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# UNIAXIAL COMPRESSION TEST OF UNSATURATED MASADO UNDER CONSTANT DEGREE OF SATURATION CONDITION AND ITS MODELING

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**ABSTRACT:** Soils in surface ground are usually unsaturated. Proper modeling of the characteristics of unsaturated soil is therefore absolutely needed. However, due to its complicated mechanical behavior, establishing a constitutive model for unsaturated soil is far more difficult than that for saturated soil. In this research, uniaxial compression tests were firstly conducted on unsaturated Masado (Decomposed granite) under constant degree of saturation condition to investigate the mechanical behavior of the soil at different degree of saturation. Then, based on the test data, the performances of previously proposed elastoplastic constitutive model for unsaturated soil were checked carefully, taking the Bishop-type skeleton stress and the degree of saturation as the state variables. It is found that the mechanical behavior of the unsaturated Masado can be properly described by the proposed elastoplastic constitutive model that takes the skeleton stress and the degree of saturation as the state variables.

*Keywords: Unsaturated Soil, Degree of Saturation, Constitutive Model, Uniaxial Compression Test*

## 1. INTRODUCTION

Soils, especially in surface layer, may exist at unsaturated state, whose void is occupied with water and air. Because of the complicated mechanical behavior of unsaturated soil, the application of constitutive models for unsaturated soil in numerical analysis for practical engineering problem is much less than those for saturated soil. However, an unsaturated soil is not a special soil but the soil whose degree of saturation, a state variable, is smaller than 1.0. It is necessary to establish a constitutive model that can describe both unsaturated state and saturated state under any stress condition so that it can properly evaluate the deformation and the failure behavior, such as slope failure, due to increase and decrease of the water content of the geomaterials.

Since the pioneering work by Reference [1], in which Barcelona Basic Model (BBM) was proposed and regarded as one of the basic models for unsaturated soil, a number of elastoplastic constitutive models have been proposed to describe the mechanical behavior of unsaturated soil [2]-[7]. In recent years, some constitutive models, using the effective stress (or skeleton stress) and the degree of saturation as the independent state variables, have also been established by Reference [8]-[10]. The model has been verified by some tests but is not sufficient enough. In order to make it applicable in practical engineering, it is needed to give an appropriate unifying evaluating method for the parameters involved in the proposed constitutive model, which is proved to be time consuming and needs advanced testing technique. The main purpose of this research is to find out the fundamental behaviors of unsaturated soil, especially the influence of the degree of saturation with laboratory test, and establish a unified constitutive model for unsaturated/saturated geomaterials. In this paper, uniaxial compression tests were firstly conducted on unsaturated Masado under constant degree of saturation condition and then the performances of previously proposed elastoplastic constitutive model for unsaturated soil were checked carefully with the test data.

## 2. OUTLINE OF ELEMENTARY TESTS AND ANALYSES

### 2.1 Uniaxial Compression Test of Unsaturated Masado under Constant Degree of Saturation Condition

Masado, typical decomposed granite that widely distributed in southwest Japan, was used as the specimen in uniaxial compression test. Physical properties and the grain size distribution curve of Masado, which has been sieved to the soil particles less than 2.0 mm, are shown in Table 1 and Fig. 1. The specimens (60 mm in diameter and 10 mm height) were compacted in one layer directly into the oedometer ring at target void ratio

$e$ = 0.50. Based on the compaction curve of Masado, as shown in Fig. 2, as a fundamental test condition, the target water content was set to $w$ = 9, 12, 15, 18, 21%, in which the optimum water content is $w_{opt}$ = 15%, and dry side is $w$ = 9, 12%; wet side is $w$ = 18, 21%.

Table 1 Physical properties of Masado

| | Unit | Value |
|---|---|---|
| Specific gravity, $\boldsymbol{G_s}$ | - | 2.66 |
| Liquid limit, $\boldsymbol{w_L}$ | % | NP |
| Plasticity index, $\boldsymbol{I_p}$ | % | NP |
| Maximum dry density, $\boldsymbol{\rho_d}$ | g/cm$^3$ | 1.85 |
| Optimum water content, $\boldsymbol{w_{opt}}$ | % | 13.7 |

