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An elasto-viscoplastic numerical analysis of the swelling process of unsaturated bentonite

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A numerical analysis of the swelling behavior of bentonite is presented using an elasto-viscoplastic theory. It is an extension of an elasto-viscoplastic model for unsaturated soil, which can describe the behavior of macrostructures, such as change of suction, pore pressure and degree of saturation. The volume increase of montmorillonite minerals due to water absorption into the interlayers, is assumed to be a part of viscoplastic volumetric strain. An internal variable H which controls an increase in water absorption into clay interlayers is adopted to describe the swelling behavior of microstructure. In addition, the internal compaction effect caused by swelling of clay unit is described by the expansion of the overconsolidation boundary surface and the static yield surface. Based on the proposed model, a fully coupled soil-water-air finite element analysis is conducted to study the development of swelling pressure. Comparing the experimental results and the simulated results, it is found that the proposed model can reproduce the effects of dry density and the initial water content on the swelling behavior.

Key Words: *bentonite, elasto-viscoplastic model, swelling pressure, unsaturation*

1. Introduction

Highly expansive soil, such as bentonite, is currently considered to be suitable barrier material for the isolation of waste, e.g., nuclear, industrial, and mining wastes, from the surrounding environment because of its low permeability. Due to the swelling property, cracks that may exist in the surrounding soil and rock can be filled with bentonite. According to conventional design practices, however, it is acceptable to assume that eventually groundwater will saturate these bentonite barriers. Therefore, it is important to evaluate the swelling pressure that bentonite imposes on containers and surrounding soil and rock due to groundwater seepage, as well as the long-term stability of the barrier structure itself. In fact, determining the swelling pressure is an important aspect of all high-level radioactive waste disposal projects (e.g., Tripathy, Sridharan and Schanz 2004).

Many attempts have been made in the past to understand the swelling mechanism of expansive soils. The Gouy-Chapman diffuse double layer theory (Gouy 1910) has been the most widely used approach to relate clay compressibility to basic particle-water-cation interaction (Marcail et al. 2002). During the last few years, a number of experimental and theoretical research works have been carried out on bentonite and bentonite-soil mixtures. Swelling pressure tests on compacted bentonite have been conducted by some researchers (Push 1982; Kanno and Wakamatsu 1992). The relationship between swelling deformation and the distance between two montmorillonite layers was proposed (Komine and Ogata 1996). Sridharan and Choudhury (2002) proposed a swelling pressure equation for Na-montmorillonite while analyzing the compression data of slurred samples of montmorillonite reported by Bolt (1956). However, some researchers (Mitchell 1993; Tripathy, Sridharan and Schanz

2004) have shown that very little information is available on the use of the diffuse double layer theory for the determination of the swelling pressure of compacted bentonite.

Numerical models have also been proposed to simulate expansive soil (Alonso et al. 1991 and 1999; Gens and Aloso 1992) based on the elastoplastic theory. According to their theory, two levels are distinguished for the structure, (1) a microstructural level that corresponds to the active clay minerals and their vicinity and (2) a macrostructural level that accounts for the larger structural soil arrangements. The microstructure, namely, the swelling domain that expands when hydrated, is thought to be water-saturated even at high levels of suction. In contrast, the macrostructure is assumed to be unsaturated when subjected to suction, and its behavior may be described by a conventional frameworks for unsaturated soils. In addition, a theoretical model has been proposed by Shuai and Fredlund (1998) to describe the volume changes during various oedometer swelling tests.

In the present paper, an elasto-viscoplastic swelling model for unsaturated bentonite is developed based on the elasto-viscoplastic model for unsaturated soil. An internal variable *H*, which controls the growth of the absorption of water into the clay interlayer, is introduced to describe the large volumetric expansive behavior of the microstructure. This model includes the effects of suction and the swelling effect into the hardening parameter. Using the proposed model, the swelling behavior of bentonite is simulated by the multi-phase finite element method.

2. Elasto-viscoplastic constitutive model for unsaturated swelling soil

Adopting skeleton stress and suction as the stress variables, an elasto-viscoplastic model for unsaturated soil has been proposed (Oka et al. 2006; Feng et al. 2006; Oka et al. 2008). Based on this model, three-dimensional multiphase numerical analysis has been carried out. The simulation results show that the behavior of unsaturated soil, such as the changes in pore air pressure, pore water pressure, and volumetric strain, can be simulated well with this model (Oka et al. 2008). This model also can describe the viscoplastic volumetric collapse phenomenon due to decrease of suction. To reproduce the swelling phenomenon caused by the clay particles, such as montmorillonite particles, the elasto-viscoplastic constitutive model for unsaturated soil is extended to be able to reproduce the volumetric swelling. In the present model, a swelling equation is proposed to describe the viscoplastic volumetric swelling.

