}I_0$, and α_{cs} is defined to coincide with the Mohr-Coulomb criterion using ϕ'_{cs} (Zienkiewicz and Humpheson., 1977).

Formulation of Subloading Surfaces

The model proposed here belongs to the original subloading surface model (Hashiguchi, 1980). Subloading surfaces are defined to be similar to the normal yield surfaces (see Fig. 4). The similarity center is postulated to coincide with the origin.

Fig. 4. Normal yield and subloading surfaces at $-I_1 \sim \sqrt{J_2}$ section

The ratio of the sizes of the subloading surfaces to the normal yield surfaces is expressed as

$$\bar{R} = \frac{\bar{I}_c}{I_c} \quad (0 \leq \bar{R} \leq 1) \quad (29)$$

where $\bar{I}_c$ is the value of I_c on the subloading surfaces and can be estimated from the postulation that the current stress point is always lying on the subloading surfaces.

A loading criterion is postulated using $\dot{\bar{R}}$ as follows.

$$\begin{aligned} \dot{\bar{R}} > 0 & \text{ Loading} \\ \dot{\bar{R}} < 0 & \text{ Unloading} \\ \dot{\bar{R}} = 0 & \text{ Neutral loading.} \end{aligned} \quad (30)$$

Here, $\dot{\bar{R}}$ is an increment of $\bar{R}$.

We specify conjugate point on the normal yield surfaces with the same outer normal direction as the current stress point σ'_{ij} on the subloading surfaces. Then, the conjugate effective stress tensor $\hat{\sigma}'_{ij}$ is

$$\hat{\sigma}'_{ij} = \frac{\sigma'_{ij}}{\bar{R}} \quad (31)$$

The hardening modulus H is postulated as

$$H = \hat{H} + (h - \hat{H})(1 - \bar{R}^*)^{\alpha_h} \quad (32)$$

$$\bar{R}^* = \frac{\bar{R} - \bar{R}_i}{1 - \bar{R}_i} \quad (33)$$

where $\hat{H}$ is the hardening modulus at the conjugate point, h an arbitrary constant which satisfies $h \gg H$, α_h a material parameter and $\bar{R}_i$ the initial value of $\bar{R}$ along a loading process.

Procedure for Estimation of Stress and Strain in Plastic Range

The stress and plastic strain increments are estimated on the basis of the generalized plasticity theory (Dafalias and Herrmann, 1982). The effective stress increment $\dot{\sigma}'_{ij}$ is given by

$$\dot{\sigma}'_{ij} = D_{ijkl}(\dot{\epsilon}_{kl} - \dot{\epsilon}^p_{kl}) \quad (34)$$

where D_{ijkl} is elastic modulus tensor, $\dot{\epsilon}_{ij}$ strain increment tensor and $\dot{\epsilon}^p_{ij}$ plastic strain increment tensor.

If plastic deformations are occurred, the plastic strain increment $\dot{\epsilon}^p_{ij}$ is assumed to be given as

$$\dot{\epsilon}^p_{ij} = \langle L \rangle \bar{n}_{ij} \quad (35)$$

where L is the loading function and it is determined by

$$L = \frac{n_{kl} D_{klmn} \dot{\epsilon}_{mn}}{H + D_{abcd} n_{ab} \bar{n}_{cd}} \quad (36)$$

Here, n_{ij} and $\bar{n}_{ij}$ are respectively unit vector of plastic loading and unit gradient vector of plastic potential at the conjugate point. They are defined as follows.

$$\bar{n}_{ij} = \frac{\frac{\partial \psi}{\partial \hat{\sigma}'_{ij}}}{\left[\left(\frac{\partial \psi}{\partial \hat{\sigma}'_{kl}} \right) \left(\frac{\partial \psi}{\partial \hat{\sigma}'_{kl}} \right) \right]^{1/2}} \quad (37)$$

$$n_{ij} = \frac{\frac{\partial f}{\partial \hat{\sigma}'_{ij}}}{\left[\left(\frac{\partial f}{\partial \hat{\sigma}'_{kl}} \right) \left(\frac{\partial f}{\partial \hat{\sigma}'_{kl}} \right) \right]^{1/2}} \quad (38)$$

Here, f and ψ denote respectively yield and plastic potential functions.

