Mudflow type Failure of Volcanic soil during the Earthquake

Toshiyasu UNNO1*
1Department of Civil Engineering and Regional Design, School of Regional Design,
Utsunomiya University, 7-1-2 Yoto, Utsunomiiya City, Tochigi 321-8585, Japan
unno@cc.utsunomiya-u.ac.jp

Abstract

Mudflow type failure of a gentle fill slope composed of volcanic sandy soil occurred during recent earthquakes. The failure detail has been reported by the authors and another researcher. Based on eyewitnesses' account and a reconnaissance report, the slope failed suddenly during the earthquake and the flowed soils contained much water. Those facts indicate that the interstitial pore water among soil particles plays an important role in the phase transformation from a static condition, as a solid phase, to a fluid-like phase after shaking. On the other hand, it is also reported that rain had not fallen for a week by the date of the earthquake and the ground surface was not fully saturated (i.e. unsaturated). Field saturation was about 50-90 %. Volcanic soils are known to have a high water retention capacity. It is considered that the porous nature of the soil is considered to be the reason for high water retention capacity. Therefore, it may be considered that the soil contained much water in its natural condition before the earthquake and the slope failure because of the peculiarly moist nature of volcanic sandy soil with pumice.

In previous research of one of the authors, the possibility of soil liquefaction triggering in unsaturated clean fine sand was discussed in relation to the developments of pore air and pore water pressures as well as volumetric strain, and it was concluded that even when the degree of saturation is small, the pore air and pore water pressures develop to become equal to the initial confining stress during cyclic loading, which then defined as the occurrence of soil liquefaction in unsaturated soils. Based on this concept, further experiments have been carried out to understand the liquefaction behavior of unsaturated sandy soil with the influence of non-plastic fine granular and water state taken into consideration.

In this manuscript, the contents of the results of liquefaction test of unsaturated soil are introduced. Here, the soil samples in this study used for the experiment is volcanic sandy soil.

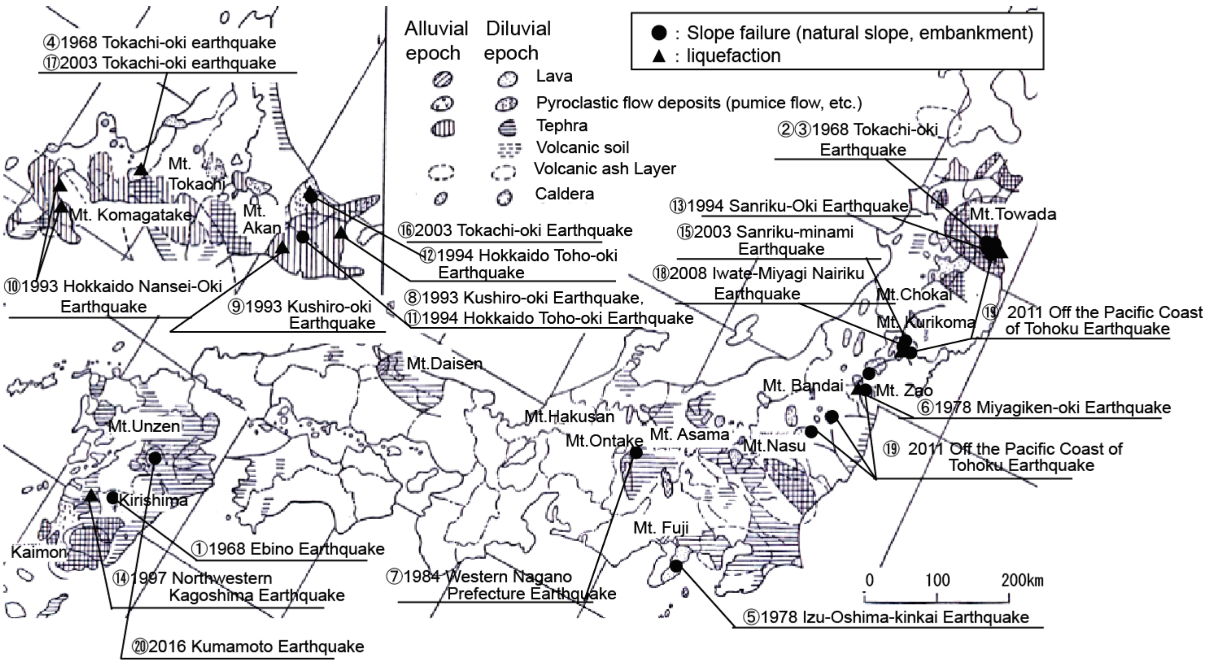
Keywords: Mudflow type failure, volcanic soil, liquefaction, cyclic shear, unsaturated soil.

