Influence of Pore Air Pressure and Contractancy on the Shear Behavior of Extremely loose Volcanic Soils causing Slope Disasters

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ABSTRACT: Many slope disasters occur every year all over the world, and a lot of human lives are deprived by them. Sometimes, disasters at very gentle slopes and disasters that flow long distance, with destructive energy, are observed in Japan. In such cases, the trigger layer is often volcanic extremely loose soils. Although the cause of destructive long-distance flow has been explained by liquefaction, there are some disasters which occurred at unsaturated layer, i.e. liquefaction should not be a major cause. Focusing on the fact that slope disasters such as long-distance flow or gentle slope flow are triggered by volcanic soil layers with extremely loose structure, artificial loose soil samples consisting of silt and cement were prepared. Triaxial tests with unsaturated condition were performed to discuss how air trapped in voids affects the shear strength properties. It is indicated that extremely loose soil can be vulnerable against risk of landslides even if it is not fully saturated.

Keywords: volcanic soils, extremely high void ratio soil, unsaturated test, pumice,

1. Instruction

Every year, multitude of landslides occurs in Japan. In 2018, the landslides caused by pumice layer occurred in Hokkaido as shown in Fig. 1. and took away 41 lives. In many historic disaster cases, volcanic soils having extremely high void ratio are the trigger layers of disasters. These layers are sometimes too loose to reproduce in the laboratory.

Figure 1. Landslide at Horonai of Atsuma in Hokkaido Eastern Iburi earthquake (2018).

In this study, focusing on the fact that dangerous landslides, which flows long distance or occurs at gentle slopes or unsaturated condition, are triggered by extremely loose volcanic pumice layers, shear behavior of extremely loose soils was studied in this research.

In Japan, there are a lot of kinds of volcanic soils. Table 1 shows typical problematic volcanic soils in Japan. The maximum void ratio data of Shirasu and Ta-d pumice was taken from real sites by our lab. In general, these volcanic soil's high void ratio is due to porous particles and cementation connecting each particle with others. Although there are a lot of case studies about problematic volcanic soils in Japan, its shear behavior has not been understood systematically yet. This is because it is hard to collect a sufficient number of undisturbed volcanic soil samples repeatedly with an equivalent quality.

To do systematic research of problematic volcanic pumice soils, artificial pumice specimens which have cementation between particles and porous particles are reproduced in the laboratory. In our group saturated CD tests and CU tests were already done before[6]. By conducting saturated and unsaturated (partially saturated) CU tests, the behavior of extremely loose soil has been observed.

Table 1. Typical Japanese volcanic soils [1-5].

	Kanto loam	*Shirasu*	*Masado*	*Scoria*	*Ta-d pumice in Hokkaido*
Cementation	Yes	Yes	No	Yes	No
Porous Paricles	No	Yes	Yes	Yes	Yes
Max. e	3.38	2.16	1.11	1.23	7.37

2. Experimental procedure

The artificial specimens were reproduced to conduct systematic research because it is challenging to acquire a sufficient number of undisturbed samples with comparable physical properties from insitu ground. The artificial specimen consists of non-plastic clay (DL Clay) and Portland cement. The non-plastic clay was used considering the fact that volcanic trigger layers of many landslides have less plasticity.

In general, the loose structures of extremely high void ratio volcanic soils are maintained by cementation effect between particles or by porous particles or both of them. In this study, artificial specimen having both cementation and crushable particles (porous particles) were prepared to see fundamental shear characteristics of extremely high void ratio soil.

2.1. Specimen preparation

In this study, artificial samples (specimen A), with equivalent quality, were prepared as follows (Fig. 2.).

(1) DL clay, a non-plastic fine, was mixed with ordinary Portland cement at a specific mass ratio (DL Clay:Cement: Water = 85:15:25).

(2) Normal water was added and thoroughly mixed with the material for 10 minutes.

(3) The mixture was sieved by using 4.75 mm opening sieve until all the material has passed through the sieve.

(4) The mixture was poured loosely into a mold with 5 cm in diameter and 10 cm in height to achieve the designed void ratio (e=2.10).

(5) The mold was kept in a moist curing box for 7 consecutive days.

(6) Subsequently, the mold was kept in a drying oven for 24 hours.

(7) Experiments were started immediately after removing the specimens from the drying oven.