SCHEMATIC ESTIMATION OF MECHANICAL BEHAVIOR OF UNSATURATED SOILS

Here, the performance of the elastoplastic model described in previous section is schematically examined simulating the typical mechanical behavior of unsaturated soils. Collapse behavior and desaturated path (suction increase consolidation) are picked up as the typical ones. As these are in the triaxial stress condition in these examples, the yield surfaces can be plotted in the $p' \sim q$ space instead of $I_1 \sim \sqrt{J_2}$ space. Here, q is

$$q = \sigma_1 - \sigma_3. \quad (39)$$

Collapse Behavior

Figure 5 shows the scheme in which collapse due to wetting is explained using the model defined in the previous section. Let us now consider the isotropic compression test with constant suction and soaking. At first, the stress point lay on point A and then applying the isotropic stress the point moves to point B, where the stress point encounters the yield surface. With further increases in the isotropic stress the stress point moves to point C. As the stress path from point B to C is plastic, the yield surface expands with its loading. The soaking starts at point C. The stress path during soaking progresses on the line $\overline{CE}$, which

Fig. 5. The scheme of explanation for collapse due to wetting using the proposed elastoplastic model

lies on the state surface. According to the experimental result conducted by Jennings and Burland (1962), the soaking path without volume change is consistent with the line $\overline{CD}$. If it is not necessary to consider the stress path on the way of soaking but it is necessary to consider only the point after soaking like point E, we may estimate the state values after soaking by taking account of the equivalent path $\overline{CDE}$ instead of the path $\overline{CE}$ on the base of Jennings and Burland's experimental result. The yield surface on the path $\overline{CD}$ shrinks due to a decrease in suction accounting for Eq. (24), and after the soaking is completed the yield surface passing through point D is formed. The amount of stress after soaking is consistent with that at point E. Therefore, the yield surface must expand until the surface which passes point E. On the process of yield surface reexpansion, the plastic volumetric strain is induced and the volume of the soil decreases. In the case of considering the stress path on the way of soaking, the cycle of the above operations on the smaller suction increments makes the simulation of the stress path during soaking possible. The operation process is also illustrated in Fig. 5 where it is expressed as stepwise processes. Collapse behavior may thus be handled as the plastic deformation on the process of yield surface reexpansion.

Desaturated Path

Figure 6 shows the similar scheme on desaturated path (suction increase consolidation test) to Fig. 5. Let us now consider the suction increase consolidation test, where a soil is initially in a slurry condition. The preconsolidation pressure is almost equal to null, since the soil is initially in slurry. At suction smaller than the air entry suction s_e (the range from point A to point B in Fig. 6), the soil is in insular air saturation condition and an increase in suction may be estimated as an increase in effective stress as described before. The process from point A to point B can thus be regarded as the plastic one. Therefore the yield surface expands with increasing suction and the yield surface passing through point B is formed at air entry suction. With further increases in suction air enters

Fig. 6. The scheme of explanation for suction consolidation using the proposed elastoplastic model

into pores and an increase in suction not only increases effective stress but also induces an increase in both yield stress and the stiffness of soil skeleton in the plastic range. The increase rate of yield stress is greater than that of effective stress. The fact is guaranteed from state surface concept where desaturated path falls below the state surface. Therefore points C and D at which suctions are greater than air entry suction lie inside of yield surfaces. Points C and D are namely in the elastic range. Desaturated path in fuzzy saturation may thus be regarded as the elastic one.

In the subloading elastoplastic model, hardening modulus is assumed to be a function of distance between the current stress point and the conjugate stress point on normal yield surfaces. Hardening modulus becomes greater with an increase in the distance. Hardening modulus shows the resistance for plastic deformation. The greater hardening modulus, the greater resistance of plastic deformation. Comparing with point C and D, the distance between point C and its conjugate point is shorter than that between point D and its conjugate point, since the increase rate of yield stress is greater than that of effective stress. The hardening modulus at point D is namely greater than that at point C. In other words, the resistance for plastic deformation

at point D is greater than that at point C. This fact is coincident with the experimental result; the rate of volume change becomes smaller with an increase in suction.

