SHEAR STRENGTH CHARACTERISTICS OF UNSATURATED VOLCANIC CLAYEY SOILS

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1. INTRODUCTION

An increasing number of cases of fill failure due to an earthquake or heavy rain have been reported. Therefore, the mechanical properties of compacted soils (unsaturated soils) have been situated a focus of attention. Constitutive equations were proposed concerning saturated and dry soils. Some models have been applied for practical purposes. For unsaturated soils, Alonso[1)], Kohgo [2)], Karube[3)] and others have been proposed their models. These studies are still ongoing toward practical application[4)]. Discussing the strength and deformation characteristics of unsaturated soils requires consideration of the degree of saturation and matrix suction, which affect the characteristics. These two parameters have been known to have a great impact on the mechanical behavior of unsaturated soils, this mechanical behavior is directly associated with the strength and deformation characteristics of soils. In this research the consolidated undrained shear test using statically compacted Kanto loam and Koroboku with varying degrees of saturation was conducted with a tri-axial compression test apparatus for saturated soils. To compare the test results with analytical data, an elasto-plastic finite element analysis is carried out to define the shear test result by unsaturated soil which is composed of three phases, namely soil particle, water and air was regarded as a two-phase system of soil particle and pore fluid (water and air) while considering compressibility.

2. SAMPLE AND TEST METHOD

For the sample, Kanto loam and Kuroboku are collected in Tokai University (natural water content: 100 to 110%) was used. A sample with an initial degree of saturation *Sr* of 40% or higher was made by drying the soil for two days in a 110°C drying furnace, making it pass a 2 mm sieve and adding a designated amount of water. The physical properties of the sample are listed in Table 1. The sample was placed in three layers in a 5 cm diameter divided mold with a height of 12.5 cm. An overburden pressure of 200 kPa was applied in each layer to make a specimen with a height of 10 cm. The specimen was set at a triaxial compression test apparatus for saturated soils (pore pressure was measured at the lower pedestal at the bottom of the sample), consolidated at a designated isotropic consolidation pressure σ for 24 hours and cell pressure increment Δσ was applied in stages to measure of pore pressure coefficient B (= $\Delta u/\Delta\sigma$, referred to as B value). Subsequently, shear tests were conducted both under drained and undrained conditions. Undrained shear rate was 0.08 mm/min.

Table1 Physical property of soils

Sample	ρs (g/cm³)	w_L (%)	w_P (%)	Grading (%) Clay	Silt	Sand
Kanto Loam	2.897	104.9	72	20.9	42	37.1
Kuroboku	2.349	NP	NP	7	47	46

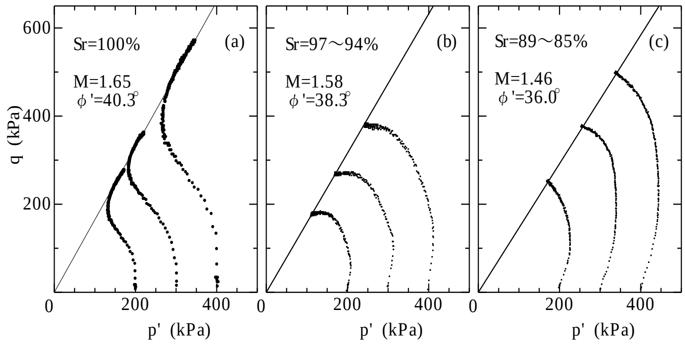

Fig. 1 Results of triaxial consolidated un-drained (CU) test

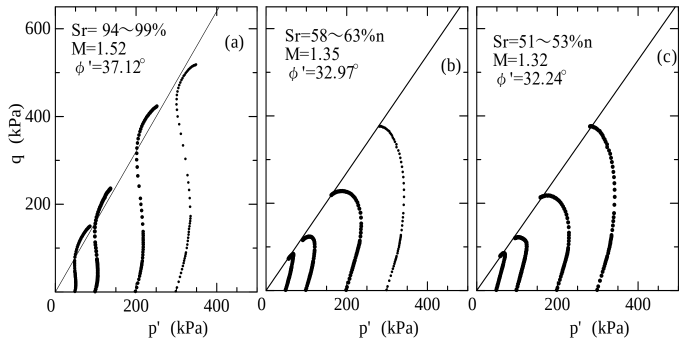

Fig.2 Results of triaxial consolidated un-drained (CU) test

3. RESULTS OF TRIAXTIL CU TEST & EFFICT OF INITIAL *Sr*, ON *cu*, *ϕu*, *ϕ'* AND *C'*.

Figure 1 and 2 shows effective stress paths for both kind of soil. The degree of saturation S_r shown in the Figure 1 is the value before consolidation. The effective stress paths with a degree of saturation of 100% in Figure 1(a) shows that dilatancy changed from negative to positive near the critical state line (CSL) and deviator stress increased subsequently along CSL. In figure 2a also the effective stress paths with the saturation of 94.4~99.4% shows dilatancy changed from negative to positive near CSL. Figure 1(b) and (c) were obtained from unsaturated specimens with S_r of 97 through 94% ($\Delta u/\Delta\sigma$ = 0.54 to 0.48) and S_r of 89 through 85% ($\Delta u/\Delta\sigma$ = 0.15 to 0.1). It is evident that the angle of shear resistance, ϕ' decreased with the degree of saturation. When the initial degree of saturation is approximately equal, shape of the effective stress paths indicate similar shape independent on consolidation pressure. In both figures, pore pressure increases with the increase in deviator stress and effective stress path heads for CSL.