2.1 Skeleton stress

The material to be modeled is composed of three phases, namely, solid (S), water (W), and air (G), which are continuously distributed throughout space. Volume fraction n^α is defined as the local ratio of the volume element with respect to the total volume, namely,

$$n^\alpha = \frac{V^\alpha}{V} \quad (\alpha = S,W,G) \tag{1}$$

The volume fraction of the void, n, is written as

$$n = \sum_{\alpha'} n^{\alpha'} = 1 - n^s \quad (\alpha' = W,G) \tag{2}$$

Finally, the material density ρ_α for α phase is given by

$$\rho_\alpha = \frac{M^\alpha}{V^\alpha} \quad (\alpha = S,W,G) \tag{3}$$

where M^α is the mass of α phase.

The effectives stress has been defined by Terzaghi for saturated soil; however, the effective stress needs to be reconsidered for unsaturated soil where the fluids are compressible. In present study, the skeleton stress tensor and suction are adopted as the stress variables. Skeleton stress is tha same as the "average skeleton stress" by Jommi (2000). Total stress tensor σ_{ij} is obtained from the sum of the partial stress values, σ_{ij}^α, namely,

$$\sigma_{ij} = \sum_\alpha \sigma_{ij}^\alpha \quad (\alpha = \mathrm{S, W, G}) \tag{4}$$

$$\sigma_{ij}^{\mathrm{S}} = \sigma_{ij}^{'} + n^{\mathrm{S}} P^F \delta_{ij}, \quad \sigma_{ij}^{\mathrm{W}} = n^{\mathrm{W}} P^{\mathrm{W}} \delta_{ij} \tag{5}$$

$$\sigma_{ij}^{\mathrm{G}} = n^{\mathrm{G}} P^{\mathrm{G}} \delta_{ij}, \quad \sigma_{ij}^{'} = \sigma_{ij} - P^F \delta_{ij} \tag{6}$$

where n^W and n^G are the volume fraction of liquid phase and gas phase, $\sigma_{ij}^{'}$ is the skeleton stress in the present study, the average pore pressure, P^F is defined as,

$$P^F = S_r P^{\mathrm{W}} + (1 - S_r) P^{\mathrm{G}} \tag{7}$$

where S_r is the degree of saturation.

2.2 Stretching

It is assumed that the strain rate tensor consists of elastic stretching tensor D_{ij}^e, the viscoplastic stretching tensor D_{ij}^{vp}, and an additional viscoplastic stretching tensor $D_{ij}^{vp(s)}$ due to the microstructural swelling. Total stretching tensor D_{ij} is defined in the following equation:

$$D_{ij} = D_{ij}^e + D_{ij}^{vp} + \frac{1}{3}\delta_{ij} D_{kk}^{vp(s)} \tag{8}$$

The elastic stretching tensor D_{ij}^e is given by a generalized Hooke type of law, namely,

$$D_{ij}^e = \frac{1}{2G}\dot{S}_{ij} + \frac{\kappa}{3(1+e_0)}\frac{\dot{\sigma}_m^{'}}{\sigma_m^{'}}\delta_{ij} \tag{9}$$

where S_{ij} is the deviatoric stress tensor, $\sigma_m^{'}$ is the mean skeleton stress, G is the elastic shear modulus, e_0 is the initial void ratio, κ is the swelling index, and the superimposed dot denotes the time differentiation.

2.3 Swelling due to absorption of water into the interlayers of clay particles

From the experimental results on bentonite (Komine and Ogata 1996; Push 1982), it has been shown that the swelling phase is followed by an asymptotic tendency towards a constant final value. In the model, an evolutional equation is used to describe the viscoplastic volumetric swelling of the particles as

$$D^{vp(s)}_{kk} = -\dot{H} \tag{10}$$

$$\dot{H} = B(A - H) \tag{11}$$

where H is an internal variable that describes the growth in the absorption of water into the clay particles, and A and B are material parameters. Figure 1 shows the swelling equation curves at various values for parameters A and B. It can be seen that A is a parameter for the potential of the absorption of water, and B is a parameter which controls the evolution rate of H.

