Test on mechanical behavior of unsaturated Masado using triaxial compression test under constant degree of saturation condition

Yuhei Kurimoto[2], S. Okino[1], T. Tsunemoto[1], X. Xi[1], H. Iwai[1], and F. Zhang[1]

[1] Department of Civil Engineering, Nagoya Institute of Technology, Gokiso-cho, Showa-ku, Nagoya, 466-8555, Japan.
[2] Institute of Technology, Shimizu Corporation, Etchujima, Koto-ku, Tokyo, 135-8530, Japan.

ABSTRACT

Generally speaking, most of the geomaterials may exist in unsaturated state. The mechanical and hydraulic behaviors of unsaturated soil are more complicated than those of saturated soil. Therefore, in order to evaluate the deformation and failure behavior, such as slope failure, due to increase and decrease of the water content of the geomaterials, it is necessary to clarify the mechanical behavior of unsaturated soil. The main purpose of this research is to find out the fundamental behaviors of unsaturated soil, especially the influence of the degree of saturation. In this paper, triaxial compression tests under constant-degree-of-saturation (CDS) condition were conducted to investigate the fundamental behaviors of unsaturated soil and the importance of proper selection of the state variables, that is, the skeleton stress and the degree of saturation in the constitutive model for unsaturated soil.

Keywords: unsaturated soil, degree of saturation, skeleton stress, triaxial test

1 INTRODUCTION

Soils, especially in the surface layer, may exist in unsaturated state, whose void is occupied with the water and the 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 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.

Since the pioneering work by Alonso et al. (1990), 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. Stress variables of these constitutive models, such as the total stress or net stress, Bishop's effective stress (Bishop, 1959), differ according to the authors. 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 Zhang and Ikariya (2011), Zhou et al. (2012a, 2012b).

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, a decomposed granite, called as Masado with high permeability, was tested with triaxial compression tests under constant-degree-of-saturation (CDS) condition in order to investigate rational state variable in unsaturated soil constitutive model.

2 LABORATORY TESTS

2.1 Test specimen

Masado, typical decomposed granite that is widely distributed in southwest Japan, was used as the test material in the triaxial compression tests. Because the permeability of Masado is much larger than that of silty clay commonly used in unsaturated tests, the testing time required for unsaturated tests could significantly shorten. Therefore, Masado was selected as the test material for the unsaturated tests under CDS condition. 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. Moreover, the compaction test was also conducted to find the optimum moisture content for specimen preparation as shown in Fig. 2.

The specimens for the triaxial compression tests have the same initial moisture content w_0 = 15%, and were 100 mm in height and 50 mm in diameter. Surplus soils from cut material were used in measuring the water content. The specimens were prepared by static compaction method (static pressure was around 1.0 MPa) for three layers and the target void ratio was 0.65.

Table 1. Physical properties of Masado.

	Unit	Value
Liquid limit, w_L	%	Non-Plastic
Plasticity index, I_p	%	Non-Plastic
Specific gravity, G_s	-	2.66
Standard Proctor Maximum dry density, ρ_c	Mg/m^3	1.85
Standard Proctor Optimum water content, w_{opt}	%	13.7

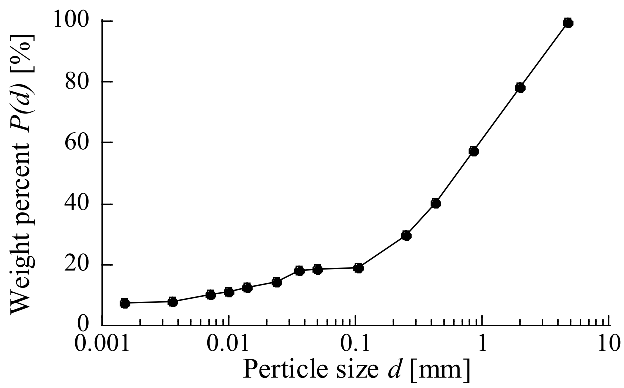


Fig. 1. Grain size distribution curve of Masado.

