A consideration on the shear strength characteristics of unsaturated Kanto loam

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1. INTRODUCTION

An increasing number of cases of fill failure due to an earthquake or heavy rain have been recently reported. Therefore, the mechanical properties of compacted soils (unsaturated soils) have been situated a focus of attention. Whenever, the height of embankments exceeds than 100m, stability of the slope after and during construction is very important to be considered. It can be say that, Japanese soil in a state of high temperature and humidity is in the unsaturated state close to saturation. So, it is practically necessary to understand the strength characteristics of the unsaturated soils with a high degree of saturation. Constitutive equations have been proposed concerning saturated and dry soils. Some models have been applied for practical purposes. For unsaturated soils, Alonso[1)] and others have proposed their models. These studies are still ongoing toward practical application. Discussing the strength and deformation characteristics of unsaturated soils requires considering 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.

To determine the strength and deformation characteristics of unsaturated soils, it requires testing unsaturated soils which is capable to control suction, air emission and drainage. In this paper, the consolidated undrained shear test using statically compacted Kanto loam with varying degrees of saturation was conducted with a triaxial compression test apparatus for saturated soils. To compare the test results with analytical data, an elasto-plastic finite element analysis is carried out to reproduce 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.

Table1 Physical property of Kanto Loam

Sample	ρs	w_L	w_P	Grading(%)		
	(g/cm³)	(%)	(%)	Clay	Silt	Sand
Kanto Loam	2.897	104.9	72	20.9	42	37.1

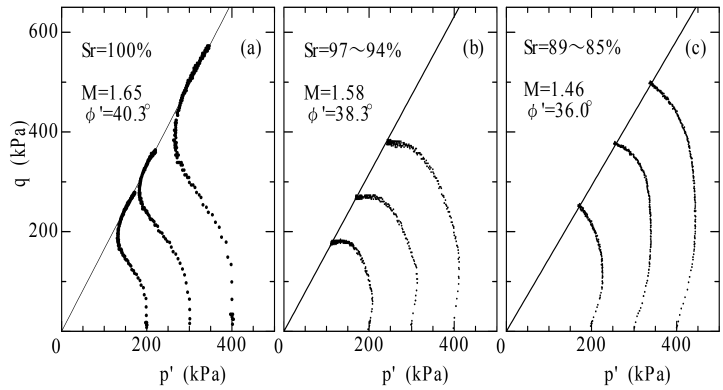

Fig. 1 Results of triaxial consolidated un-drained (CU) test

2. SAMPLE AND TEST METHOD

For the sample, Tachikawa loam collected in Hiratsuka City, Kanagawa Prefecture (natural water content: 100 to 110%) was used. A sample with an initial degree of saturation S_r of 80% 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 $\Delta\sigma$ 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 and drained shear rate was 0.004 mm/min, respectively.

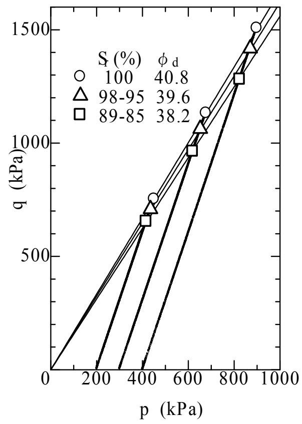

Fig. 2 Results of CD

3. RESULTS OF TRIAXTIL TESTS AND DISCUSSIONS

Fig. 1 shows effective stress paths in the tests using specimens with an initial degree of saturation of 100, 95 and 85%. The degree of saturation S_r shown in the fig.1 is the value before consolidation. The effective stress paths with a degree of saturation of 100% in Fig. 1(a) shows that dilatancy changed from negative to positive near the critical state line (CSL) and deviator stress increased subsequently along CSL. Fig. 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).

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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. 2 shows effective stress paths (total stress paths) obtained in triaxial consolidated drained test. As in consolidated undrained test, ϕ_d decreased with the decrease of S_r. At S_r = 100%, ϕ_d was in agreement with ϕ' in consolidated undrained test.

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 Fig. 3. 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.

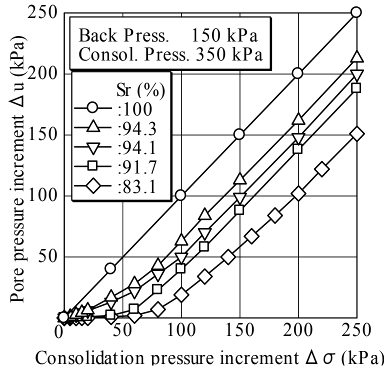

Fig. 3 Cell pressure increment and pore pressure increment

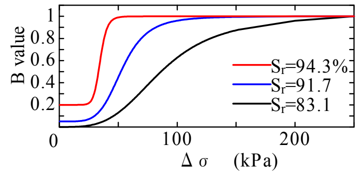

Fig. 4 Cell pressure increment and *B*

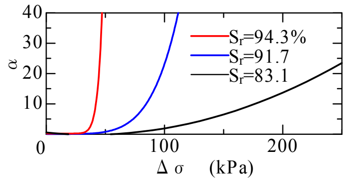

Fig. 5 Cell pressure increment and α

Fig. 4 shows the relationship between the $\Delta\sigma$ obtained from Fig. 3 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.

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

Fig. 5 shows the relationship between the $\Delta\sigma$ and coefficient α. In Fig. 5, *B* value in Fig. 4 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 Fig. 5 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.

5. CALCULATION TO REPRODUCE THE RESULTS OF TRIAXTIAL CU TEST

Calculations were made by incorporating the relationship shown in Fig. 5 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 Fig. 6 by solid lines. The modified Cam Clay model could reproduce no reverse of dilatancy observed in Fig. 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.

Table2 Constants for calculation

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

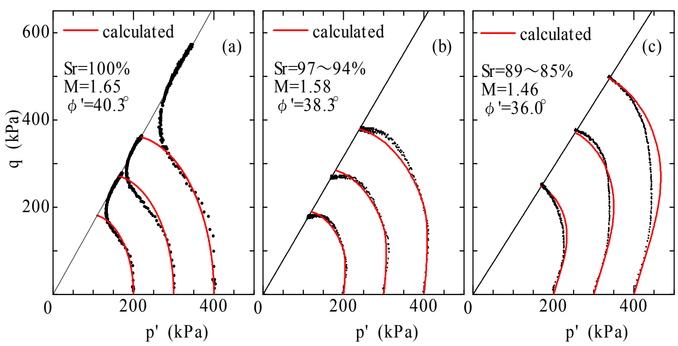

Fig. 6 Comparison of the results of calculations to reproduce triaxial

6. CLOSING REMARK

Unsaturated soil was assumed to be a two-phase system of pore fluid (water and air) with compressibility taken into consideration and soil particles. Calculations were made to reproduce the results of a triaxial consolidated undrained compression test. The results of calculations well reproduced test results up to the limit state. In the future, examination of stress-strain relationship will be required.

REFERENCES

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