2.4 Internal compaction phenomenon

With the absorption of water into the interlayers, the distance between the two platelets increases from 15 Å to 20 Å. Figure 2 illustrates the swelling process in the case where the swelling deformation is restricted. With wetting, the macro voids gradually becomes packed with swollen montmorillonite particles. As shown in Figure 3, the SEM image of the wetting process of the bentonite has been reported by Komine and Ogata (2004), in which the bentonite content is 100%. It can be seen that the macro voids are finally filled up by the volume increase in bentonite.

As a result, the sample becomes stiffer and has a higher strength, which is similar to the soil being highly compacted. Hereafter, the phenomenon is called the "internal compaction effect"; it is different from traditional compaction in the changes in water content.

2.5 Overconsolidation Boundary Surface

In the model, the overconsolidation boundary surface is defined to delineate the normal consolidation (NC) region, $f_b \geq 0$, and the overconsolidation (OC) region, $f_b < 0$, as follows:

$$f_b = \bar{\eta}^*_{(0)} + M^*_m \ln \frac{\sigma'_m}{\sigma'_{mb}} = 0 \tag{12}$$

$$\bar{\eta}^*_{(0)} = \left\{\left(\eta^*_{ij} - \eta^*_{ij(0)}\right)\left(\eta^*_{ij} - \eta^*_{ij(0)}\right)\right\}^{\frac{1}{2}} \tag{13}$$

where η^*_{ij} is the stress ratio tensor ($\eta^*_{ij} = S_{ij} / \sigma'_m$), and (0) denotes the state at the end of the consolidation, in other words, the initial state before deformation occurs. M^*_m is the value of $\eta^* = \sqrt{\eta^*_{ij}\eta^*_{ij}}$ at the critical state M^*_f . σ'_{mb} is the strain-hardening and softening parameter, which controls the size of the boundary surface.

2.6 Static yield function

To describe the mechanical behavior of clay at its static equilibrium state, a Cam-clay type of static yield function is assumed to be

$$f_y = \bar{\eta}^*_{(0)} + \tilde{M}^* \ln \frac{\sigma'_m}{\sigma'^{(s)}_{my}} = 0 \tag{14}$$

where $\sigma'^{(s)}_{my}$ is the strain-hardening and softening parameter which controls the size of the static yield function.

2.7 Hardening rule considering internal compaction effect

According to the elasto-viscoplastic model for unsaturated soil (Oka et al. 2006; Feng et al. 2006), hardening parameters: σ'_{mb} and $\sigma'^{(s)}_{my}$ in Eqs. (11) and (13) are assumed to be function of viscoplastic strain ε^{vp}_{ij} and suction P^C as follows:

$$\sigma'_{mb} = \sigma'_{ma} \exp\left(\frac{1+e}{\lambda-\kappa}\varepsilon^{vp}_{kk}\right)\left[1+S_I \exp\left\{-S_d\left(\frac{P^c_i}{P^c}-1\right)\right\}\right] \tag{15}$$