By following this procedure, a lot of specimens were produced, having almost equivalent cementation effect, porous particles, and density. The void ratio of the loosest DL Clay (non-plastic silt) obtained by “test method for minimum densities of sands (JIS A1224)” was lower than the artificial pumice. This indicates that artificial specimens reproduced the extremely loose natural volcanic soil whose looseness cannot be reproduced after disturbed. Although, the structure of the artificial pumice can be different from natural volcanic soils in a micro scale, in a macro scale, it can be said that this specimen could have cementation effect, crushable particles and extremely loose structure, which are the characteristics of natural soils. Figure 3 shows particle size distribution after cementation was broken by using hands and 4.75 mm opening sieve. The prepared samples were used to conduct experiments systematically.

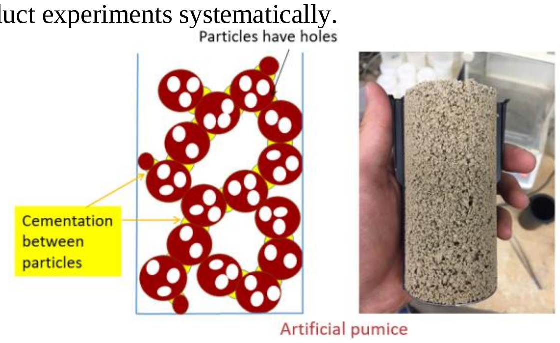


Figure 2. Artificial specimen having high void ratio.

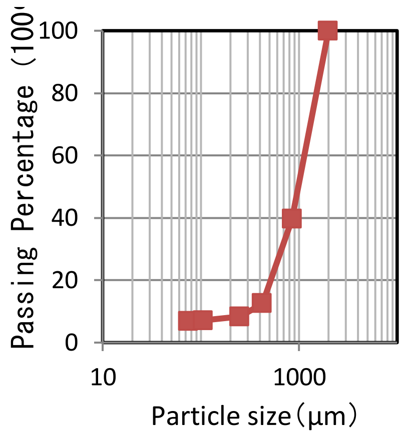


Figure 3.

2.2. Physical properties of the specimen

Table 2. shows particle density (ρ_s), dry density (ρ_d) and void ratio (e) of artificial specimen prepared in the laboratory. The loose DL clay specimen was also prepared to for comparison.

Table 2. Physical properties of the prepared specimens.

Specimen	ρ_s (g/cm³)	ρ_d (g/cm³)	*e*
Specimen A (Artifical pumice)	2.69	0.89	2.02
Loose DL clay	2.65	1.03	1.58

2.3. Test program

Table 3. Table 4. Experimental cases.

Test	Specimen	Type	Degree of saturation before consolidation, Sr(%)	Consolidation pressure(kPa)
A-CU50	Aritifical Pumice	CU	100	50
A-CU100				100
A-CU250				250
A-CU300				300
D-CU100	Loose Dl clay	CU	100	100
Ad-100	Aritifical Pumice	Partially saturated CU	0 (oven dried)	100
Ad-300				300
A60-CU100			60	100
A60-CU300				300
A70-CU100			70	100
A70-CU300				300

A series of the laboratory tests were carried out using the triaxial compression apparatus (Fig. 3). The shear characteristics of each artificial specimen were investigated by performing saturated CU tests and unsaturated (partially saturated) CU tests at several confining pressures. In the consolidation process of the saturated tests, a double vacuum method was applied to obtain the Skempton's pore water pressure parameter B[7] exceeding 0.95. The rate of confining pressure increment was set to 2.5 kPa/min in the consolidation process, and the rate of axial strain increment was set to 0.29 %/min in the shearing process. A summary of the conditions in all experiments are shown in Table 3.

2.4. Double Cell System for partially saturated test

Fig. 3 illustrates the traditional double cell system installed in the triaxial test apparatus used in this study[8]. To obtain volumetric strain and pore air & water pressure of partially saturated specimens, some new equipment was introduced. Pore air pressure transducer (No.3) was set to measure air pressure from a top cap (No.5). Pore water pressure transducer (No.7) was attached to Pedestal (No.6) to measure pore water pressure from the bottom of a specimen. *DPT(Differential pressure transducer)* was connected to inside water of Inner cell (No.8). Although porous stone was ordinarily installed to Double Cell System to get suction of unsaturated soil, filter paper was used instead of Porous stone in this study. This is because the specimen in this study had little suction due to relatively large particles and the lower part of the specimen was considered fully saturated (shown in Fig. 4.). Although this is not an element test in the strict sense of the word, it is the only way to keep reproduce exactly similar saturation degree for specimens having cementation effect.

