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DEFORMATION-STRENGTH EVALUATION OF CRUSHABLE VOLCANIC SOILS BY LABORATORY AND IN-SITU TESTING

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ABSTRACT

This paper aims to clarify static and cyclic deformation-strength properties and evaluation method for volcanic coarse-grained soils with particle crushing, based on site investigations which were conducted using standard penetration test (SPT), cone penetration test (CPT) and seismic cone penetration test (SCP). In addition to these in-situ tests, a series of laboratory tests on disturbed and undisturbed samples was also carried out to obtain the mechanical properties such as shear strength, cyclic undrained triaxial strength, pseudo elastic shear modulus and damping ratio. From the results of drained and undrained triaxial compression tests, plane strain compression test and cyclic undrained triaxial test on volcanic soils, it was understood that the effect of particle crushing and fabric anisotropy on the cyclic and static strength-deformation behavior of volcanic coarse-grained soils could not be ignored. Furthermore, the effect of particle breakage on results of dynamic penetration test was also discussed based on the correlation between the sounding data and triaxial test results.

Key words: anisotropy, in-situ test, laboratory test, liquefaction, particle breakage, shear modulus, stress-strain relation, volcanic coarse-grained soil (**IGC**: D6/D7)

INTRODUCTION

Volcanic activities from the Neogene to the Quaternary are the origin of many kinds of volcanic soil deposited widely over the area of Japan. Sedimentary structure, components, distributional area and degrees of weathering greatly differ with the depositional environment. Therefore, it is anticipated that the mechanical property of volcanic soil grounds will be divergent. Such volcanic soils have been used as a useful construction material. However, research on volcanic coarse-grained soils from an engineering standpoint is still extremely superficial in comparison with cohesionless soils.

Recent big earthquakes around Hokkaido generated the most serious damage in the grounds composed of volcanic soils with particle crushing (Miura et al., 1995 and 1996). Especially, liquefaction-induced damages which covered roads and housings have motivated research to understand the mechanical behavior of the volcanic soil grounds during earthquake more accurately.

Some researches on particle crushing under various levels of confining stresses have been performed. Hyodo et al. (1996, 1998 and 2002) and Miura et al. (1996 and 1997) have revealed a peculiarity of the stress-strain characteristics and cyclic strength of crushable soils and volcanic soils. It was clarified that not only the initial density but also the initial effective confining pressure was an important factor controlling the mechanical behavior. Bopp and Lade (1997) and Yamamuro and Lade (1997) have carried out undrained monotonic triaxial tests on hard grained soils under high confining stress to investigate the correlation between soil stability and particle crushing. Coop and Airey (2002) and Luzzani and Coop (2002) also discussed particle crushing due to isotropic compression and shearing for a reconstituted carbonate sand and its relation to the critical state. However, little is known about the mechanical behavior obtained from in-situ tests on crushable grounds.

The first purpose in this paper is to reveal the monotonic and cyclic mechanical behavior both for reconstituted and undisturbed volcanic soils by laboratory tests. Secondly, the effects of particle crushability and fabric anisotropy on mechanical behavior inherent to volcanic coarse-grained soils in Quaternary volcanic deposits in Kyushu and Hokkaido are discussed based on the results of laboratory and in-situ tests.

SAMPLING SITES AND TEST MATERIALS

Ten kinds of volcanic coarse-grained soils were used, which spouted out from four volcanoes in Hokkaido and Kyushu. Each sampling site is shown in Fig. 1.

Pyroclastic fall deposit (K_0-d), which originated from the eruption of Mt. Komagatake (1640), was sampled

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from Mori town in Hokkaido. This sample is hereafter referred to as Mori volcanic soil. The ejecta of Shikotsu caldera was sampled from 5 sites in Hokkaido. These sites are: Tomikawa district in Monbetsu town (Tomikawa volcanic soil); Kashiwabara and Utonai districts in Tomakomai City (Kashiwabara volcanic soil and Utonai volcanic soil); Bibi district in Chitose City (Bibi volcanic soil); and Hayakita town (Hayakita volcanic soil). It is estimated that the eruption age for these volcanic soils belonging to Shikotsu primary tephra (*Spfa-1*) is 31,000 ~ 34,000 years ago. In the eastern region of Hokkaido, pyroclastic fall deposit (*Ma-l*), which belongs to the Mashu volcanic product, was taken from the Musa district (Nakashibetsu Musa volcanic soil), Touhoro district (Touhoro volcanic soil) and the airport area in Nakashibetsu town (Nakashibetsu A volcanic soil). These volcanic soils were formed 11,000 ~ 13,000 years ago.

Shirasu sampled in Kagoshima City seems to be primarily the secondary sedimentation of pyroclastic flow deposit (*A-Ito*) which erupted from the Aira caldera about 24,000 years ago.

Figures 2(a) to (d) show the soil profiles and in-situ test results at the sites of Utonai, Hayakita, Nakashibetsu airport and Kagoshima. The black marks (■) in the figures depict the depth of thin-walled or triple tube sampling. SPT and SCP denote the standard penetration test and the seismic cone penetration test, respectively.