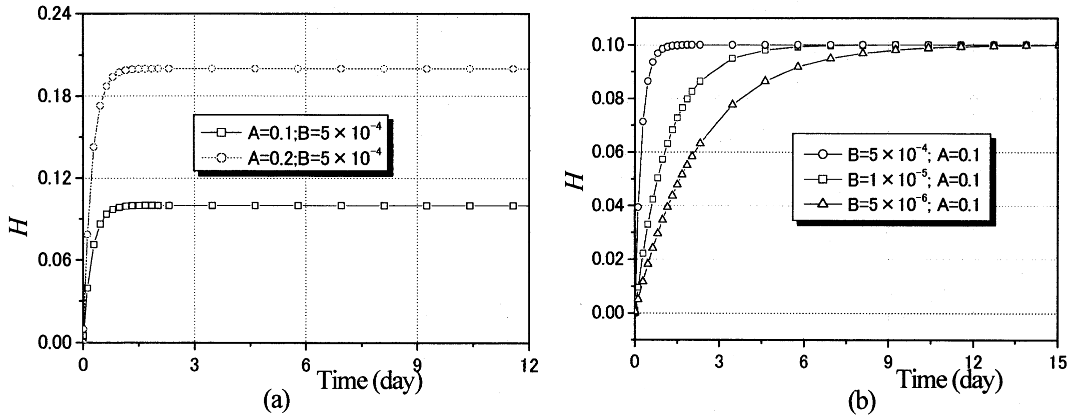

Fig. 1 Swelling equations with different parameters for A and B

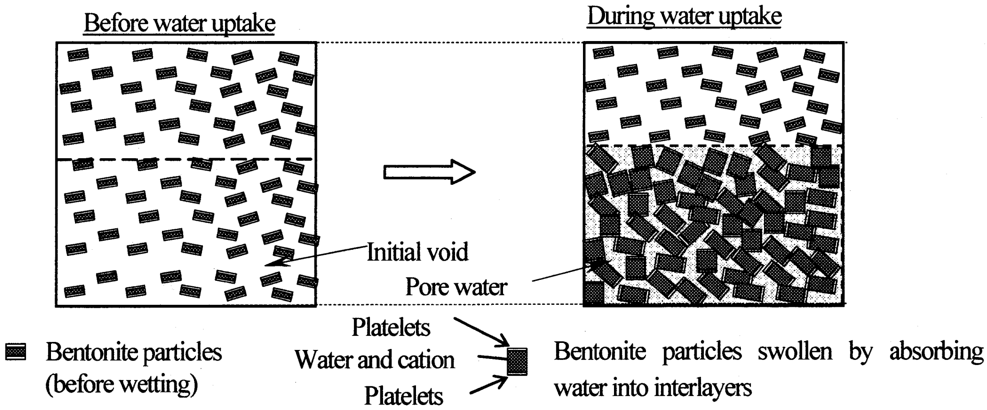

Fig. 2 Process of swelling with restricted deformation

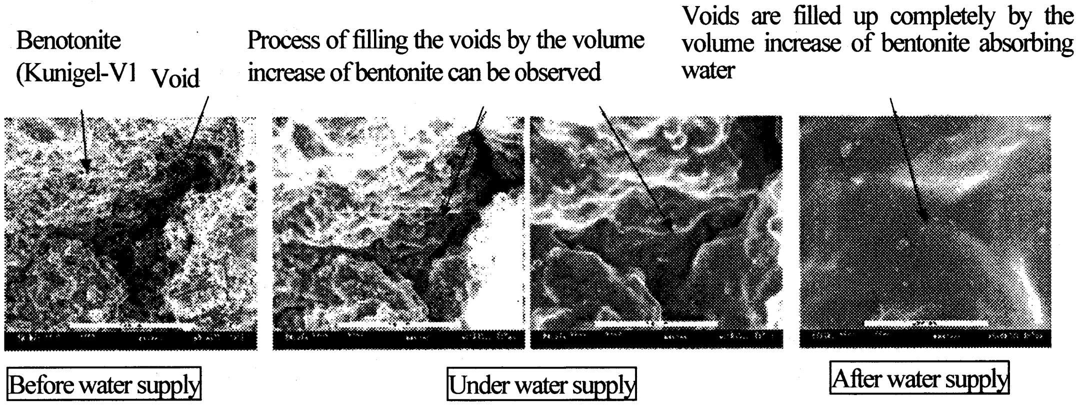

Fig. 3 Swelling behavior of bentonite (bentonite A, Kunigel-V1)(Komine and Ogata 2004)

$$\sigma'^{(s)}_{my} = \frac{\sigma'^{(s)}_{myi}}{\sigma'_{mai}} \sigma'_{ma} \exp(\frac{1+e}{\lambda-\kappa}\varepsilon^{vp}_{kk})\left[1+S_I \exp\left\{-S_d\left(\frac{P^c_i}{P^c}-1\right)\right\}\right] \quad (16)$$

where ε^{vp}_{kk} is the viscoplastic volumetric strain, λ and κ are the compression and the swelling index, respectively, and e_0 is the initial void ratio. σ'_{ma} is a strain-softening parameter used to describe the effect of structural degradation, which is assumed to decrease with an increase in viscoplastic strain, namely,

$$\sigma'_{ma} = \sigma'_{maf} + \left(\sigma'_{mai} - \sigma'_{maf}\right)\exp(-\beta z) \quad (17)$$

$$z = \int_0^t \dot{z}dt\ ; \quad \dot{z} = \sqrt{\dot{\varepsilon}^{vp}_{ij}\dot{\varepsilon}^{vp}_{ij}} \quad (18)$$

in which, σ'_{mai} and σ'_{maf} are the initial and the final values for σ'_{ma}, respectively, and β is a material parameter that controls the rate of structural changes. Since the viscoplastic strain is equal to zero at the initial state, we obtain the consolidation yield stress σ'_{mbi} is equal to σ'_{mai}.