Fig. 5b. schematically illustrates the measurement principle of the volume change of the specimen under undrained conditions. The total volume change of the specimen (ΔV) is obtained by equation (1)

$$\Delta V = -(\Delta V\_DPT - \Delta V\_VDT) \quad (1)$$

Where ΔV_DPT is the volume change of all substances beneath *WLi (Water level in the inner cell)*. A portion of ΔV_DPT is the intrusion volume of the top cap ΔV_VDT obtained by the measurement of the *VDT (Vertucak displacement transducer)*.

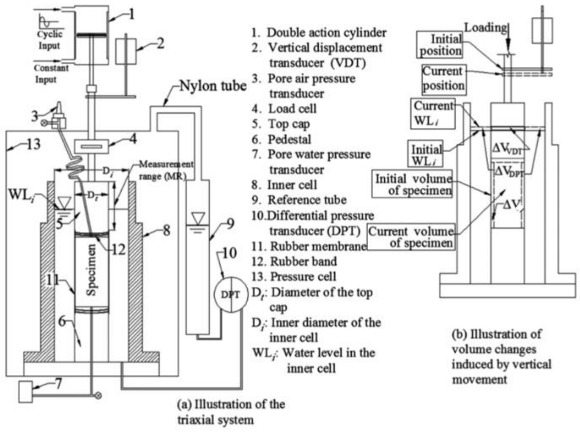


Figure 4. Layout of the traditional double cell system[8].

For the test of partially saturated specimen, a sufficient amount of water to saturate specific degree of saturation was calculated beforehand and supplied from the pedestal before the consolidation process. That is the reason why upper part of the specimen was dry in the case of *Sr*=60% (Fig. 4.). Thus, the tests conducted by the double cell system in this study was “partially saturated test” rather than “unsaturated test”. Supplied water amount was monitored by a differential pressure transducer.

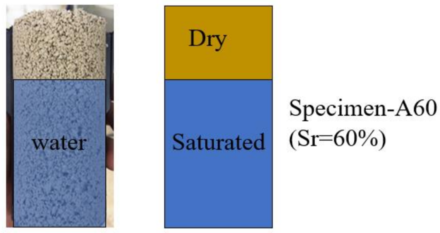


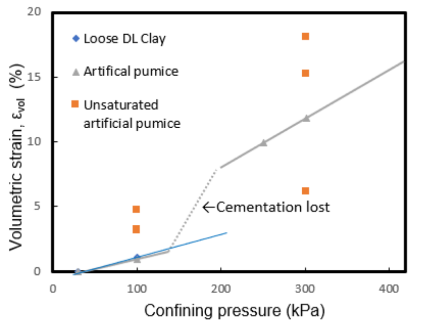
Figure 5. Illustration of partially saturated specimen.

Although ceramic stone is generally installed to double cell system to measure suction force, filter papers were used instead of it in this study. This is because it is considered that suction is low due to a relatively large particle size.

3. Results and discussion

3.1. Results of consolidation process

The volumetric strain of artificial specimens during the consolidation stage is illustrated in Fig. 5. In saturated cases, it shows low volumetric strain at low confining pressure levels, but its compressibility changes suddenly at around 170 kPa. This implies that cementation between particles was damaged at this pressure level. A greater compressibility was observed after the loss of cementation, compared with the loose DL clay. It is hypothesized that an extremely loose structure was ascribed to this high compressibility. In unsaturated cases, the results are scattered. Although the reason is unclear though, probably it is related with the strength difference between wet and dry specimens..



Test	ε_{vol} (%)
A-CU50	0.4
A-CU100	0.9
A-CU250	9.9
A-CU300	11.8
D-CU100	1.0
Ad-100	3.2
Ad-300	6.1
A60-CU100	3.2
A60-CU300	15.2
A70-CU100	4.7
A70-CU300	18.1

Figure 6. Volumetric strain during consolidation stage.