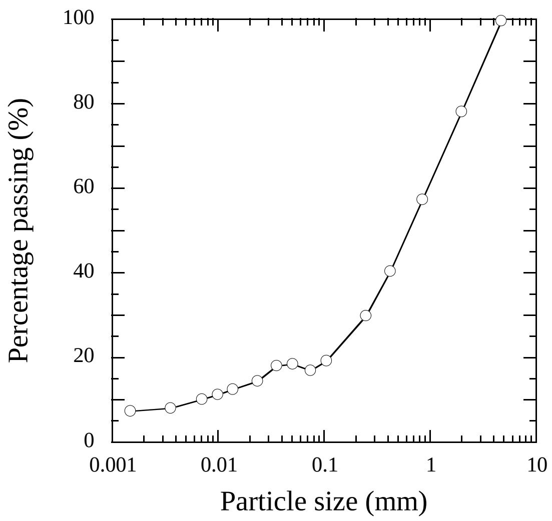


Fig. 1 Grain size distribution curve of Masado

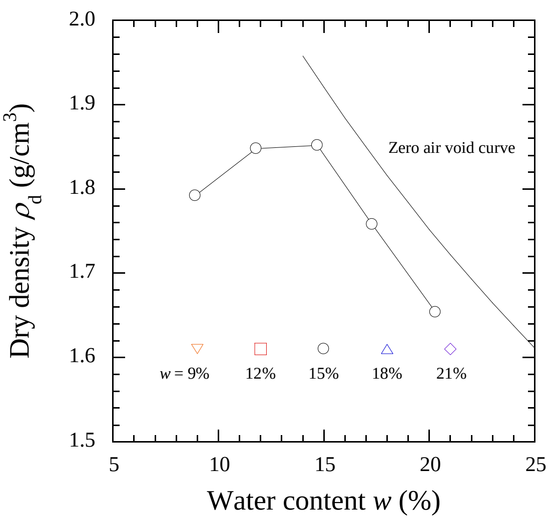


Fig. 2 Compaction curve of Masado

In order to verify the influence of the initial degree of saturation on the mechanical behavior of Masado, uniaxial compression tests of the unsaturated Masado under the condition of constant degree of saturation with pressuring method were conducted. Fig. 3 shows the test apparatus for uniaxial compression tests. Pore water pressure was controlled and measured through a ceramic disc with an air entry value (AEV) of 1500 kPa, installed at the bottom of the specimen. Pressure/volume controller (PVC, GDS product), which can control the pore pressure or the pore water volume according to the usage, was implemented in the uniaxial compression test in order to be able to conduct the tests under constant degree of saturation condition. On the other hand, pore air pressure was applied using a pneumatic regulator at the top of the specimen.

In the tests, the net stress ($\sigma_v^{net} = \sigma_v$ - $u_a$, $\sigma_v$: total stress, $u_a$: pore air pressure) was applied to 20 kPa at the beginning, and then the suction ($s = u_a$ - $u_w$, $u_w$: pore water pressure) was applied to 100 kPa ($u_a$ = 500 kPa, $u_w$ = 400 kPa). After confirming that the drainage of water from the specimen became stable, switch on the control unit to keep the degree of saturation being constant, and simultaneously start applying the net stress from 20 kPa to 965 kPa with a rate of 60 kPa/h. The control technique proposed by Reference [11], which can adjust the drainage of water from the specimen by increasing/decreasing the pore water pressure under constant pore air pressure, was adopted in order to keep the degree of saturation being constant. The expression for the controlling of the degree of saturation is given as,

$$dV_w - S_{r(init.)}dV_v = 0 \quad (1)$$

where, $dV_w$ is the change in pore water volume, $S_{r(init.)}dV_v$ is the product of the initial degree of saturation and the change in volume of the voids. In the uniaxial compression test, $dV_w$ is obtained from the product of the cross-sectional area and the vertical displacement of the specimen. Meanwhile, $dV_w$ is adjusted using the PVC to keep the degree of saturation.