The last terms in Eqs. (15) and (16) are adopted to reflect the suction effect on unsaturated soil, in which P^c is the present suction values, P^c_i is the reference suction, and S_I is a material parameter denoting the strength ratio to the saturated soil when the suction equals to reference suction P^c_i. S_d is a material parameter which controls the rate of increase or decrease in σ'_{mb} with changes in suction.

In the present study, the internal compaction effect is expressed as the expansion of the overconsolidation boundary surface, and the static yield surface as follows:

$$\sigma'_{mb} = \sigma'_{ma} \exp\left(\frac{1+e}{\lambda-\kappa}\varepsilon^{vp*}_{kk}\right)\left[1+S_I \exp\left\{-S_d\left(\frac{P^c_i}{P^c}-1\right)\right\}\right] \quad (19)$$

$$\sigma'^{(s)}_{my} = \frac{\sigma'^{(s)}_{myi}}{\sigma'_{mai}} \sigma'_{ma} \exp(\frac{1+e}{\lambda-\kappa}\varepsilon^{vp*}_{kk})\left[1+S_I \exp\left\{-S_d\left(\frac{P^c_i}{P^c}-1\right)\right\}\right] \quad (20)$$

where ε^{vp*}_{kk} is the viscoplastic volumetric strain including the special swelling effect taken into account which is defined as

$$\varepsilon^{vp*}_{kk} = \varepsilon^{vp}_{kk} + \gamma|H| \quad (21)$$

where, γ is adopted to reflect the percentage of the swelling viscoplastic strain considered, which varies from 0 to 1. γ being equal to 0 means that the swelling viscoplastic strain does not affect the expansion or shrinkage of the overconsolidation boundary surface and the static yield boundary surface, while γ being equal to 1 means that the viscoplastic swelling volumetric strain has an effect on expansion of the boundary surface.

2.8 Viscoplastic potential function

The viscoplastic potential function is given by

$$f_p = \overline{\eta}^*_{(0)} + \widetilde{M}^* \ln \frac{\sigma'_m}{\sigma'_{mp}} = 0 \tag{22}$$

where $\widetilde{M}^*$ is assumed to be constant in the NC region and varies with the current stress in the OC region as

$$\widetilde{M}^* = \begin{cases} M^*_m & : NC \quad region \\ -\dfrac{\sqrt{\eta^*_{ij}\eta^*_{ij}}}{\ln(\sigma'_m/\sigma'_{mc})} & : OC \quad region \end{cases} \tag{23}$$

where σ'_{mc} denotes the mean skeleton stress at the intersection of the overconsolidation boundary surface and the σ'_m axis.

2.9 Viscoplastic flow rule

The viscoplastic stretching tensor is expressed by the following equation which is based on Perzyna's type of viscoplastic theory (Perzyna, 1963; Kimoto & Oka 2005) as

$$D^{vp}_{ij} = \gamma\langle\Phi_1(f_y)\rangle\frac{\partial f_p}{\partial\sigma'_{ij}} \tag{24}$$

in which $\langle\ \rangle$ are Macaulay's brackets; $\langle\Phi_1(f_y)\rangle = \Phi_1(f_y)$, if $f_y > 0$ and $\langle\Phi_1(f_y)\rangle = 0$, if $f_y \le 0$. Φ_1 indicates strain rate sensitivity. Based on the experimental data from the strain rate constant triaxial tests, the material function is given as

$$\gamma\Phi_1(f_y) = C\sigma'_m \exp\left\{m'\left(\overline{\eta}^*_{(0)} + \widetilde{M}^* \ln\frac{\sigma'_m}{\sigma'_{mb}}\right)\right\} \tag{25}$$

$$C = C' \exp\left(m'\widetilde{M}^* \ln\frac{\sigma'_{mai}}{\sigma'^{(s)}_{myi}}\right) \tag{26}$$

3. Multiphase finite element formulation for the analysis of unsaturated bentonite

It is found that the water absorbed into these aggregates does not flow, which can be considered as a part of solid phase. In contrast, the micro pores between the clay aggregates are assumed to be occupied by free water and air, which can be described within the framework of a macroscopic continuum mechanical approach through the use of the theory of porous media.