1. INTRODUCTION

Based on recent earthquakes in Japan, mudflow type slope failures occurred in gentle slope fills and most of the failures were surface failure (Photo 1.), and the sediment of failure was probably in unsaturated condition (S. Nakamura et al., 2014). Recent studies have shown that a shaking disturbance affects the moisture state of soil, and that the liquefaction potential of unsaturated soil can be evaluated by taking the volume compressibility of the soil particle structure, the degree of saturation and the confining pressure into account (Kazama et al., 2006). Based on this concept, further experiments have been carried out to understand the liquefaction behavior of unsaturated sandy soil with the influence of non-plastic fines fraction and water state taken into consideration. In this paper, two different types of soil is used, such as mixture of Quartz powder with Silica sand, and Karasuyama soil (Photo 2). Karasuyama soil was taken from mudflow type slope failure in Tochigi Prefecture of Japan (Figure 1). The particle size of soil specimen in this test is the same size as the collapse soil of the mudflow type slope failures. In this study, the tests were conducted with varying pore water state and pore air in the soil specimen.

(courtesy of Miyagi Engineering Co.Ltd.)

Photo 1. Mudflow type slope failures of 2003 & 2008 Eartquake in JAPAN



Earthquake geotechnical disaster map in the distribution area of volcanic soil in Japan

2. PHYSUCAL PROPERTIES OF THE COLLAPSE SOIL

From a practical engineering point of view, when discussing the liquefaction of soils under the unsaturated condition, the target soil should be one with a high water retention capacity (Unno et al., 2006). In this study, soil differ in water retention capacity is used as the testing material in order to explain and compare the liquefaction state of unsaturated soils. Figure 2 shows the image of tested soils under microscope. According to Table 1, mixture of Quartz powder and Silica sand which is a non-plastic soil has 44.7 % of fine content and a specific gravity of 2.649 g/cm^3. The specific gravity of the Karasuyama soil is 2.682 g/cm^3 and 32.5 % of fine content. Figure 3 shows the grain size distributions of the soil used in this study. For comparison purposes, the curve of the Karasuyama soil is also shown in the figure.

Photo 3 Karasuyama soil

Figure 1. Karasuyama Slope failure in Tochigi Prefecture Japan (Tochigi prefectural government office (2014): Disaster report of the 2011 Tohoku Earthquake)

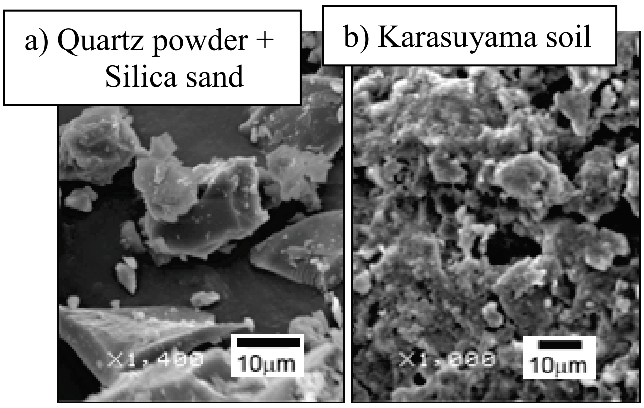


Figure 2. Tested soils specimens under microscope
a) Quartz powder + Silica sand mixture (x1400)
b) Karasuyama soil (x1000)

Table 1. Characteristics of tested soils.

Characteristics	Quartz powder + Silica sand (ratio 1:1)	Karasuyama soil
Specific gravity (g/cm^3)	2.649	2.682
Optimum water content (%)	9.7	23.2
Max. dry density (g/cm^3)	1.621	1.183
Liquid limit (%)	Non	Non
Plastic limit (%)	Non	29.3
Fine content Fc (%)	44.7	32.5

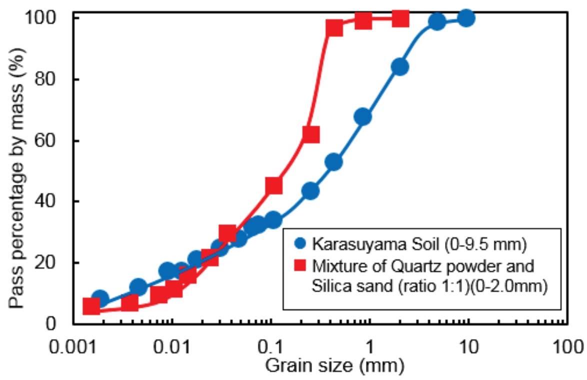


Figure 3. Grain size distribution of tested soils

The relationship between suction and the degree of saturation was studied using two different methods. Water retention test (JGS 0151-2009) and consolidated unsaturated samples which are the initial condition for triaxial tests. The relationship between the suction head and the degree of saturation following a drying path of the mixture of Quartz powder with Silica sand (ratio 1:1), and Karasuyama soil is shown in Figure 4. Suction (s=suction) during the tests was controlled using the axis translation technique. It is measured that the air entry value (AEV) of the mixture of Quartz powder and Silica sand is about s=17 kPa (Sr= 70 %) and that of Karasuyama soil is about s=17 kPa (Sr= 65 %).