3.2. Saturated CU test results of cementation

Referring to Fig. 6 for saturated CU test results, in the case of low confining pressures of 50 kPa and 100 kPa, the specimen shows a peak strength at a low axial strain. In all the cases, the deviator stress reaches a residual (steady) state[9] when the axial strain reaches 10%. This behaivior is ascribed to its extremely loose structure. It can be hypothesized that once the cementation effect is lost in extremely loose soils, high strains can be developed drastically within a short time in the field. The strength of residual state and cementation depends on void ratio, cementation strength, particle crusbaility and so on. Therefore dangerousness of extremely loose soil layers is contingent on properties of volcanic features and earth pressure of real ground

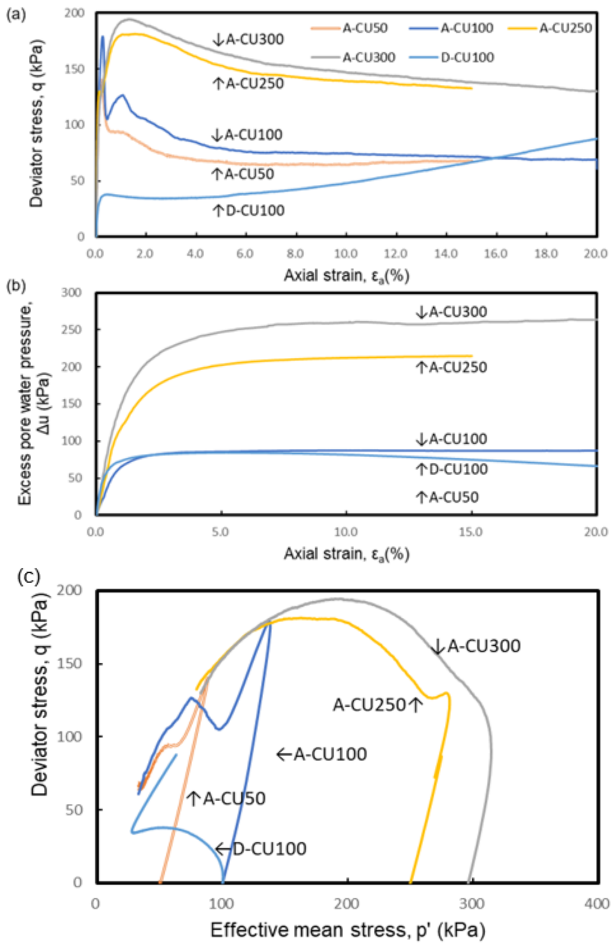


Figure 7. Relationship between (a) stress and strain (b) excess pore water pressure and strain (c) effective stress path in saturated CU tests.

3.3. Unsaturated (partially saturated) CU test results

Table 4. Saturation degree after consolidation process.

Test	Ad-100	Ad-300	A60-CU100	A60-CU300	A70-CU100	A70-CU300
Sr(%) before consolidation	0	0	60	60	70	70
Sr(%) after consolidation	0	0	63	76	75	93

Table 4 shows the degree of saturation after the consolidation process of unsaturated test cases. The saturation degree is increased after consolidation because some amount of air was drained from the top cap during consolidation.

CU test results of the partially saturated specimens are shown in Fig. 7 and Fig. 8. At confining pressures of 50 and 100 kPa, prior to the loss of cementation, a peak strength was observed at a small strain as same as saturated condition. In each confining pressure, the dry sample showed higher strength than saturated and partially saturated samples. Although this reason is unclear, possible causes include drying shrinkage effect. After cementation was lost, specimen showed loose behaivior. In all tests, significant negative dilatancy was observed from volume change. Although pore air pressure and pore water pressure was mesured separatedly, the air pressure was seemed to be same with the water pressure. The reason of this is considered that the suction force was low due to its relatively large particles. Both of pore water and air pressure were much higher when specimen is more than 70% saturated at confining pressure = 300kPa. It is thought that the reason of this is the fact that soil has less capacity to shrink when it has less pore air. The high pressure trapped air can be dangerous in the real site. In A70-CU300 case, the pore air pressure reached 200kPa. It means that air having one third volume of atmosphere air‘s volume can exist in the ground when slope disaster occurs in a short time. Overall, specimens of higher saturation degree shows looser behaviors because of cavoid air. It indicates that extremely loose soil can be vulnerable to landslides even if it is not fully saturated.