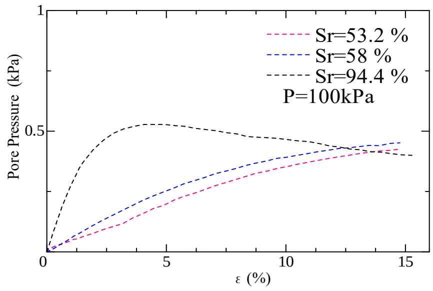

Fig. 3 Axial strain vs. pore pressure of KUROBOKU

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In figure 3 shows relation between pore pressure against strain. Its shows the behavior of unsaturated soil close to saturation, with increase in saturation pore water behavior also changed.

4. B VALUE OF UNSATURATED SOIL

Specimens with different degree of saturation were consolidated in 200 kPa. Then, cell pressure increment $\Delta\sigma$ was applied gradually under the undrained condition. The relationship between the $\Delta\sigma$ and measured pore pressure increment Δu is shown in Figure 4. Pore pressure identical with cell pressure increment occurred from the initial stages of loading at S_r =100 %. With the decrease of S_r, pore pressure that was generated in the early stages of loading decreased. In unsaturated specimens, the incidence of pore pressure increases with increasing of the$\Delta\sigma$. Then, B value after the cell pressure increasing amount exceeds 100 kPa is closer to 1. It is thus evident that B value in unsaturated soils would vary due to the increase of cell pressure at the time of measurement.

Figure 5 shows the relationship between the $\Delta\sigma$ obtained from Figure 4 and *B* value. *B* value under the undrained condition is expressed by equation (1) according to Naylor [2]. Equation (1) is transformed into (2), where K_f is the volumetric modulus of elasticity of saturated soil element, *K'* is the volumetric modulus of elasticity for effective stress and α is a coefficient indicating the compressibility of pore fluid.

$$B=\frac{K_f}{K'+K_f} \qquad (1) \qquad , \qquad K_f=\frac{B}{1-B}K'=\alpha K \quad \left(\alpha=\frac{B}{1-B}\right) \qquad (2)$$

Figure 6 shows the relationship between *B* value and coefficient α in equation (2). In Figure 7, *B* value in Figure 5 was replaced with coefficient α using equation (2). It is evident that coefficient α increased rapidly with a smaller stress increment at a greater degree of saturation. It was assumed that the relationship shown in Figure 7 was applicable to the increase in *B* value due to the increase of mean stress *p* during shearing process, and calculations were made to reproduce the results of triaxial consolidated undrained test.

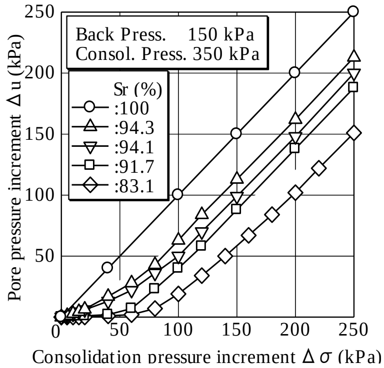

Fig. 4 Cell pressure increment and pore pressure

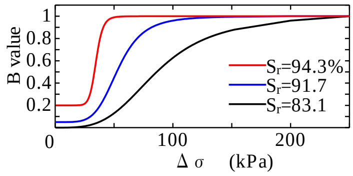

Fig. 5 Cell pressure increment and *B*

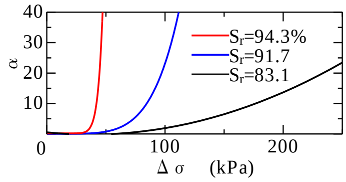

Fig. 6 α and *B* value

5. CALCULATION TO REPRODUCE THE RESULTS OF TRIAXTIAL CU TEST

Calculations were made by incorporating the relationship shown in Figure 7 into CRISP, an elasto-plastic finite element analysis program for a modified Cam Clay model. The constants required for calculation are listed in Table 2. Poisson's ratio was estimated from coefficient of earth pressure at rest K_0, which was estimated from the angle of shear resistance ϕ'. The results of calculations to reproduce effective stress paths using CRISP are shown in Figure 6 by solid lines. The modified Cam Clay model could reproduce no reverse of dilatancy observed in Figure 7(a) under the saturated condition. The results of calculations, however, could reproduce test results in cases with varying degrees of saturation and consolidation pressures up to the critical sate line.

Table 2 Constants for calculation

λ	κ	Γ-1	K_0	ν
0.169	0.021	3.307	0.42	0.261

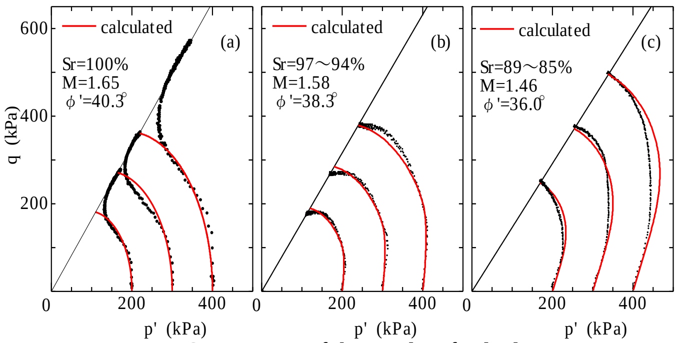

Fig. 7 Comparison of the results of calculations to reproduce triaxial tests consolidated undrained test and test results

6. CLOSING REMARK

In this study unsaturated soil was assumed to be a two-phase system of pore fluid (water and air) with compressibility taken into consideration and soil particles. Triaxial apparatus for saturated soil used to examine the mechanical behavior of unsaturated soil. By considering the calculation of B value changing in the degree of saturation and shearing process we can simulate the behavior of unsaturated stress paths. The examination of stress-strain relationship will be required in the future.

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