Proceeding from the general geometrically non-linear formulation, the governing balance relations for multiphase material can be obtained (e.g., Ehlers 2004; Loret and Khalili 2000). Using the skeleton stress and suction as stress variables, Oka et al. (2006) and Kimoto et al. (2007) proposed an air-water-soil coupled finite element model. And a three-dimensional numerical analysis has been carried out to investigate the triaxial behavior of unsaturated silt (Feng et al, 2006).

3.1 Conservation of momentum

The momentum balance for each phase is obtained with the following equations when the acceleration is disregarded,

$$\begin{aligned}\sigma'_{ji,j} + \left(n^S P^F \delta_{ji}\right)_{,j} + \rho_S n^S \overline{F}_i - D^{SW}\left(v_i^S - v_i^W\right) \\ - D^{SG}\left(v_i^S - v_i^G\right) = 0\end{aligned} \tag{27}$$

$$\begin{aligned}\left(n^W P^W\right)_{i,j} + \rho_W n^W \overline{F}_i - D^{WS}\left(v_i^W - v_i^S\right) \\ - D^{WG}\left(v_i^W - v_i^G\right) = 0\end{aligned} \tag{28}$$

$$\begin{aligned}\left(n^G P^G\right)_{i,j} + \rho_G n^G \overline{F}_i - D^{GS}\left(v_i^G - v_i^S\right) \\ - D^{GW}\left(v_i^G - v_i^W\right) = 0\end{aligned} \tag{29}$$

in which $\overline{F}_i$ is the body force and the interaction between water and gas phases D^{WG} and D^{GW} is assumed to be zero. the interaction between solid phase and other two phases are given as

$$D^{WS} = D^{SW} = \frac{\left(n^W\right)^2 \rho_W g}{k^W},\quad D^{GS} = D^{SG} = \frac{\left(n^G\right)^2 \rho_G g}{k^G} \tag{30}$$

in which, g is the acceleration of gravity, k^W and k^G are the permeability coefficients for water and air phase, respectively. The sum of Eqs. (27)-(29) leads to

$$\sigma_{ji,j} + \rho^E \overline{F}_i = 0,\quad \rho^E = \sum_\alpha n^\alpha \rho_\alpha \quad (\alpha = S, W, G) \tag{31}$$

Finally, we used the rate type of conservation of momentum neglecting body force for updated Lagrangian scheme, which is given by

$$\dot{S}_{ji,j} = 0 \tag{32}$$

where $\dot{S}_{ij}$ is the total nominal stress rate tensor.

3.2 Conservation of mass

The conservation laws of mass for liquid and gas phase are given in the following equations:

$$S_r D_{ii} + \dot{S}_r n = -V^W_{i,i} \tag{33}$$

$$(1 - S_r)D_{ii} - \dot{S}_r n + (1 - S_r)n\frac{\dot{\rho}_G}{\rho_G} = -V^G_{i,i} \tag{34}$$

where V^W_i and V^G_i are the apparent velocity for water and air, n is the porosity, D_{ij} is the stretching tensor.

3.3 Soil water characteristic curve

The relation between suction and saturation is given in the following equation proposed by van Genuchten (1980)

$$S_{re} = \left\{1 + (\alpha P^C)^{n'}\right\}^{-m} \tag{35}$$

where α, m, and n are material parameters and the relation $m = 1 - 1/n'$ is assumed. S_{re} is an effective degree of saturation.

4. Simulation results

In the finite element analysis, an eight-node quadrilateral element is used for the displacement and the four nodded element is used for the pore pressure. The finite element mesh and boundary conditions for the simulation are shown in Figure 4. The bottom boundary is assumed to be the wetting boundary with water pressure of 10 kPa. The other boundaries are assumed to be impermeable for water and air. The main parameters and initial conditions are listed in Table 1. Elastic shear modulus, viscoplastic parameter C, S_I , S_d , air permeability, and shape factor a and b are assumed and the other parameters are determined by the existing data (Horikoshi et al. 2007). For the initial pore water pressure, i.e., initial suction is assumed to be 100kpa which is smaller than the expected one. The reason is that the initial suction is only for the initial matrix suction due to the meniscus between particles.