Undisturbed sampling using the block sampling method was carried out at the areas of Mori, Kashiwabara and Touhoro. On the other hand, undisturbed samples of Utonai, Hayakita and Kagoshima sites were taken by the triple-tube sampling. Moreover, thin-walled sampling was also performed at Nakashibetsu airport.

Physical properties of reconstituted and undisturbed samples are shown in Tables 1 and 2, respectively, compared to that of Toyoura standard sand. Fines content (less than 75 μm) of all samples is less than 20%. The mean grain size D_{50} of Nakashibetsu, Touhoro and Nakashibetsu A volcanic soils is significantly large (D_{50} = 3.9 ~ 6.6 mm) in comparison with other volcanic soils. Except for Mori volcanic soil, a low value of dry density ρ_d is shown in each volcanic soil because constituent particles are very porous.

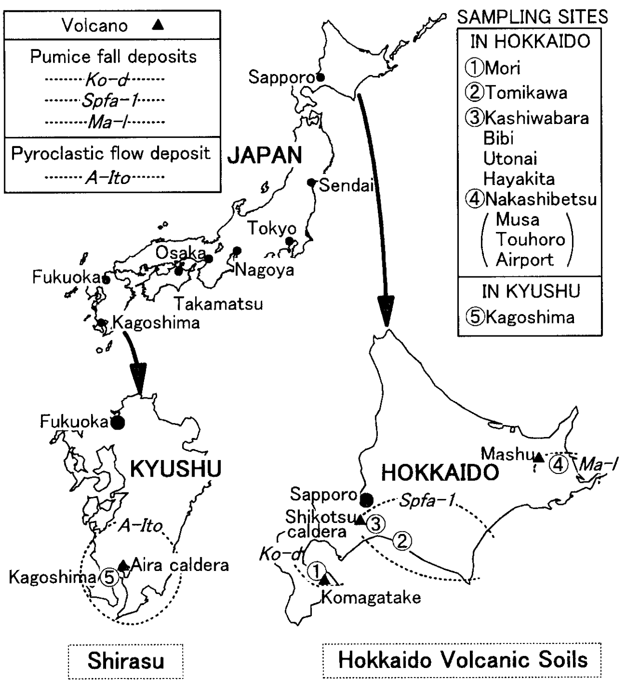

Fig. 1. Location of sampling sites

TESTING PROCEDURE

The procedure of laboratory and in-situ tests complied with the standards of JGS (2000). All reconstituted specimens were produced using the multiple sieving pluviation apparatus (MSP method) by which various desired specimen density can be achieved exclusively by controlling the rate of material discharge (Miura and Toki, 1982). In laboratory testing, the density of specimen ρ_{dc} after consolidation was aimed to be the in-situ dry density $\rho_{d\ in\text{-}situ}$, which is shown in Table 2. The variations in ρ_{dc} from the

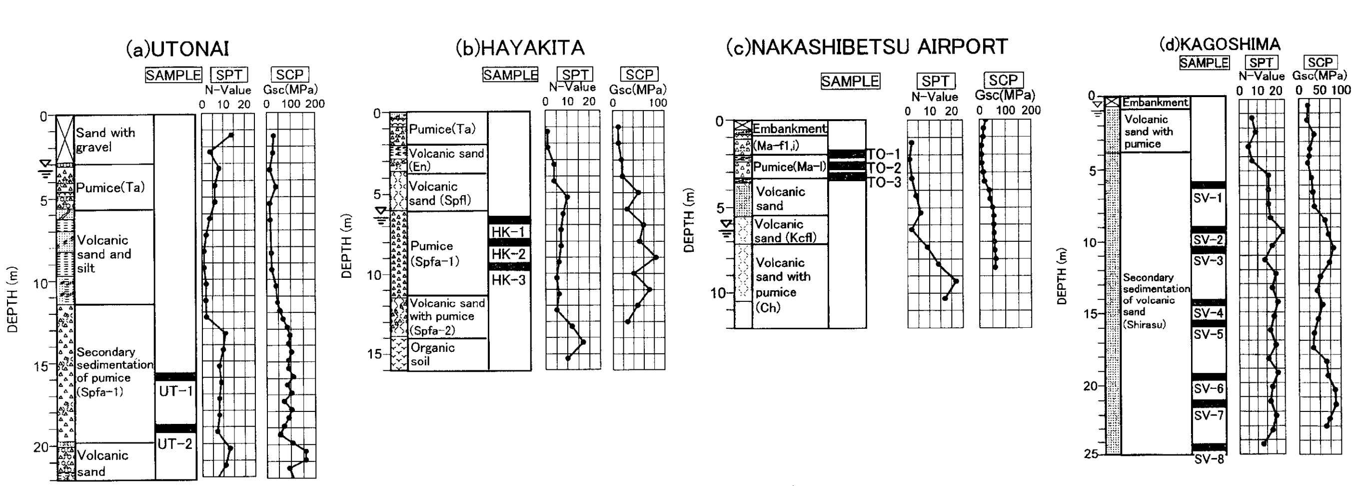

Fig. 2. Soil profiles and in-situ test results: (a) Utonai, (b) Hayakita, (c) Nakashibetsu airport and (d) Kagoshima

desired one are limited within ±5%. The variations in relative density after consolidation D_{rc} (=80%) for Toyoura sand specimens are limited within ±3%.