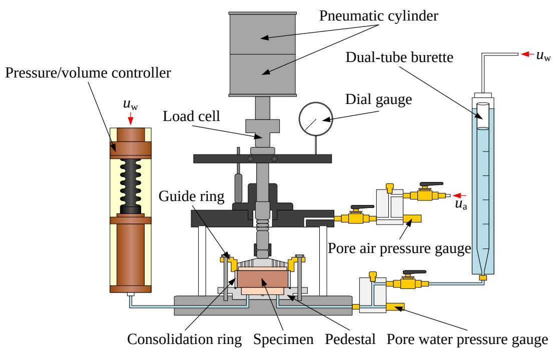


Fig. 3 Schematic illustration of the test apparatus for uniaxial compression tests

### 2.2 Performance of Proposed Elastoplastic Constitutive Model for Unsaturated Soil

In the simulations, an elastoplastic constitutive model proposed by Reference [8], was used in theoretical simulation of the tests. The model is based on the modified Cam-Clay model [12], in which the Bishop-type skeleton stress ($\sigma'' = \sigma_v^{net} + S_r s$) and the degree of saturation are used as the state variables, can take into consideration wetting-drying moisture hysteresis of an unsaturated soil. The material parameters for the constitutive model and the parameters of moisture characteristic curve (MCC) are listed in Table 2. Here, it is assumed that normally consolidated line in unsaturated state (*N.C.L.S.*) is parallel to the normally consolidated line in saturated state (*N.C.L.*) but in a higher position than *N.C.L.*, which means that under the same mean skeleton stress, the unsaturated soil can keep higher the void ratio than those of saturated soil. In other words, compression index $\lambda$ is always constant with the degree of saturation. The *N.C.L.S.* and the *C.S.L.S.* are given in the following relations as,

$$N.C.L.S.:\ \ e = N(S_r) - \lambda \ln \frac{p}{p_r},\ \ \left(\eta = \frac{q}{p} = 0\right) \quad (2)$$

$$C.S.L.S.:\ \ e = \Gamma(S_r) - \lambda \ln \frac{p}{p_r},\ \ \left(\eta = \frac{q}{p} = M\right) \quad (3)$$

where, $N(S_r)$ and $\Gamma(S_r)$ are the void ratios at *N.C.L.S.* and *C.S.L.S.* under the reference mean skeleton stress ($p_r$ = 98 kPa) and certain degree of saturation. $p$ and $q$ are the mean skeleton stress and the second invariant of deviator skeleton stress tensor. $M$ is the stress ratio at critical state and has the same value for saturated and unsaturated states. $N(S_r)$ is expressed as,

$$N(S_r) = N + \frac{N_r - N}{S_r^s - S_r^r}\left(S_r^s - S_r\right) \quad (4)$$

where, $S_r^s$ is the saturated degrees of saturation and $S_r^r$ is the residual degrees of saturation, which have definite physical meaning and can be determined by water retention test easily. $N_r$ is the void ratio at *N.C.L.S.* under the reference mean skeleton stress when the degree of saturation is at residual dry state.

## 3. RESULTS AND DISUCUSSION

Fig. 4 shows the comparison between the test data and the simulated data of the water retention test based on the pressuring method. In the figure, it is very clear that there is a good agreement between the test data and the simulated data. It should be noted that the material parameters for elastoplastic constitutive model and the parameters of MCC have been unified in all simulation cases.

Table 2 Material parameters for constitutive model and parameters of MCC of Masado

| | | |
|---|---|---|
| **Material parameters for constitutive model** | Compression index, $\lambda$ | 0.089 |
| | Swelling index, $\kappa$ | 0.0080 |
| | Critical state parameter, $R_{cs}$ | 4.01 |
| | Void ratio, $N$ ($p'$ = 98 kPa on *N.C.L.*) | 0.69 |
| | Poisson's ratio, $\nu$ | 0.25 |
| | Parameter of overconsolidation, $a$ | 60.0 |
| | Parameter of suction, $b$ | 20.0 |
| | Parameter of overconsolidation, $\beta$ | 2.0 |
| | State variable of overconsolidation, $\rho_e$ | 0.040 |
| | Void ratio, $N_r$ ($p'$ = 98 kPa on *N.C.L.S.*) | 0.71 |
| **Parameters of moisture characteristic curve** | Saturated degrees of saturation, $S_r^s$ | 0.85 |
| | Residual degrees of saturation, $S_r^r$ | 0.27 |
| | Parameter corresponding to drying AEV (kPa), $S_d$ | 5.0 |
| | Parameter corresponding to wetting AEV (kPa), $S_w$ | 1.0 |
| | Initial stiffness of scanning curve (kPa), $k_{sp}^e$ | 2000 |
| | Parameter of shape function, $c_1$ | 0.014 |
| | Parameter of shape function, $c_2$ | 0.060 |
| | Parameter of shape function, $c_3$ | 50.0 |

Note: *N.C.L.* is the normally consolidated line at saturated state; *N.C.L.S.* is the normally consolidated line at unsaturated state.