3. TEST APPARATUS

Figure 5 and Photo 3 show the schematic diagram of testing apparatus for cylindrical specimens 50 mm in diameter and 100 mm in height. A glass fiber filter and ceramic disk with an AEV of 50 kPa installed at the top and bottom of specimen, respectively. A step motor was used for strain-controlled cyclic loading, and the volume change of the whole specimen was measured directly from the differential pressure meter between the inner cell and outer water head, and its measurement capacity is 15 % volumetric strain.

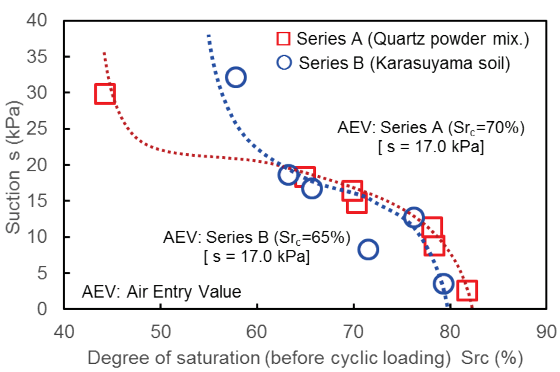


Figure 4. Relationship between suction and degree of saturation as initial condition for triaxial tests

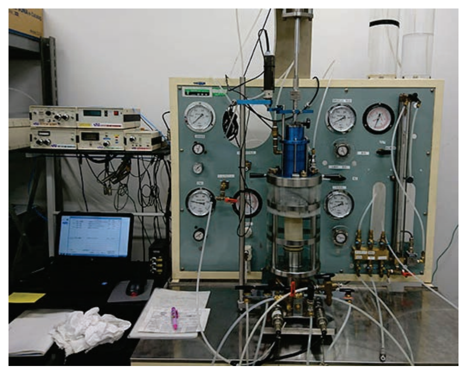


Photo 3. Cyclic unsaturated triaxial test system for liquefaction in Utsunomiya-university

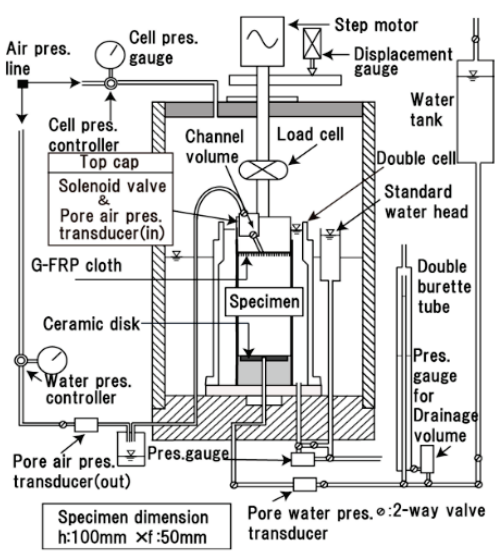


Figure 5. Configuration of equipment of the unsaturated triaxial test system

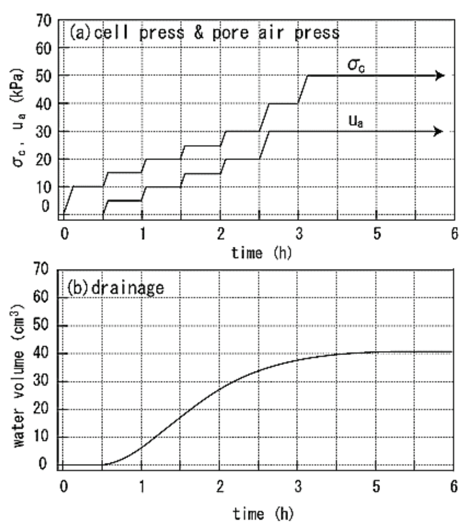


Figure 6. (a) Example of time history of cell pressure and pore pressure, (b) the time history of drained water volume during consolidation process

Continuous measurements of the pore water and air pressure are taken to obtain the soil suction. The pore air pressure during cyclic shear was measured by the air transducer attached directly above the specimen. Besides that, a solenoid valve is attached in order to avoid the effect of air compressibility in the pipeline.

The initial stress condition including the soil water condition was achieved in the following way. All specimens used wet tamping method. For mixture of Quartz powder with Silica sand , 70 cm3 of water was measured beforehand, corresponds to about 85 % saturation in the case of Dr0=85 %, and about 25 % water content when the Quartz powder is put in. Next, the soil is mix with the water thoroughly before being poured into the mold. During this process, the sand absorbs the water and the specimen becomes a uniform moisture state. The amount of the Quartz powder was decided from preliminary test in order to make initial relative density of 85 %.

For Karasuyama soil, 115 cm^3 of water was measured beforehand, corresponds to about 85 % saturation, and about 48 % water content when the Karasuyama soil is put in. Next, same as the mixture of Quartz powder and Silica sand, the soil in Karasuyama soil is mixed with the water thoroughly before being poured into the mold. During this process, the soil absorbs the water and the specimen becomes a uniform moisture state. The amount of the soil was decided from preliminary test in order to make the initial dry density nearly equal to 1.10 g/cm3. In the consolidation process of both series, the confining pressure is applied step by step. Because of the difficulty in controlling the air pressure to achieve the prescribed initial degree of saturation, we controlled the drained water volume from the initial condition (i.e. through the drying process) by controlling the air pressure to achieve suction at the prescribed value. That is, when the target degree of saturation was achieved during the consolidation process, as shown in Figure 6.