Undisturbed specimens were taken using triple tube, thin-walled tube and block sampler as noted above. After sampling, undisturbed specimens were frozen in the insulated boxes, and transported to a freezer (about −25°C) in the laboratory. Thereafter, a frozen specimen for triaxial shear testing was trimmed to 70 mm in diameter, 170 mm in height. It is desirable for the diameter of a triaxial specimen to be more than 20 times that of the maximum grain size (JGS, 2000). For non-uniform soils, however, JGS (2000) permits the specimen with a diameter above 5 times the maximum grain size. In this study, therefore, discussions are limited to the volcanic soil specimens where the maximum grain size is smaller than 15 mm.

The following two kinds of undisturbed test specimens, which are specified according to the cutting directions, were prepared from the frozen blocks of Mori and Kashiwabara volcanic soils (Miura and Toki, 1984).

(1) BV-specimen: This was cut from frozen block so that the z-direction of specimen coincided with the in-situ vertical direction.

(2) BH-specimen: The specimen was prepared by cutting in the direction which differed from that of BV-specimen by 90°. Accordingly, it follows that the z-direction of this undisturbed specimen was coincidental with the horizontal direction in natural deposits.

After each specimen was set up in the cell, the frozen specimen was allowed to melt under an effective confining pressure 19.6 kPa for 2 hours. Undisturbed specimens were saturated using the methods proposed by Rad and Clough (1984) and JGS (2000). In the case of reconstituted specimens, carbon dioxide was percolated through the specimen according to JGS (2000). Subsequently, deaired water was permeated into the voids at a small differential head (4.9 kPa). A back pressure of 196 kPa was, thereafter, applied to ensure the saturation of the specimen. By this procedure, Bishop's B value of all specimen was equal to or larger than 0.96.

After isotropic consolidating for 2 to 24 hours under confining pressure $\sigma_c'=49\sim392$ kPa, a series of triaxial compression and plane strain tests were carried out under the drained and undrained conditions with an axial strain rate of 0.20% per min. The principal stress and principal strain used are shown in Fig. 3. Effective mean principal stress p' in the triaxial compression and plane strain tests can be expressed as $(\sigma_a'+2\sigma_r')/3$ and $(\sigma_x'+\sigma_y'+\sigma_z')/3$, respectively. Where σ_a' and σ_z' is σ_1', σ_r' and σ_x' is σ_3' and σ_y' is σ_2. In the test, the axial strain was measured by an electrical dial gauge (displacement transducer). The radial strain was calculated through the measurements of axial and volumetric strains.

Table 1. Physical properties for reconstituted samples

Sample name	ρ_s (g/cm^3)	$\rho_{d\ in\text{-}situ}$ (g/cm^3)	D_{50} (mm)	U_c	F_c (%)
Mori	2.82	1.49	0.64	2.3	0.2
Tomikawa	2.22	0.49	1.1	2.8	1.0
Kashiwabara	2.34	0.53	1.3	3.1	1.3
Bibi	2.28	0.65	1.4	4.0	1.4
Nakashibetsu (Musa)	2.51	0.41	4.6	5.1	1.6
Touhoro	2.56	0.45	6.6	6.0	2.1
Toyoura sand	2.68	—	0.18	1.5	0

Table 2. Physical properties for in-situ volcanic soils

Sample name		Depth (m)	σ_v' (kPa)	ρ_s (g/cm^3)	ρ_d (g/cm^3)	e_0	ω_0 (%)	D_{50} (mm)	U_c	F_c (%)
Utonai* (Triple tube)	UT-1	15.8	98.0	2.37	0.51	3.63	110.7	3.00	5.88	2.63
	UT-2	19.0	98.0	2.46	0.66	2.71	95.0	4.50	7.12	2.23
Hayakita* (Triple tube)	HK-1	6.5	78.4	2.32	0.56	3.14	76.7	1.36	4.52	0.81
	HK-2	8.0	88.2	2.27	0.56	3.04	96.5	2.40	5.65	2.95
	HK-3	9.5	98.0	2.18	0.49	3.45	132.6	3.30	5.68	1.03
Nakashibetsu A* (Thin wall)	TO-1	1.8	29.4	2.42	0.41	4.90	184.9	4.20	6.82	2.66
	TO-2	2.5	29.4	2.61	0.43	6.07	157.5	4.76	28.18	5.17
	TO-3	3.2	39.2	2.42	0.38	5.37	136.0	3.90	6.00	1.67
Touhoro* (Block)	TO-4	1.8	29.4	2.49	0.40	5.23	140.9	6.35	36.36	4.38
	TO-5	—	98.0	2.54	0.51	4.03	114.0	5.25	41.18	5.32
Shirasu** (Triple tube)	SV-1	6.4	49.0	2.57	1.27	1.03	27.4	0.31	2.69	3.06
	SV-2	9.1	68.6	2.55	1.24	1.05	35.1	0.27	2.34	3.59
	SV-3	10.6	78.4	2.57	1.06	1.44	49.5	0.22	2.68	5.87
	SV-4	14.1	98.0	2.40	1.09	1.20	43.7	0.22	—	10.23
	SV-5	15.8	107.8	2.44	1.10	1.22	45.9	0.17	—	19.13
	SV-6	19.5	127.4	2.45	0.99	1.47	54.9	0.23	—	12.59
	SV-7	21.2	137.2	2.46	0.92	1.68	60.1	0.32	—	10.60
	SV-8	24.5	156.8	2.40	0.97	1.48	57.2	0.21	—	10.16