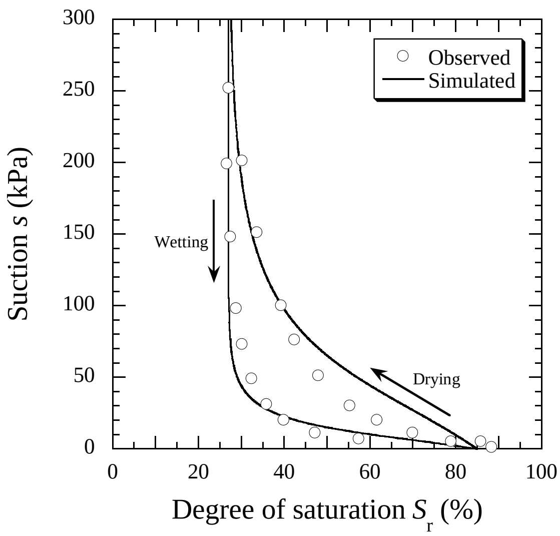


Fig. 4 Moisture characteristic curve of Masado

Table 3 Physical properties at different stages

| Target water content | Initial condition | | | Stage beginning | | | Stage end | | |
|---|---|---|---|---|---|---|---|---|---|
| | $w_0$ (%) | $e_0$ | $S_{r0}$ (%) | $w$ (%) | $e$ | $S_r$ (%) | $w_f$ (%) | $e_f$ | $S_{rf}$ (%) |
| $w$ = 9% | 8.5 | 0.642 | 35.2 | 7.4 | 0.613 | 31.6 | 5.3 | 0.453 | 31.1 |
| $w$ = 12% | 12.1 | 0.650 | 49.3 | 9.6 | 0.625 | 40.7 | 7.3 | 0.479 | 41.0 |
| $w$ = 15% | 15.0 | 0.650 | 61.7 | 11.0 | 0.625 | 46.8 | 8.0 | 0.446 | 47.7 |
| $w$ = 18% | 17.8 | 0.651 | 72.8 | 11.1 | 0.613 | 48.0 | 7.9 | 0.441 | 48.8 |
| $w$ = 21% | 21.8 | 0.660 | 87.8 | 11.5 | 0.623 | 49.0 | 8.5 | 0.455 | 49.6 |

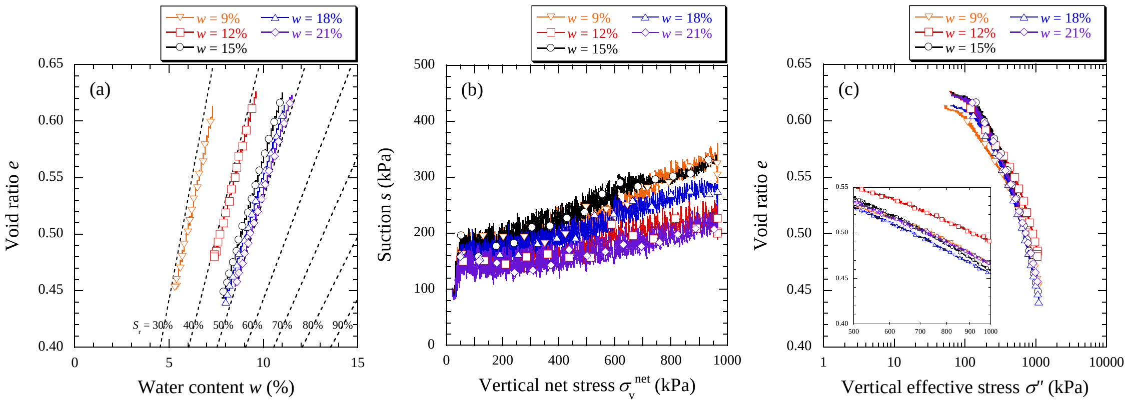


Fig. 5 Uniaxial compression tests results under constant degree of saturation; (a) $e$ vs. $w$, (b) $s$ vs. $\sigma_v^{net}$, (c) $e$ vs. $\sigma''$