In Karasuyama soil, there are some cases that the excess pore water pressure build up when pore water valve is closed. This condition occurred when the pore water in specimens was not sufficiently stabilized. In the test series, two initial suction conditions was simulated considering the actual ground conditions, as shown in Figure 6.

Table 2. Initial condition after consolidation process (before cyclic loading process)

Sample	Case	Suction	Pore water pressure	Pore air pressure.	Dry density	Void ratio	Degree of saturation	Mean effective principal stress (kPa)	Cell pressure	Net stress	Vol. of air
	No.	s_c (kPa)	u_{wc} (kPa)	u_{ac} (kPa)	ρ_{dc} (g/cm^3)	e_c	Sr_c (%)	$\sigma'_{m0}=(\sigma_{m0}-u_{a0})+\chi s_c$	σ_m (kPa)	σnet_0 (kPa)	V_{ac} (cm^3)
Quartz powder and Silica sand (ratio 1:1) Dr_0 = 85 %	a-1	2.5	0.0(98.0)	2.5(100.5)	1.50	0.76	81.8	22.0	22.5(120.5)	20.0	15.1
	a-2	8.7	0.0(98.0)	8.7(106.7)	1.46	0.81	78.4	28.9	29.9(127.9)	21.2	19.7
	a-3	11.3	0.0(98.0)	11.3(109.3)	1.45	0.82	78.2	28.7	31.8(129.8)	20.5	19.7
	a-4	14.7	0.5(98.5)	14.7(112.7)	1.48	0.79	70.4	29.8	34.5(132.5)	19.8	26.5
	a-5	16.3	-0.6(97.4)	15.9(113.9)	1.48	0.79	69.9	31.2	35.1(133.1)	19.2	27.2
	a-6	18.2	-0.4(97.6)	17.9(115.9)	1.48	0.79	65.0	32.8	38.2(136.2)	20.0	30.3
	a-7	29.8	0.0(98.0)	29.9(127.9)	1.51	0.75	44.3	33.9	50.5(148.5)	20.5	47.2
	a-8	0.0	-	-	1.49	0.78	100.0	100.6	199.2(297.2)	100.3	-
Karasu-yama soil	b-1	3.4	0.88(98.9)	4.29(102.3)	1.13	1.37	79.3	21.0	22.6(120.6)	18.3	23.6
	b-2	8.2	-3.84(94.2)	4.33(102.3)	1.13	1.38	71.6	21.5	20.9(118.9)	16.5	34.4
	b-3	12.7	0.03(98.0)	12.69(110.7)	1.11	1.42	76.3	27.1	30.1(128.1)	17.5	28.6
	b-4	16.7	-3.33(94.7)	13.37(111.4)	1.12	1.40	65.8	27.7	30.1(128.1)	16.8	42.0
	b-5	18.5	-0.3(97.7)	18.22(116.2)	1.13	1.38	63.2	29.6	36.1(134.1)	17.9	44.0
	b-6	32.1	-2.36(95.6)	29.7(127.7)	1.18	1.27	57.9	35.8	46.9(144.9)	17.2	47.5
	b-7	0.0	-	-	0.99	1.69	100.0	100.3	199.5(297.5)	100.0	-

Table 2 indicates the condition of Quartz powder & Silica sand and Karasuyama soil, in which the initial pore water pressure is near or equal to the atmospheric pressure 98 kPa. The initial suction data for different test cases, s = 2, 8, 10, 15, 18, 30 kPa. The full saturation specimen's test cases (a-8, b-7) had the consolidation process like the ordinary cyclic shear test method that is, the same as the liquefaction test using the ceramic disk of unsaturated triaxial test system but without using the glass fiber filter. In order to achieve full saturation condition of Sr=100%, the back pressure was given to the specimen by 100 kPa.After simulating the initial isotropic stress condition described above, cyclic axial strain is applied. This test were under step loading — each axial strain single amplitudes of the sinusoidal wave was 0.2, 0.4, 0.8, 1.6, and 2.0 with every ten cycles. The concept of strain controlled cyclic shear test is shown in the literature by one of the author. The frequency of the sinusoidal wave is 0.005 Hz. This loading rate is slow enough to achieve an equilibrium condition between air and water pressure.

In order to study the cyclic behavior of unsaturated soil from various initial suction conditions, several series of strain-controlled cyclic triaxial tests were conducted under the undrained condition. For our purposes, an undrained condition is one in which the migration of pore water and air to the outside of specimen is not allowed during cyclic shear. In other words, this condition assumes that the actual seismic loading rate is more rapid than the migration rate of pore water and air, as is the case under saturated conditions. Because the permeability of pore air is generally smaller than that of water under the nearly full saturation condition, this assumption is considered reasonable.