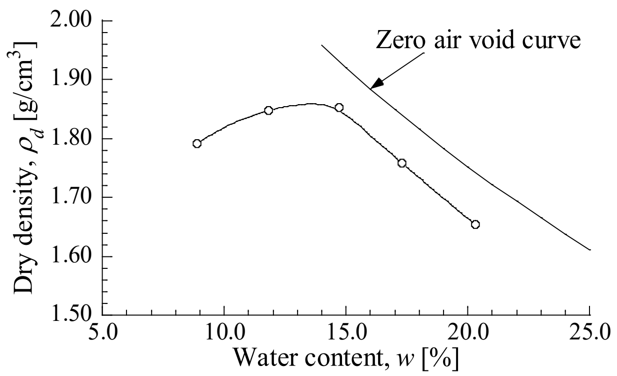


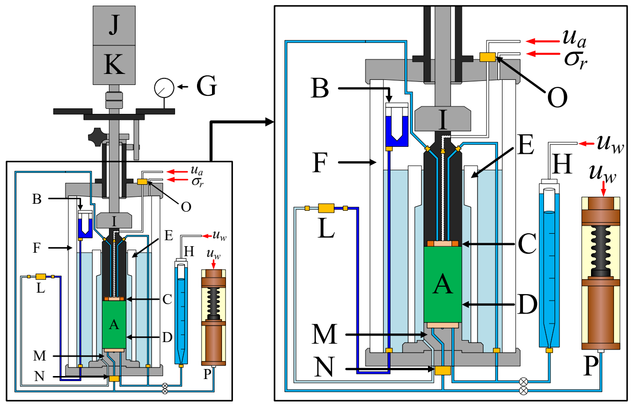
Fig. 2. Compaction curve of Masado.

2.2 Test apparatus

The unsaturated triaxial test apparatus shown in Fig. 3 was used in the compression tests. Axis-translation method is also adopted to control the suction. Pressure/Volume Controller (PVC) and pore water piezometer are connected to the specimen by ceramic disk. The air entry value of the ceramic disk used in the triaxial test is 1.5 MPa (15 bar). When the air pressure is less than the AEV of the ceramic disk, only water is allowed to pass through the ceramic disk. Therefore, the pore water pressure of the specimen can be measured by the PVC and the pore water piezometer.

In the unsaturated triaxial test apparatus, four kinds of pressures, that is, the axial pressure, the confining pressure, the pore air pressure and the pore water pressure can be controlled manually or automatically by PC with E-P regulators under the primary air pressure of 1.0 MPa.

The main feature of the test apparatus is that both the porous stone (pore air pressure) and the ceramic disks (pore water pressure) are embedded both in up and lower axial caps shown in Fig. 4, and the test time can be shortened to one fourth under double-end-face drainage condition. To obtain the volume change of unsaturated specimens, differential manometer is utilized to measure the pressure difference between the burette with the standard water surface and the inner chamber.



(A) Specimen, (B) Standard burette, (C) Axial cap, (D) Rubber membrane, (E) Inner cell, (F) Outer cell, (G) Dial gauge, (H) Double burette, (I) Load cell, (J) Axial cylinder (top), (K) Axial cylinder (bottom), (L) Differential manometer, (M) Pedestal, (N) Pore water piezometer, (O) Pore air piezometer, (P) GDS PVC

Fig. 3. Outline of unsaturated triaxial test apparatus.