EXAMPLES OF ANALYSES

Here, a consolidation (K_0 and isotropic) test series with and without soaking and a constant water content triaxial compression test series are simulated using the elastoplastic model described above. These experimental data were obtained by Maswoswe (1985) using Lower Cromer Till. Table 1 presents the material parameters. The water retention curve is shown in Fig. 7. Here, the bulk and shear

moduli are postulated to be functions of p' as follows.

$$K = \frac{-2.3(1+e_0)}{\kappa} p' + K_i \quad (40)$$

$$G = -\beta_G p' + G_i \quad (41)$$

where K and G are bulk and shear moduli, respectively. G_i , K_i and β_G are material parameters. These material parameters shown in Table 1 were obtained as follows. The air entry suction s_e was obtained from soil water retention curve shown in Fig. 7. The material parameters s_e and a_e concerned with effective stress can be decided as follows from the shear strength data at the wet side of critical state. From the shear strength data shown in Fig. 10(d), if the shear failure occurs on the failure line, we can get the u_{eq} values as the distances of horizontal axis between the failure points on $((\sigma_1 - \sigma_3)/2) \sim ((\sigma_1 + \sigma_3)/2) - u_a$ stress paths and the points with the same shear strength $((\sigma_1 - \sigma_3)_f/2)$ on the failure line. Once we can get u_{eq} values, s_e and a_e can be estimated from the linear relationship between s^* and s^*/f , where $f = u_{eq} - u_a + s_e$, using the least square method. Next, the material parameters λ , Γ , e_0^0 , a_s and n_s concerned with state surface can be obtained as follows. The parameters λ and Γ can be estimated from the relationship between e and $\log p'$ at saturation condition. Referring to the $e \sim \log p'$ relationships as shown in Fig. 1(b), e_0^0 and λ^* values at each s^* values can be obtained. As the values of y can be estimated from Eq. (7) and the relationship between $\log y$ and $\log s^*$ from Eq. (9) is linear, we can get a_s and n_s using the least square method. Other parameters concerned with the elastic and plastic properties could be estimated from experimental data obtained by triaxial shear and consolidation tests except R , h and α_h . The value of parameter h concerned with subloading surface was employed the value usually used. The parameters R and α_h were estimated from the try and error numerical simulations.

The results of these simulations are shown in Figs. 8 ~ 10. Figure 8 represents the K_0 consolidation simulation results. Figures 9 and 10 represent respectively the isotropic consolidation and triaxial compression simulation

Table 1. Material parameters used in the analysis

Elasticity	$\kappa=0.035$, $G_i=K_i=0.0$ kN/m ² , $\beta_G=118.6$
Subloading	$h=2 \times 10^5$ kN/m ² , $\alpha_h=4.8$
Plasticity	$\phi'=28^\circ$, $\phi'_{cs}=30^\circ$, $R=0.42(1.0)$, $I_c=-1050(-930)$ kN/m ² $p'_0=-280(-240)$ kN/m ²
Effective stress	$s_e=a_e=162.5$ kN/m ² $s_e=0.0$ kN/m ²
State surface	$\lambda=0.166$, $\Gamma=0.840(0.819)$, $e_0^0=1.087(1.024)$, $a_s=360.5$ kN/m ² , $n_s=1.528$
Initial conditions	$s_0=240$ kN/m ² , $e_0=0.661(0.655)$, $S_{r,0}=0.45$

The values in () denote those used in the isotropic consolidation and triaxial test simulations.

Fig. 7. Soil water retention curve of Lower Cromer Till

Fig. 8. K_0 consolidation simulation results on Lower Cromer Till. (a) Relationship between void ratio and vertical stress (b) Stress paths with soaking (SK2) (c) Stress paths with soaking (SK3) (d) Stress paths without soaking (SK5)

results. Here, the effective stresses are the stresses estimated using Eqs. (3) ~ (5).

Figure 8(a) shows the $e \sim \log \sigma_v$ relationships for the K_0 consolidation test simulations. The results in the K_0 consolidation process become asymptotic to the saturated line as σ_v increases. This is ascribed to the fact that the compression of the soil skeleton with an increase of σ_v makes the degree of saturation higher. In the soaking processes, a great volume decrease (the collapse phenomenon) is induced and the points after soaking are almost on the saturated line. Simulation results are consistent with the identical test results.