Table 2 depicts the initial conditions of the triaxial tests after the consolidation process (after the pore water valve and solenoid valve were closed, i.e. just before the cyclic shear process). The initial soil conditions described in the Table1 were the testing parameters: the initial degree of saturation (0-100 %), the net stress (σnet_0 = 20 kPa in target value) and the suction (0-30 kPa). Here, the test conditions of Quartz powder & Silica sand (a-1 ~ a-7), and Karasuyama soil (b-1~ b-6) are indicated.

・Quartz powder & Silica sand specimen: Mixture of Quartz powder and Silica sand (ratio 1:1) with the initial relative density about 85 % and the initial pore water pressure is equal to the atmospheric pressure.

・Karasuyama soil specimen: Karasuyama soil with the initial relative density about 80% of the maximum dry density. The initial pore air pressure is between 2.0kPa ~ -1.7kPa from the actual atmospheric pressure.

The reason for taking two kinds of initial suction states into account is as follows. According to the Japanese Geotechnical Society Standard (JGS-standard 0527-1998), an initial suction state can be produced to introduce certain pore air pressure, which is usually larger than an atmospheric pressure, to an unsaturated specimen in triaxial shear test. In such a condition, the pore air pressure may be different from the atmospheric pressure. On the other hand, when the suction is at or near the ground surface, capillary tension is demonstrated, the pore air pressure is thought to be identical to the atmospheric pressure and the pore water pressure is smaller than the atmospheric pressure. The conditions of Karasuyama soil correspond to this situation.

It is well known that there are many definitions of effective stress for unsaturated soils. In this study, we have used Bishop's proposed equation (Bishop et al., 1963) to evaluate the effective stress, as shown in Equation [1].

$$\sigma'_m = (\sigma_m - u_a) + \chi\,(u_a - u_w) \qquad [1]$$

$$s = u_a - u_w \qquad [2]$$

In the equation, u_a, u_w and χ represent the pore air pressure, the pore water pressure, and the material parameter, respectively. σ'_m and σ_m represent the mean effective principal stress and the effective stress. The suction is defined by Equation [2]. Several definitions of parameter have been proposed by many researchers (e.g. Bishop et al., 1963; Vanapalli et al., 1996; Gallipoli et al., 2002). In this study, parameter adopts the degree of saturation. To discuss the effectiveness of the parameter used here during the cyclic shear process, the results using another definition of parameter in the following section. By using the above definition, the effective stress reduction ratio during the cyclic shear can be defined as follows:

$$\text{Effective stress reduction ratio} = 1 - \frac{\sigma'_{m}}{\sigma'_{mo}} \qquad [3]$$

This index indicates the degree of effective stress loss ranging from zero to unity, when the effective stress reduction ratio is 0.95~1.00, the specimen becomes complete liquefaction state due to cyclic shear. In this paper, capital letter ESRR is used as the abbreviation of Effective Stress Reduction Ratio.

4. TEST RESULTS

Stress-strain Relationships, Effective Stress Paths and Suction Behaviors

The final state after cyclic shear for both Quartz powder & Silica sandre summarized in Table 3. Figure 8 and 9 illustrate stress and strain relationship of selected specimen in Quartz powder & Silica sand (a-1) and Karasuyama soil (b-1) during cyclic loading. Figure 10 and 11, show the effective stress path of selected specimen in Quartz powder & Silica sand (a-1) and Karasuyama soil (b-1) during cyclic loading.

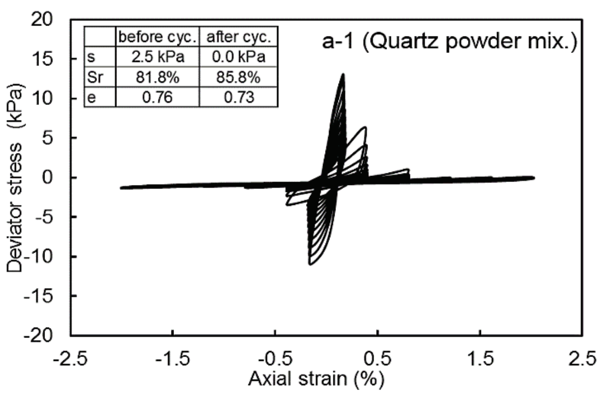


Figure 8. Stress and strain relationship during cyclic loading (Quartz powder & Silica sand, a-1)

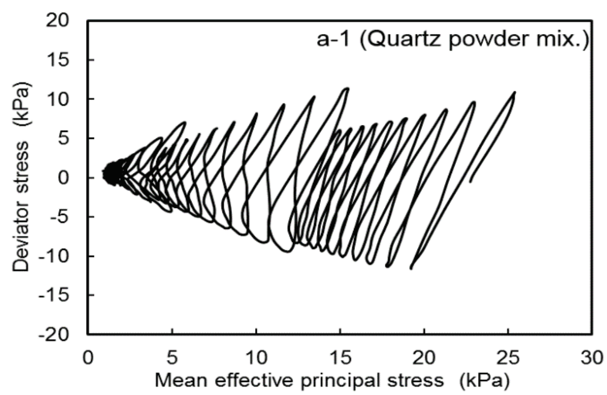


Figure 10. Effective stress path during cyclic loading (Quartz powder & Silica sand, a-1)