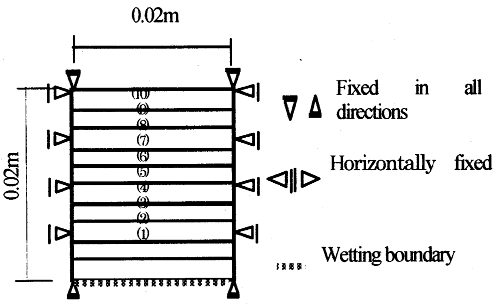

Fig. 4 Finite element mesh and boundary conditions

Figure 5 shows the changes in the degree of saturation for each element with wetting. In this analysis, it is assumed that the element starts swelling when the saturation reaches a given value. From Figure 5, it can be seen that swelling starts from bottom element and moves upwards, element by element.

To investigate the swelling behavior of bentonite, as shown in Table 2, swelling pressure tests of bentonite (Kunigeru GX) have been carried out by Ono et al.(2006). The experimental results are shown in Figure 6(a). It is confirmed that dry density controls the finial swelling pressure. Meanwhile, initial water content affects the type of swelling curve.

In the model, parameter A is adopted to describe the swelling potential, namely, dry density effect. The parameter γ in Eq.(21) is a parameter used to describe the "internal compaction effect". Considering that the initial water content also can lead to some degree of swelling. It is reasonable to investigate the swelling behavior with different dry density and initial water content by adopting proper value of A and γ. Table 3 lists the value of A and γ used in this simulation. For cases SW3 and SW6, with higher dry densities, higher A values (0.18, and 0.16) are adopted. For cases with higher initial water content values (SW5, SW6), a larger γ value (0.3) is used to represent the initial hardening effect.

Table 1 Material parameters

Elastic shear modulus(kPa)	G_0	3.74×10^4
Initial yield stress (kPa)	σ'_{mbi}	100
Initial pore water pressure (kPa)	P^W	-100
Swelling index	κ	0.078
Compression index	λ	0.117
Viscoplastic parameter	m'	95.4
Viscoplastic parameter(1/s)	C	9.47×10^{-18}
Stress ratio at critical state	M^*_m	0.4736
Suction parameter	S_I	0.5
Suction parameter	S_d	0.25
Structural parameter(kPa)	σ'_{maf}	83
Structural parameter	β	5.0
van Genuchten parameter	α	0.015
van Genuchten parameter	n	1.517
Permeability of water (m/s)	k^W	2.0×10^{-13}
Permeability of air(m/s)	k^G	1.3×10^{-11}
Shape parameter	a	3.0
Shape parameter	b	2.3
Expansive parameter	B	0.00001
Expansive parameter	A	In Table 3
Expansive parameter	γ	In Table 3

Table 2 Experimental cases (Ono, 2006)

Case	Dry density(Mg/m^3)	Water content (%)	Degree of saturation (%)
SW2	1.6	6.5	27
SW3	1.8	6.5	39
SW5	1.6	21.6	91
SW6	1.8	15.3	92

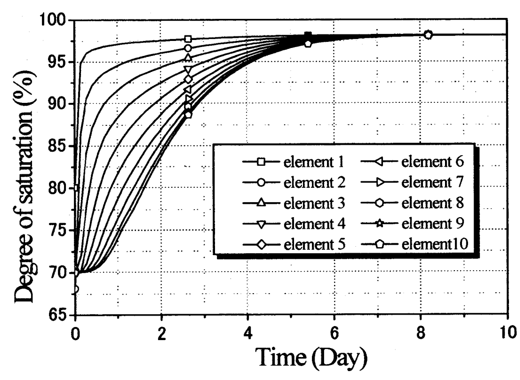

Fig. 5 Simulation results of degree of saturation-time relation with wetting

The predicted swelling pressures during wetting process are shown in Figure 6(b). Simulation result shows that large parameter A gives the large magnitude of swelling pressure. It is seen that the effect of dry density can be described by parameter A. According to experimental results, for samples with high initial water content, such as SW5 and SW6, swelling pressure increases monotonically, while a time-softening behaviour can be observed in the samples with low initial water content, such as SW2, SW3. As mentioned before, parameter γ is adopted to describe the "internal compaction effect" in this model. The softening comes from the structural degradation effect by the parameter β (Kimoto and Oka, 2005). When larger value of γ is adopted the hardenging behavior is significant(SW5,SW6). On the other hand, in the case of smaller value of γ, the softening effect is significant due to degradation of materialsis. These cases correspond to the lower initial water content cases (SW2,SW3). Simulational results show that by adjusting the value of parameter γ we can reproduce the hardeing effect associated with initial water contents. For the maximum value of the swelling pressure, it is seen that the higher magnitude of the swelling pressure is obtained by the large value of A.