Figures 8(b) ~ (d) show the associated stress paths. Figures 8(b) and (c) represent the K_0 stress paths with soaking and Figure 8(d) represents those without soaking. During soaking, the values of deviator stress decrease. The stress points after soaking are almost on the saturated K_0 line. It can be seen from these figures that the effective stress paths trace on the saturated K_0 lines.

Figure 9 shows the $e \sim \log p'$ and $e \sim \log p$ relationships for the isotropic consolidation with soaking test simulations. The point after soaking is almost on the saturated line and this is the same as the one in the K_0 test simulations. The values of p' during soaking decrease as expected. The $e \sim \log p'$ line after soaking is consistent with the saturated line. Simulation results also agree with the experimental ones.

Figure 10 represents the triaxial compression simulation results. Simulation were per-

Fig. 9. Isotropic consolidation simulation results on Lower Cromer Till

Fig. 10. Triaxial compression test simulation results on Lower Cromer Till. (a) Relationship between deviator stress and axial strain (b) Relationship between volumetric strain and axial strain (c) Relationship between void ratio and vertical stress (d) Stress paths (e) Relationship between void ratio and mean effective stress at failure

formed under the following conditions. The soil samples were sheared at a constant strain rate after isotropic consolidation with constant water content, while in the experiments (test No. T3 and T4), suction was increased until the prescribed values ($s=255$ kN/m² on No. T3 and $s=240$ kN/m² on No. T4) after isotropic consolidation with constant water content and then the samples were sheared at a constant strain rate. These suction values applied after consolidation, however, did not satisfy with the soil-water retention curve shown in Fig. 7 and did not keep their

balance. They were therefore temporary.

Figure 10(a) shows the relationship between deviator stress and axial strain. Predicted values do not agree well with the experimental ones except No. T5 at low strain range. This seems to arise from the temporary applying suction after consolidation. However, overall predicted values are consistent with those obtained by the experiments.

Figure 10(b) shows the relationship between volumetric strain and axial strain during shear. There is no marked difference in volume change between the tests in spite of the

different confining pressures (The initial values of p are 150, 300 and 450 kN/m² in T5, T3 and T4, respectively.). Predicted values are consistent with experimental ones.

Figure 10(c) represents the relationship between void ratio and vertical stress during shear. Predicted values agree closely with the experimental ones.

Figure 10(d) shows stress paths during shear. The figure indicates that the predicted shear strength is less than the experimental one. The maximum difference is about 13%. This arises from a pore water pressure behavior difference between analyses and experiments. The pore water pressure behavior during shear in the experiments did not satisfy the soil water retention curve shown in Fig. 7. Therefore, we must reexamine the soil water retention characteristics to obtain better simulation results.

Figure 10(e) shows the relationship between the void ratio and mean effective stress at failure. The relationship in this case can almost be represented as a line in $e \sim \log p'$ space and simulated results are also on this line. The slope of the line is almost the same as that for the unsaturated normally consolidated line at the same p' stress range. Such a property is also seen in saturated soils. However, the $e \sim \log p'$ relationship described above may be acceptable at a limited stress range and must be nonlinear at a wider stress range.

The elastoplastic model was useful thus to express the typical mechanical behavior of unsaturated soils.

CONCLUSIONS

We presented an elastoplastic constitutive model for unsaturated soils. The model contains two suction effects which refer to increases in effective stresses, yield stress and stiffness of the soil skeleton against plastic deformation due to an increase in suction. The first suction effect; an increase in suction induces an increase in effective stress was taken into account formulating the model based on the newly defined effective stress. The second suction effect; an increase in suction increases both yield stress and the stiffness of soil

skeleton against plastic deformations was taken into account using state surface concept as shown in Eqs. (24) ~ (26). The smooth transition from elasticity to plasticity was presented introducing the subloading surface model. Such a treatment is useful to analyze many practical problems. The performance of the model was conceptually and analytically examined simulating the typical mechanical behavior of unsaturated soil. The model could well express them. We will examine the application of the model to the boundary value problems.

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