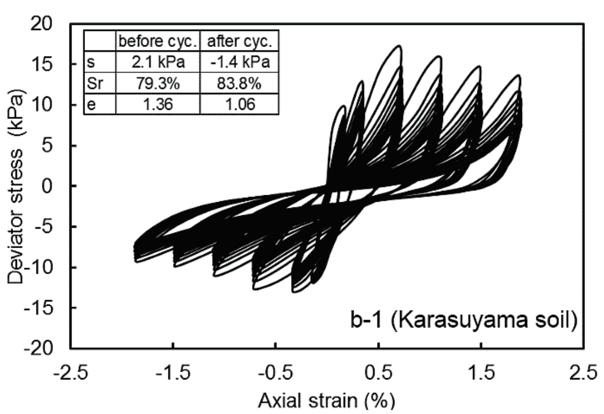


Figure 9. Stress and strain relationship during cyclic loading (Karasuyama soil, b-1)

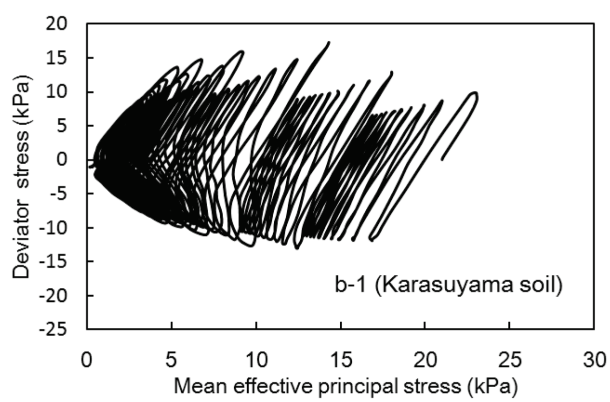


Figure 11. Effective stress path during cyclic loading (Karasuyama soil, b-1)