*For pumice, **For volcanic sand
σ_v': Effective overburden pressure, ρ_s: Specific gravity, ρ_d: Dry density, e_0: Void ratio, ω_0: Water content, D_{50}: Mean grain size, U_c: Uniformity coefficient, F_c: Fines content

A series of cyclic triaxial tests by the conventional stress-controlled type equipment (JGS, 2000) was conducted to examine liquefaction strength at larger strain under the undrained condition with a loading frequency of 0.1 Hz.

In this study, a strain-controlled type equipment for cyclic undrained triaxial test was also developed in order to estimate accurately the dynamic mechanical behavior (shear modulus and damping values) at small strain. This triaxial shear equipment can control the single amplitude axial strain of 0.0001 to 10% (Asonuma et al., 2002). Cyclic loading was imposed to eleven stages under the undrained condition, using a loading frequency of 0.1 Hz. Before the next stage with a higher magnitude of cyclic stress, the specimen was fully drained. Axial displacement was measured by a non-contact type displacement gauge (gap sensor). The measurement resolution of a gap sensor is 0.6 μm.

SPT and CPT were performed according to the Japanese Industrial Standard (JIS A 1219-1995 and JIS A 1220-1995) and the JGS method (JGS 1435-1995). SCP was conducted using the down hole method in which a shear wave is generated on the ground surface by hitting a plank and is caught by the receiver located on the shaft of the cone, as described by Tanaka and Tanaka (1998). The time-of-arrival of shear wave is monitored during testing.

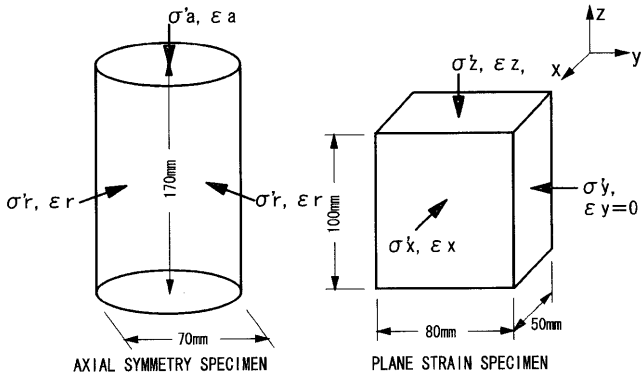

Fig. 3. Dimension of the specimen and stress and strain parameters

The shear modulus G_{sc} from SCP may be calculated by Eq. (1).

$$G_{sc}=\rho_t V_s^2 \tag{1}$$

where ρ_t is the bulk density of soil (wet density of samples) and V_s is the shear wave velocity.

TRIAXIAL COMPRESSION BEHAVIOR

Figures 4(a) and (b) show the relationships between principal stress ratio ($\sigma_1'/\sigma_3'=\sigma_a'/\sigma_r'$) and principal strain ($\varepsilon_1=\varepsilon_a$ and $\varepsilon_3=\varepsilon_r$) obtained from the triaxial compression tests on reconstituted specimens of Mori, Tomikawa and Nakashibetsu volcanic soils with consolidation pressure $\sigma_c'=49$ and 196 kPa.

From test results under the drained condition, it can be pointed out that stress-strain behavior for Mori volcanic soil resembles that for dense sand or overconsolidated clay. While the stress-strain relation for Tomikawa and Nakashibetsu volcanic soils is similar to loose sand or normally consolidated clay whose principal stress ratio monotonically increases with straining. The contractive stress-strain behavior of crushable volcanic soils is more significant with the increase in effective confining pressure σ_c'. It can be seen that the dependency of stress-strain behavior on the effective confining pressure becomes more significant under the drained condition.

In order to clarify the difference in the effective stress path for each volcanic soil, p'-q relations for undrained tests are shown in Fig. 5 in which p' and q are normalized by consolidation pressure σ_c'. For dense Toyoura sand and Mori volcanic soil, the contractive behavior changes to dilative behavior within one stress level, whereas a failure corresponding to the maximum principal stress ratio in Fig. 4(b) is generated at the positive dilatancy region. There is no such transition from dilation to contractive behavior for Tomikawa and Nakashibetsu volcanic soils.