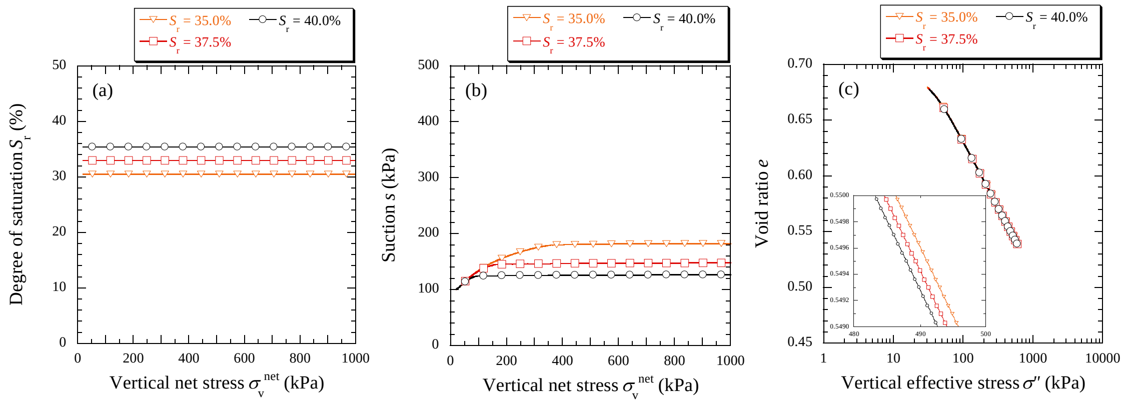


Fig. 6 Simulated results of uniaxial compression tests under constant degree of saturation; (a) $S_r$ vs. $\sigma_v^{net}$, (b) $s$ vs. $\sigma_v^{net}$, (c) $e$ vs. $\sigma''$

Fig. 5 shows the test results obtained from the uniaxial compression tests under constant degree of saturation. The constant degree of saturation lines were drawn based on $eS_r = wG_s$. See Table 3 for the physical properties of Masado in each test. From these results, it is known that the water content declines with the decreasing of the void ratio, while the degree of saturation is kept constant. In Fig. 5(b), the suction is built up along with the increasing of the net stress, and the increment of suction is depending on the initial degree of saturation. In two cases ($w$ = 18, 21%), even the degrees of saturation at the beginning of suction are almost the same, the specimen with higher initial degree of saturation shows lower increment of suction. As shown in Fig. 5(c), the *N.C.L.* of dry side ($w$ = 9, 12%) is located above those of wet side ($w$ = 18, 21%) and the optimum water content ($w_{opt}$ = 15%). In other words, the lower the initial degree of saturation is, the more the *N.C.L.* will be shifted upward.

Fig. 6 shows the simulated results of uniaxial compression tests under constant degree of saturation. The simulated loading path is the same as that of the tests. In the simulations, the target degrees of saturation were set into three cases ($S_r$ =

35.0, 37.5, 40.0%), which are corresponded to the value located in the inner side of MCC from Fig. 4. From these results, the degree of saturation decreases due to the suction, after then the degree of saturation is kept constant by the increment of the net stress. In Fig. 6(b), it can be observed that the simulated data is on the whole agreed with the test data. As shown in Fig. 6(c), the proposed model can qualitatively describe the parallel upward moving of *N.C.L.* along with the decreasing of the initial degree of saturation, though quantitatively there still exists some discrepancies.

## 4. CONCLUSION

In this paper, uniaxial compression tests were firstly conducted to verify the importance of proper selection of the state variables, that is, the skeleton stress and the degree of saturation in the proposed constitutive model for unsaturated soil. Then, corresponding theoretical simulation were conducted to compare with the test results based on a proper MCC. The following conclusions can be drawn:

1. By using the skeleton stress as a state variable, the influence of the degree of saturation on the deformation and the strength of unsaturated soil can be identified.
2. In the simulations, it is proved that the proposed model can describe the mechanical behaviors of unsaturated soil under different degrees of saturation to some extent on the whole. However, the comparison between the test data and the simulated data has not been checked directly yet, because the range of the degree of saturation depending on the suction can be applied for the simulation is limited. In future study, it is necessary to determine the parameters, especially for MCC in a more accurate way, based on more test cases.

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