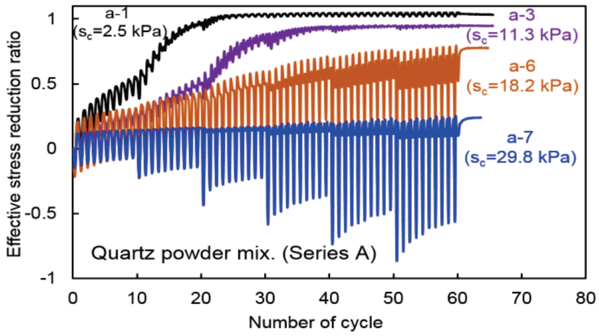


Figure 12. Effective stress reduction ratio during cyclic loading

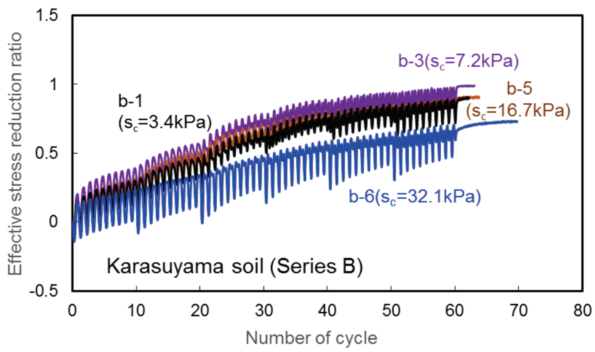


Figure 13. Effective stress reduction ratio during cyclic loading

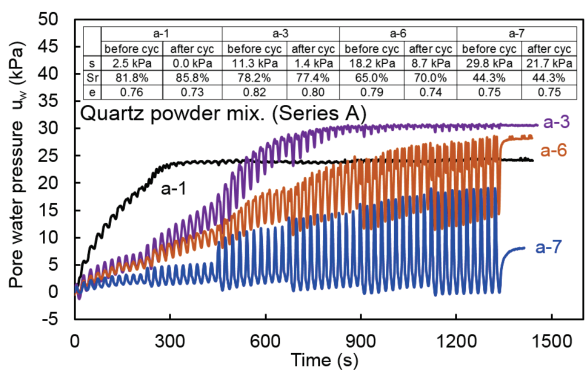


Figure 14. Time history of pore water pressure during cyclic loading

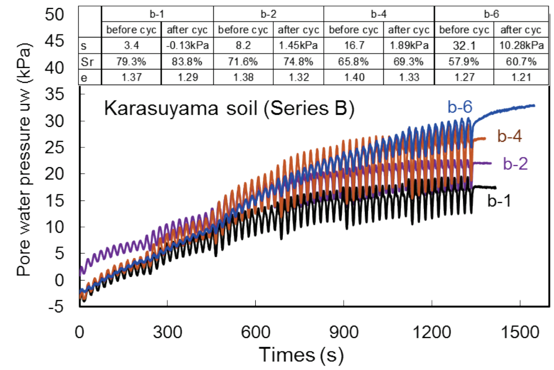


Figure 15. Time history of pore water pressure during cyclic loading

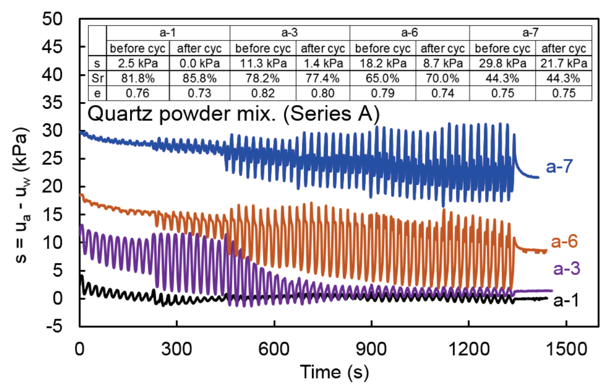


Figure 16. Time history of suction during cyclic loading

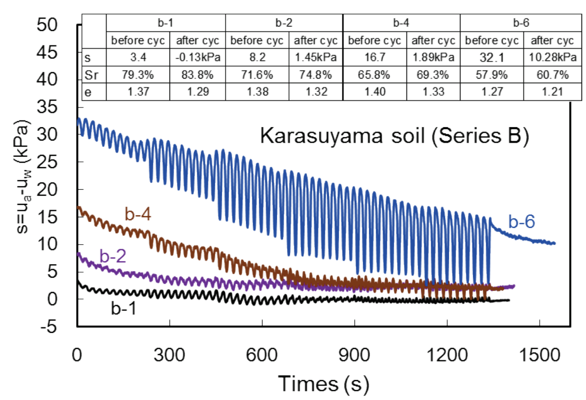


Figure 17. Time history of suction during cyclic loading

Figure 12 and 13 show the effective stress reduction ratio of selected specimen in Quartz powder & Silica sand (a-1, a-3, a-6, a-7) and Karasuyama soil (b-1, b-2, b-4, b-6). Figure 14 and 15 show the time history of pore water during cyclic loading. The difference between u_a and u_w which represents suction during cyclic loading in both Quartz powder & Silica sandre indicated in Figure 16 and 17. Based on the graph, a gradual decrease in the peak of the deviator stress with increasing strain amplitude, the stiffness decreased with each loading cycle. Consequently, the mean effective principal stress reached zero. It is noteworthy that, even in cases with a considerably small degree of saturation, the soil particle skeleton is degraded by the cyclic shear and reaches a zero effective stress state, thereby causing a failure in the microstructure and engendering the reduction of the soil shear strength. In such state, the specimen behaves like a liquid.In Figure 14 (Quartz powder & Silica sand, a-1), the pore water pressure gradually increased and reached the initial mean confining stress at around 300 seconds. At this point, the net stress reaches zero. On the other hand, Figure 15 (Karasuyama soil, b-1), pore water pressure gradually decreased and reached zero at around 600 seconds. At this point, the effective stress reaches zero. As shown in the results of the two test cases, the reduction processes of the two tests were different, and the contribution of the net stress and suction to effective stress were completely diminished in both cases.

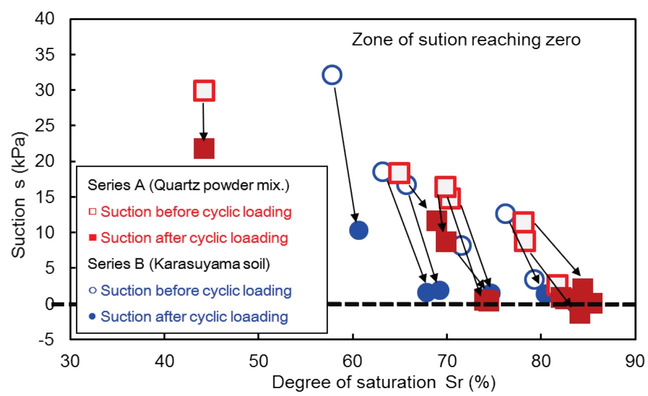


Figure 18. Changes of suction and the degree of saturation between before and after cyclic shear

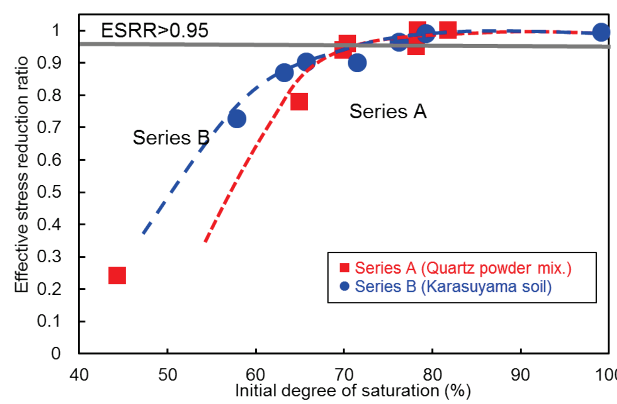


Figure 19. Relationship between effective stress reduction ratio and degree of saturation as initial condition for triaxial tests