Figure 6 shows the relationship between ϕ_d, ϕ' and effective mean principal stress at failure p_f'. Regardless of the variation in σ_c', p_f' values for undrained test are approximately the same. However, it is understood that

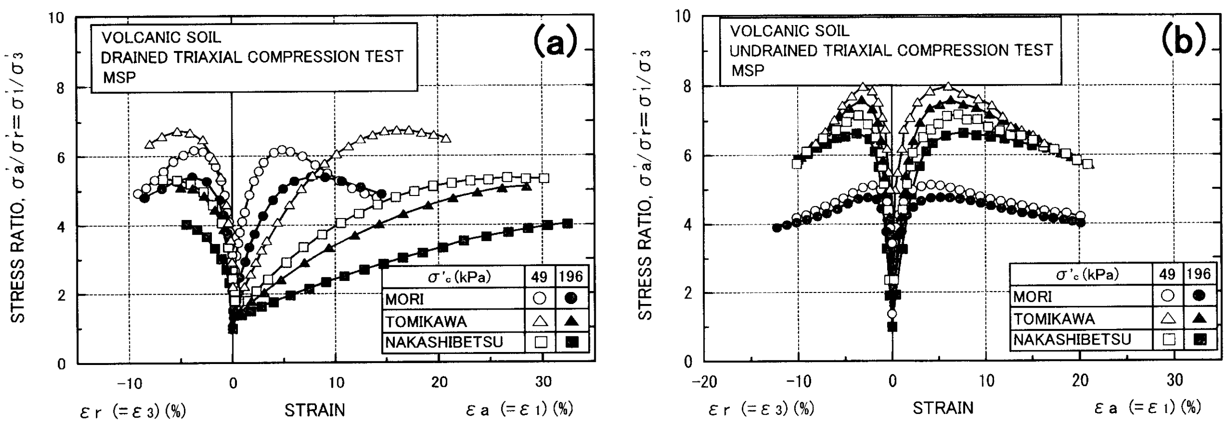

Fig. 4. Comparisons of stress-strain relationships: (a) drained triaxial compression tests and (b) undrained triaxial compression tests

for each volcanic soil that there is a unique relation between ϕ_d, ϕ' and p'_f under the drained and undrained conditions. Moreover, the ϕ_d, ϕ'-p'_f relationships obtained in Tomikawa, Kashiwabara and Bibi volcanic soils are very similar, because they are the eruption products from the same volcano. In the figure, significant decrease in the shear strength with increasing p'_f can be clearly observed for Nakashibetsu volcanic soil, whereas for Mori volcanic soil, whose constituent particles have high rigidity (Yagi et al., 2000), the level of reduction is the smallest. It is considered that the variation in shear strength due to the change of effective mean principal stress is closely related to the crushability of constituent particles. Although the dependency of particle breakage on the rigidity of constituent particles has been investigated by McDowell and Bolton (1998), Nakata et al. (1999, 2001a and b) and Yagi et al. (2000), further researches are necessary to establish a concrete relationship between the rigidity of individual particle and the particle crushing characteristics of aggregates.

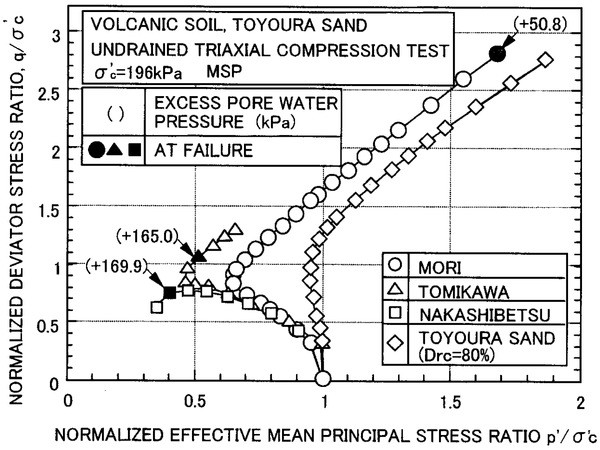

Fig. 5. Undrained effective stress paths for volcanic soils and Toyoura sand ($\sigma'_c = 196$ kPa)

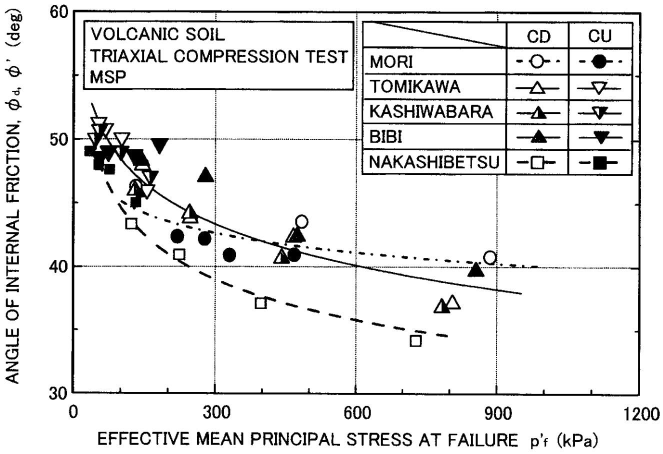

Fig. 6. Dependency of ϕ_d and ϕ' on effective mean principal stress

PARTICLE CRUSHING DUE TO SHEARING

An index which shows the quantity of particle crushing is necessary in order to quantitatively grasp the effect of particle crushing on mechanical property. An evaluation method for particle crushing proposed by many researchers is based on the changes of grain size distribution or specific surface. Miura and Yagi (1997) have shown that the degree of particle crushing for volcanic coarse-grained soils can be estimated by increment of fines content ΔF_c (75 μm or less) induced during consolidation and shear process. They also reported that there is a unique relation between ΔF_c and the indices proposed by Marsal (1965), Lee and Farhoomand (1967), Miura and Yamanouchi (1977) and Lade et al. (1996). Therefore, the particle crushing of volcanic soils here will be discussed based on ΔF_c.