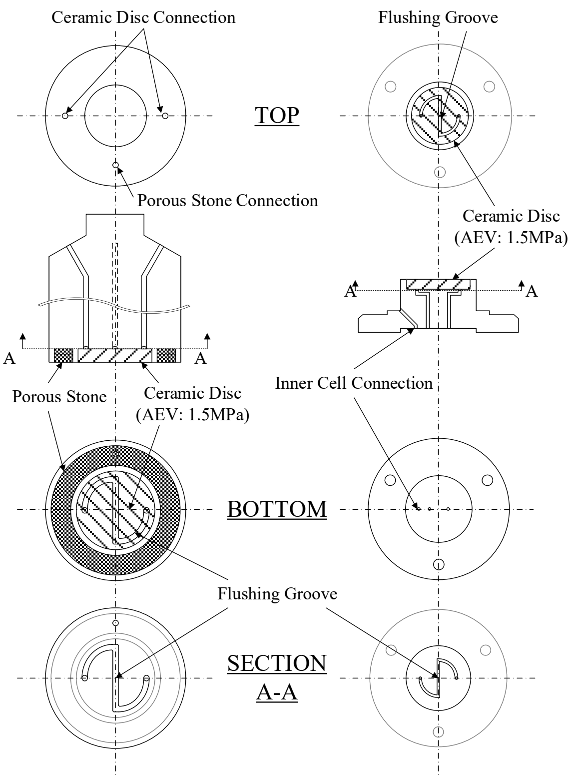


Fig. 4. Top and bottom pedestals based on Barrera (2002) used in this research.

2.3 Test procedure

Fig. 5 shows the stress paths of the unsaturated specimens reached the initial stress state of triaxial compression tests under CDS condition, in which the specimens consolidated at different suction and confining stresses to reach the different initial states. During the shear stage, the degree of saturation was controlled to keep constant by the method proposed by Burton et al. (2016). The pore air pressure was kept constantly and the water content of sample was adjusted by changing the pore water pressure through PVC with the following equation,

$$dV_{\mathrm{w}} - S_{\mathrm{r(init.)}} \cdot dV_{\mathrm{v}} = 0 \quad (1)$$

where dV_{w} is pore water volume change, $S_{\mathrm{r(init.)}}$ is the initial degree of saturation and dV_{v} is the sample volume change. The pore air pressure was kept stable, and the water content of specimen was adjusted by changing the pore water pressure through a PVC during the shear stage.

In all cases, the specimens were under drained and vented conditions and the shear rate was 0.0025%/min and the maximum deviatoric strain was set to be 15%. The confining stress (σ_3^{net}) was kept constant during the shear stage.

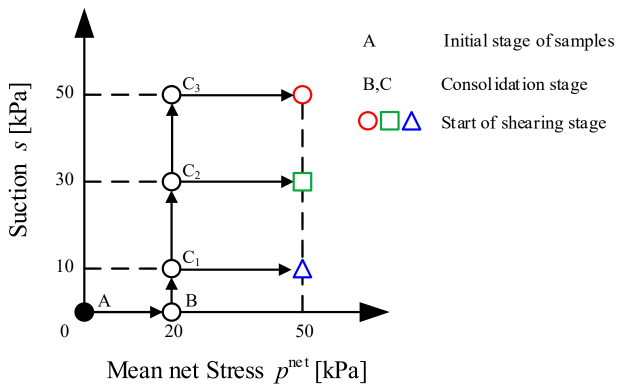


Fig. 5. Stress path of unsaturated sample before reached initial stress state of triaxial compression test under CDS condition.

2.4 Test results

Table 2 shows the physical properties of the specimens in each test. As shown in Table 2, the higher the initial suction is, the smaller the degree of saturation at the beginning of shear stage will be.

Fig. 6 shows the relationship between the degree of saturation and the deviatoric strain. As shown in Fig. 6, the degree of saturation was kept constant quite well during the shear stage, indicating that the triaxial compression test under CDS condition was successful.

Fig. 7 shows the stress-strain-dilatancy relations of triaxial compression tests. As shown in Fig. 7 (a), it is known that the smaller the degree of saturation is, the higher the peak stress will be, while the stress ratio at critical stress was almost the same. Meanwhile, the volumetric strain transformed from contraction to dilation as the degree of saturation decreased. From Fig. 7 (b), basically, the smaller the degree of saturation is, the higher the peak stress ratio will be but the residual stress ratio was almost the same in all cases.