Figure 18 shows the relationship between the suction and the degree of saturation before and after cyclic shear. Approximate zone of degree of saturation which is estimated to become liquefaction with cyclic shear, based on the change of volume, is shown as “zone of the suction reaching zero” in the Figure 18. If we look at the detailed suction behavior shown in Figures 16 and 17, firstly, the suction decreased during cyclic shear in all test cases and the suction of some specimens reached zero. It is likely that suction decreases with almost direct proportionality to effective stress loss. However, zero suction does not imply zero effective stress. As suction decreases according to Equation [1], the mean effective principal stress decreases.

The effective stress of unsaturated soil consists of both effect of soil particle skeleton of the first term and that of the suction of the second term in Equation [1].

Figure 12 and 13 shows the time histories of the effective stress reduction ratio for several specimens during cyclic loading. As shown in the figures, when the same axial strain history is applied, it is more difficult to bring the effective stress reduction ratio to unity when the degree of saturation is low and the relative density is high. Consequently, the final effective stress after cyclic shear in all cases can be summarized as a function of the initial degree of saturation, as is shown in Figure 19. The liquefaction of unsaturated soils is affected not only by the volume compressibility of the soil particle structure, which is reflected by dry density, but also by the degree of saturation and the initial confining pressure.

5. CONCLUSION

he conclusions can be summarized as follows:

a. The air entry value (AEV) of the mixture of Quartz powder and Silica sand is about s= 17 kPa (Sr= 70 %), and that of Karasuyama soil is also about s= 17 kPa (Sr= 65 %).

b. In the initial suction lower than AEV of soil-water characteristics curve, the effective stress of the specimens became zero or nearly zero, and the stiffness decreased.

c. It is considered that the variation of air distribution patterns in specimens under the AEV and over the AEV cause the apparent difference in cyclic shear behavior.

d. Our test results reveal that, even in the case where the degree of saturation is quite small, the soil particle skeleton is degraded by cyclic shear and reaches a zero effective stress state, thereby causing a failure of the microstructure and engendering the reduction of the soil shear strength. In such a state, the specimen behaves like a liquid, in the same manner as a saturated specimen. In other words, even if the soil is unsaturated, the soil reaches a zero effective stress state under certain conditions.

e. Based on the test results, it can be concluded that, the difference in water retention capacity and the existence of non-plastic fines fraction caused the soil test samples to reached liquefaction within lower degree of saturation.

ACKNOWLEDGEMENT

This work was supported by JSPS KAKENHI Grant Numbers 15K20900, 15H02263.

The Tochigi Prefecture Government supported the soil sampling of Karasuyama soil used in this study. The author express their sincere appreciation to the organization.

The alumni in Geotechnical Laboratory, Utsunomiya University were very helpful for the site investigation, laboratory tests. Mr. Taro Kumamoto and Ms. Afifah Yasmin Binti Zakaria did the unsaturated triaxial test.

REFERENCES

- Bishop, A.W. and Blight, G.E. (1963): Some aspects of effective stress in saturated and partly saturated soils. Geotechnique, 13(3), 177-197.
- Gallipoli, D., Gens, A., Vanet, J. and Romero, E. (2002): Role of degree of saturation on the normally consolidated behavior of unsaturated soils, Proc. of 3rd Int. Conf. on Unsaturated Soils, 115-120.
- Grozic, J.L., Robertson, P. K. and Morgenstern, N. R. (1999): Cyclic liquefaction of loose gassy sand, Can. Geotech. J., 37, 843-856.
- Kazama, M., Takamura, H., Unno, T., Sento, N. and Uzuoka, R. (2006): Liquefaction mechanism of unsaturated volcanic sandy soils, JSCE, Journal of Geotechnical Engineering, 62(2), 546-561 (in Japanese).
- S. Nakamura, A. Wakai, J. Umemura, H. Sugimoto and T. Takeshi (2014): Earthquake-induced landslides: distribution, motion and mechanisms, The Japanese Geotechnical Society, Special Issue on Geotechnical Aspects of the 2011 off the Pacific Coast of Tohoku Earthquake, Soils and Foundations, 54(4), 544-559.
- Tochigi prefectural government office (2014): Dosha Saigai Fukkyu Jigyou No Kiroku, Disaster report of the 2011 off the Pacific coast of Tohoku Earthquake, p.75 (in Japanese).
- Unno, T., Kazama, M., Sento, N. and Uzuoka, R. (2006): Cyclic shear behavior of unsaturated volcanic sandy soil under various suction conditions, Proc. of 3rd Int. Conf. on Unsaturated Soils, 1133-1144.
- Vanapalli, S.K., Fredlund, D. G., Pufahl, D. E. and Clifton, A. W. (1996): Model for the prediction of shear strength with respect to soil suction, Canadian Geotechnical Journal, Vol. 33, No. 3, pp. 379-392.