In Figs. 7(a) and (b), the ΔF_c is depicted for effective stress obtained from drained and undrained plane strain tests (D.PS and U.PS tests) and drained and undrained triaxial compression tests (D.AS and U.AS tests) for volcanic soils. Regardless of the difference in the drainage condition and stress system, it seems the $\Delta F_c - p'$ relation is arranged uniquely, if the volcano is the same. In the figure, Nakashibetsu volcanic soil may be regarded

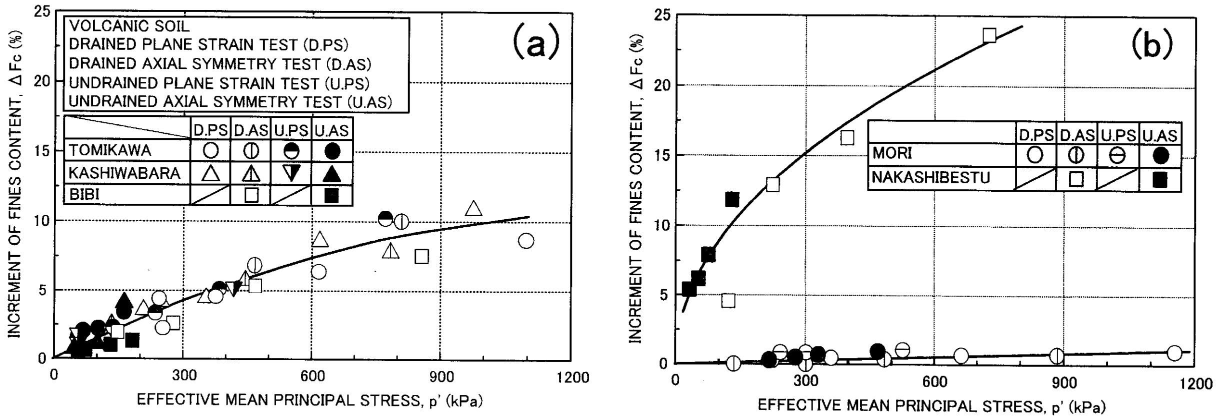

Fig. 7. Change in ΔF_c with the increase of effective mean principal stress for volcanic soils: (a) Tomikawa, Kashiwabara and Bibi, (b) Mori and Nakashibetsu

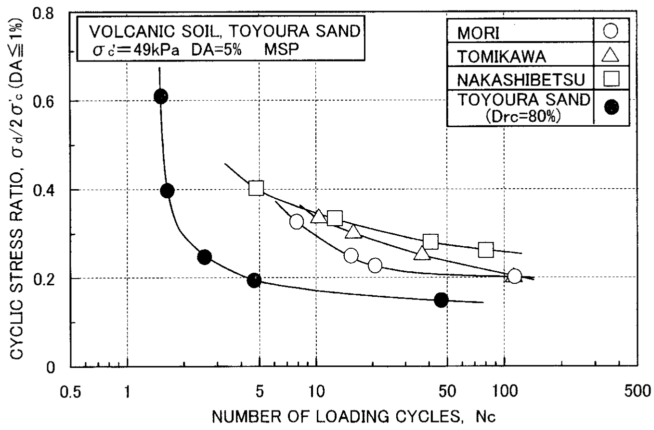

Fig. 8. Liquefaction strength for volcanic soils and Toyoura sand

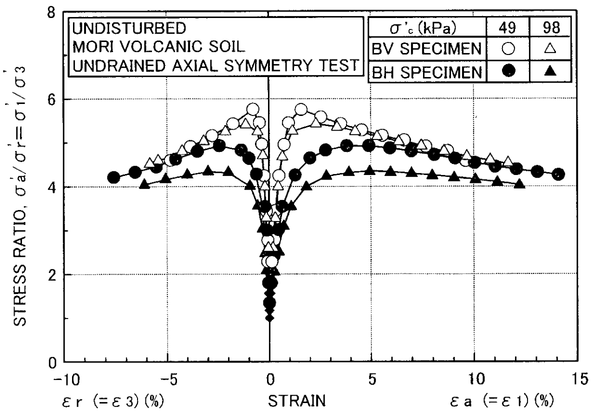

Fig. 9. Effect of fabric anisotropy on static stress-strain behavior for undisturbed Mori volcanic soil

apparently as composed of the weak particle (Yagi et al., 2000), because particle crushing of these samples is conspicuous. For example, the value of ΔF_c at $p'=600$ kPa exceeds 20%.