Fig. 8 shows the stress paths under CDS condition, in which the stress paths were expressed with net stress and skeleton stress. The black solid line in the figure is the critical state line (*C.S.L.*) estimated from the test results. If pay attention to the critical state line, only in the case of skeleton stress space, the stress paths will finally reach the same critical state lines. Therefore, by using the degree of saturation and the skeleton stress as the state variables, the mechanical behavior of unsaturated soil can be described more reasonably with unified parameters.

Fig. 9 shows the relationship between the suction/water volume change and the deviatoric strain. It is known from Fig. 9 (a) that the suction changed during the shearing stage. It experienced a small reduction firstly and then increased until the peak. After the peak, the suction decreased gradually and finally approached to a constant value. As shown in Fig. 9 (b), it is interesting to find that the drainage discharge of specimens changed with the increase of the compression loading, and had the same tendency with the dilatancy that the drainage discharge changed from positive to negative during the shearing.

Table 2. Values of state variable at different stages.

Case	Initial condition			Stage beginning			Stage end		
	w_0 (%)	e_0 (-)	S_{r0} (-)	w (%)	e (-)	S_r (-)	w_f (%)	e_f (-)	S_{rf} (-)
s50-1	15.0	0.611	0.65	11.0	0.558	0.53	11.6	0.586	0.53
s50-2	14.3	0.629	0.61	11.1	0.571	0.52	11.2	0.577	0.52
s30-1	14.8	0.622	0.63	11.4	0.569	0.53	11.6	0.579	0.53
s30-2	14.7	0.608	0.64	11.3	0.536	0.56	11.4	0.541	0.56
s10-1	14.4	0.610	0.63	11.9	0.549	0.57	11.9	0.550	0.57
s10-2	14.3	0.612	0.62	12.7	0.557	0.60	12.6	0.556	0.60

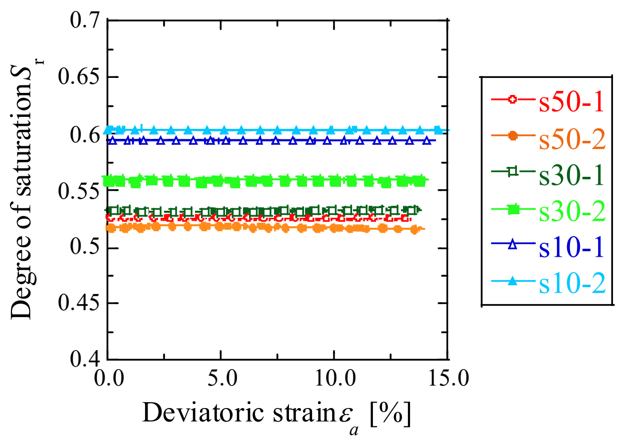


Fig. 6. Variation of degree of saturation.

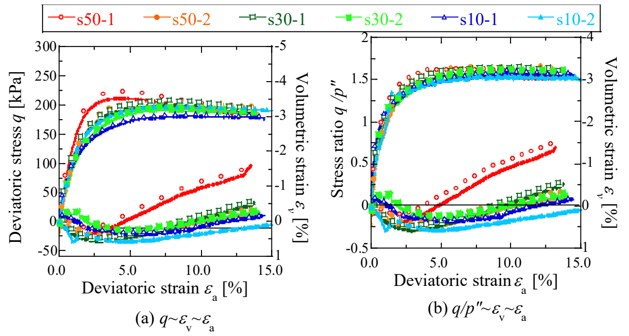


Fig. 7. Stress-strain-dilatancy relations of Masado in unsaturated triaxial compression tests under CDS condition.