Table 3 Parameter for simulation

Case	A	B	γ	No. of Exp.
S2	0.12	0.00001	0.1	SW2
S3	0.18	0.00001	0.1	SW3
S5	0.10	0.00001	0.3	SW5
S6	0.16	0.00001	0.3	SW6

5. Conclusion

In this paper, an elasto-viscoplastic model for unsaturated expansive soil is proposed. An internal variable H that reflects the growth of the absorption of water into the interclay is adopted to describe the expansive behavior of microstructures. An internal compaction effect affects the expansion of the overconsolidation boundary surface and the static yield surface. Using the proposed model, FEM analyses were conducted to simulate the swelling pressure. Parameter A and γ are adopted to describe the swelling potential and the internal compaction effect of bentonite (Kunigel GX). Compared with the experimental results, it has been found that the proposed

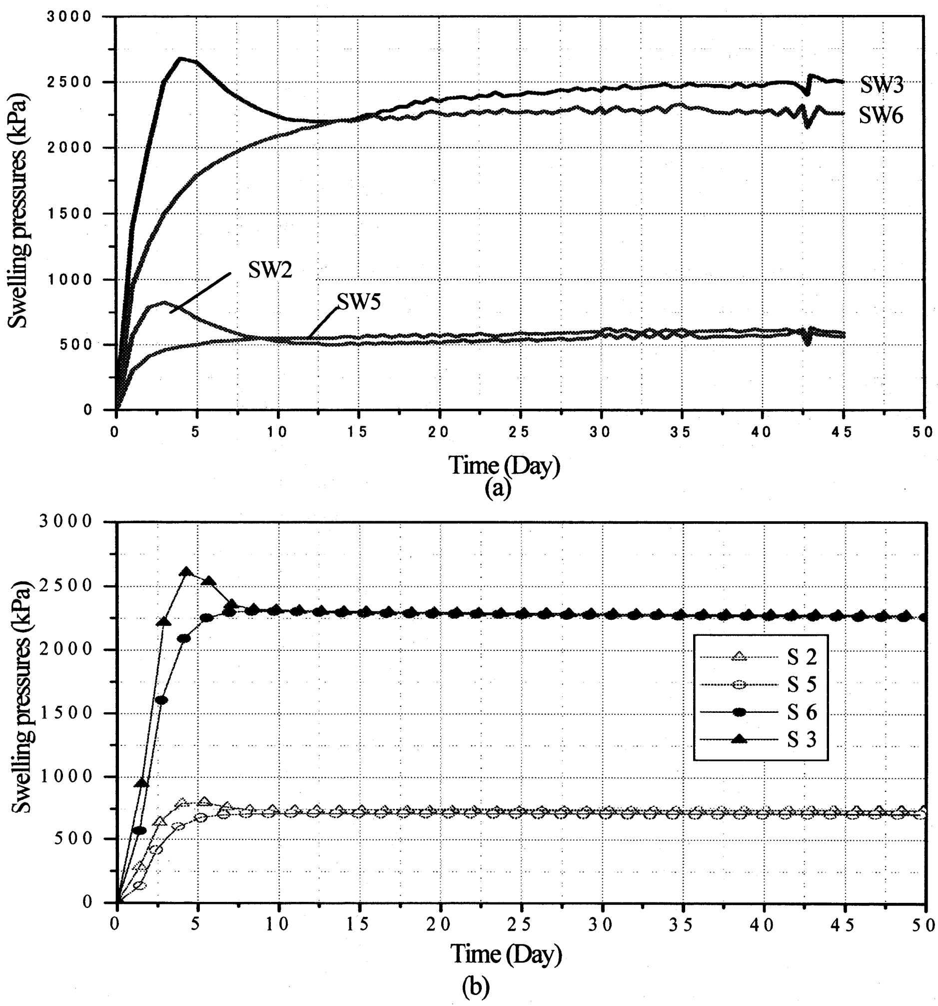

Fig. 6 Swelling pressure-time profile (a) experimental results (after Ono et al., 2006); (b) simulation results

model can well reproduce the effect of dry density and initial water content on the swelling pressure.

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