As explained previously, it is well known that the characteristics of particle crushing must be evaluated as an important factor influencing the stress-strain-strength relationship for granular materials with crushable particles. It is also noted that ΔF_c under the drained condition, where effective mean principal stress increases with the progress of shearing, is higher than that under undrained condition. Moreover, particle crushing in the shearing condition where the effective stress is reduced during shear, is within the measurement error of sieve analysis which is around $\pm 1\%$ for dry samples, because the stress level is low in this study.

CYCLIC UNDRAINED SHEAR BEHAVIOR

Figure 8 shows the cyclic stress ratio $\sigma_d/2\sigma_c'$ ($=SR$) versus the number of loading cycles N_c to double amplitude axial strain $DA=5\%$ obtained from cyclic undrained triaxial tests ($\sigma_c'=49$ kPa), which were performed on reconstituted specimens of volcanic soils and Toyoura sand. Although there is a variation in the specimen density ρ_{dc} after the consolidation which is reconstituted to simulate the in-situ value, the liquefaction strength of the volcanic soils is almost the same. Strength comparison between these volcanic soils and Toyoura sand ($D_{rc}=80\%$) indicates that the liquefaction resistance of volcanic soils seems to be higher than that of dense sand.

Still, particle crushing which was brought about only by the cyclic undrained shear, is very slight for all volcanic soils, as has been illustrated by Miura and Yagi (2002).

MECHANICAL BEHAVIOR OF UNDISTURBED VOLCANIC SOIL

For comparison of the mechanical behavior of BV and BH specimens, triaxial compression and plane strain tests were carried out on the Mori and Kashiwabara volcanic soils. Principal stress ratio σ_1'/σ_3' versus principal strain ε_1, ε_3, which were obtained from undrained triaxial compression tests at $\sigma_c'=49$ and 98 kPa, is depicted in Fig. 9. Comparison of test results indicates that there are differences in the static deformation-strength characteristics between both specimens. Because in-situ sampling condition and laboratory sample treatments in the present study are completely the same for both BV and BH specimens, the only difference which exists between both specimens is that the axial direction of the BV specimen is selected to be the vertical direction in natural volcanic deposits and that of the BH specimen to be the horizontal direction in natural volcanic deposits. The experimental facts indicate that the BV specimen is by far more resistant to deformation than the BH specimen. Such differences in the deformation-strength behavior of volcanic soils are very similar to that of Toyoura sand prepared by the pluviation of sand through air method (Miura and Toki, 1984). It may be estimated from the test results of Miura and Toki (1984) and Haruyama and Kitamura (1984) that anisotropy in deformation-strength properties such as that observed above is attributed to the anisotropic fabric of in-situ volcanic soils. Miura and Yagi (2002) have also shown that there is a variation in the cyclic undrained behavior attributed to the existence of fabric anisotropy in natural volcanic deposits as above.

SHEAR MODULUS AND DAMPING RATIO

Figures 10(a) to (d) depict the relationships among equivalent shear modulus G_{eq}, damping ratio h and single amplitude shear strain γ_{sa} in the cyclic triaxial tests on undisturbed (UN) and reconstituted (RE) samples. G_{eq} is defined as $G_{eq}=E_{eq}/3$. This figure also indicates the comparison of cyclic deformation properties between the volcanic soils and Toyoura sand ($D_{rc}=80\%$).

It is obvious from these figures that cyclic deformation behavior of volcanic soils depends strongly on the magnitude of the shear strain. The value of G_{eq} ($=G_0$) at $\gamma_{sa}=10^{-6}$ is shown in the table of the figures. The G_0 value