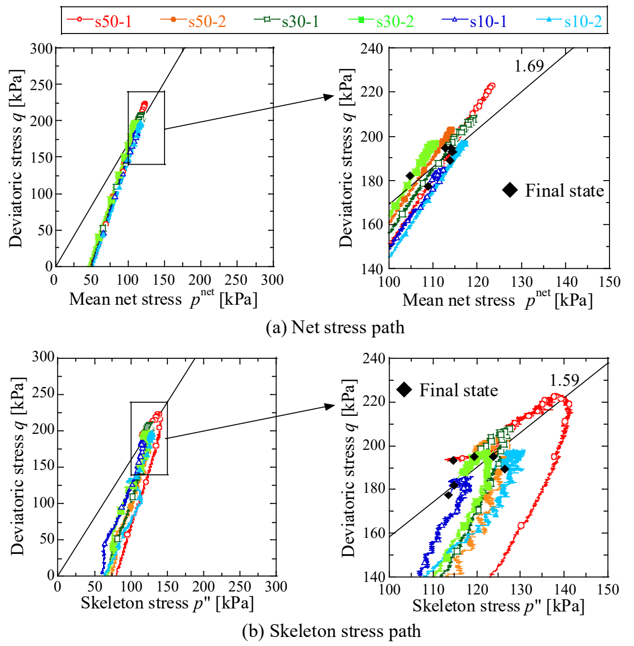


Fig. 8. Stress paths of Masado in unsaturated triaxial compression tests under CDS condition.

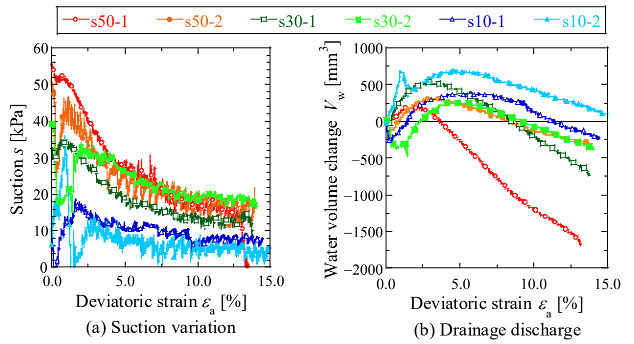


Fig. 9. Test results of Masado in unsaturated triaxial compression tests under CDS condition.

3 CONCLUSION

In this paper, triaxial compression tests under constant-degree-of-saturation (CDS) condition were 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 (Zhang and Ikariya, 2011) for unsaturated soil. The following conclusions can be drawn:

1. The smaller the degree of saturation is, the higher the peak stress will be but the stress ratio at critical stress was almost the same.

2. The volumetric strain transformed from contraction to dilation as the degree of saturation decreased.

3. By using the degree of saturation and the skeleton stress as the state variables, the mechanical behavior of unsaturated soil can be described more reasonably with unified parameters. In future study, in order to enhance the reliability of these results, more test cases should be conducted.

4. The drainage discharge of specimens had the same tendency with the dilatancy that the drainage discharge changed from positive to negative during the shearing.

ACKNOWLEDGEMENTS

The authors would like to thank Mr. Seiji Kobayashi and Mr. Xiaoie Qiu, former students of Nagoya Institute of Technology, for their support in conducting experiments.

REFERENCES

Alonso, E. E., Gens, A. and Josa, A. (1990). A constitutive model for partially saturated soils. Géotechnique, 40(3), 405-430.

Barrera, M. (2002). Estudio experimental del comportamiento hidromecánico de suelos colapsables. PhD Thesis, Universitat Politècnica de Catalunya, Barcelona, Spain.

Bishop, A.W. (1959). The principle of effective stress. Tecnisk Ukevblad, 39, 859-863.

Burton, G. J., Pineda, J. A., Sheng, D., Airey, D. and Zhang, F. (2016). Exploring one-dimensional compression of compacted clay under constant degree of saturation paths. Géotechnique, 66(5), 435-440.

Zhang, F. and Ikariya, T. (2011). A new model for unsaturated soil using skeleton stress and degree of saturation as state variables. Soils and Foundations, 51(1), 67-81.

Zhou, A., Sheng, D., Sloan, S. W., Gens, A. (2012a). Interpretation of unsaturated soil behaviour in the stress-saturation space: II: Constitutive relationships and validations. Computers and Geotechnics, 43, 111-123.

Zhou, A., Sheng, D., Sloan, S. W., Gens, A. (2012b). Interpretation of unsaturated soil behaviour in the stress - Saturation space, I: Volume change and water retention behavior. Computers and Geotechnics, 43, 178-187.