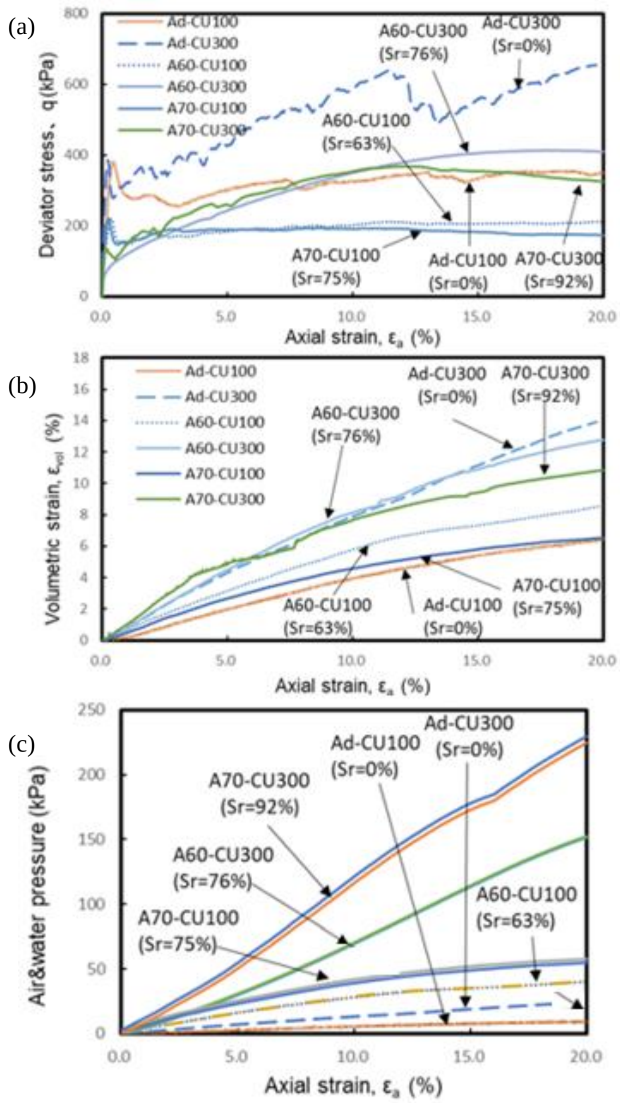


Figure 8. Relationship between (a) stress and strain (b) volumetric strain and strain (c) pore pressure and strain in unsaturated (partially saturated) CU tests.

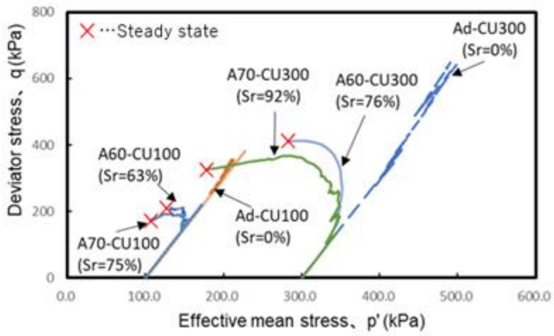


Figure 9. Effective stress path in unsaturated (partially saturated) CU tests.

4. Conclusions

Artificial extremely loose soil having porous particles and cementation effects were investigated in saturated and unsaturated CU tests, in order to understand characteristic features of extremely loose volcanic soils. Extremely loose volcanic soil kept by cementation exhibits a higher shear strength and lower compressibility when cementation is maintained. However, once cementation is lost, high compressibility and significant negative dilatancy were observed. This compressibility and negative dilatancy can be ascribed to the extremely loose structure. In saturated undrained compression tests, after the shear strength reached its peak and the cementation was lost, a brittle behavior was observed in which the shear strength converges to a residual state, called a steady state. This complex behaivior was caused by the extremely loose structure. In some cases, even if the specimen is not fully saturated after the shear strength reached its peak and the cementation was lost, a brittle behaivior was observed in which the shear strength converges to a residual state, called steady state. It implies that there is a possibility that high pressure trapped air could exist under the ground when the slope disaster occurred. These results indicate that extremely loose soil can be vulnerable to landslides even if it is not fully saturated.

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