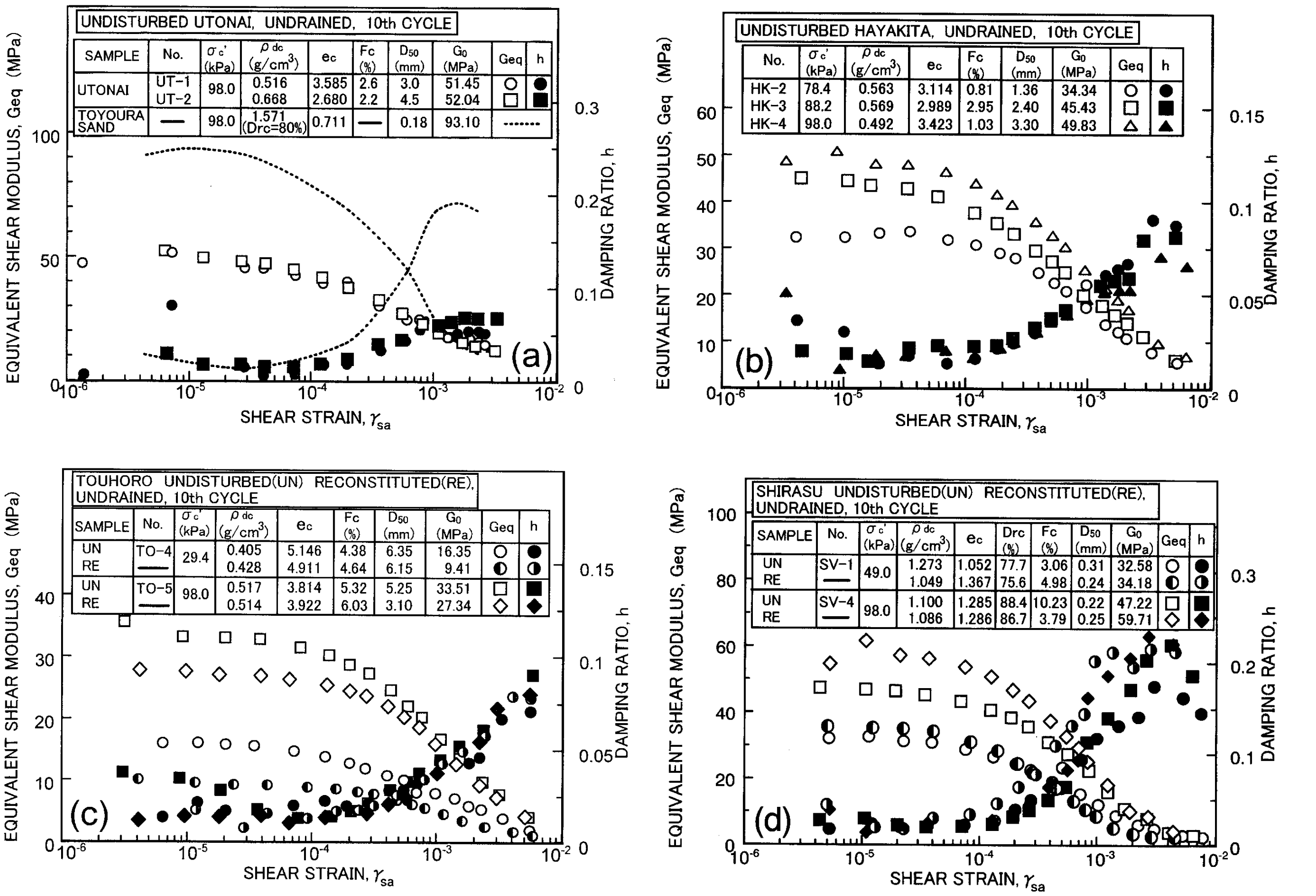

Fig. 10. Cyclic deformation properties for undisturbed (UN) and reconstituted (RE) volcanic soils: (a) Utonai, (b) Hayakita, (c) Touhoro and (d) Shirasu

for undisturbed volcanic soils (Utonai, Hayakita and Touhoro) is lower because the void ratios of volcanic soils are larger than that of Toyoura sand. Although there is no great difference in the relative density of Shirasu and Toyoura sand, G_0 of Shirasu is around half of the value of Toyoura sand.

The difference of shear modulus between undisturbed (block sample) and reconstituted specimens of Touhoro volcanic soil is clearly depicted in Fig. 10(c). Therefore, it can be pointed out that the effect of cementation exhibited among constituent particles of in-situ volcanic soils on the cyclic deformation behavior at small strain level is extremely important. However, G_{eq} of undisturbed Shirasu is lower than that of reconstituted specimen. Asonuma et al. (2002) have pointed out that this might be attributed to the disturbance at freezing which is induced by insufficient dehydration from triple tube samples.

It can be seen in the illustration of $h-\gamma_{sa}$ relations of volcanic soils (Utonai, Hayakita and Touhoro) that the damping ratio for Hokkaido volcanic soils is lower in comparison with Toyoura sand and Shirasu. On the other hand, no difference in the damping behavior between undisturbed and reconstituted specimens of Touhoro volcanic soil could be seen. It also is apparent that the h value ($\fallingdotseq 0.15$) at $\gamma_{sa}=10^{-3}$ of undisturbed Shirasu is almost equivalent to clean sand, such as Toyoura sand, and the variation of h with the increase of γ_{sa} seems smaller than that of the sand. It is further shown in the figure that there is a sudden increase of the h value at $\gamma_{sa}=10^{-3}$ for reconstituted specimen.

For comparing strain dependency of shear modulus for each volcanic soil and Toyoura sand, Figs. 11(a) and (b) are arranged based on Fig. 10 in terms of the shear modulus ratio G/G_0 versus γ_{sa} at $\sigma_c'=98$ kPa. Test results on the gravel ($e_c=0.420$, $D_{50}=10.3$ mm, $F_c=0$, $U_c=7.0$), which were obtained by Matsumoto et al. (1990), are also shown in Fig. 11(b).

As can be seen in Fig. 11(a) for undisturbed volcanic soils, the dependency of shear modulus on strain level is lower than that of Toyoura sand, and the decrease of G_{eq}/G_0 with increasing γ_{sa} is gentle. For reconstituted Touhoro volcanic soil indicated in Fig. 11(b), the strain dependency is much lower than that for other reconstituted specimens, and the $G_{eq}/G_0-\gamma_{sa}$ relation differs from that of Shirasu and Toyoura sand. It is also confirmed that lowering of G_{eq}/G_0 in the gravel induced by shear straining begins from a small strain level, and that the strain dependency is stronger than that of the volcanic soils.

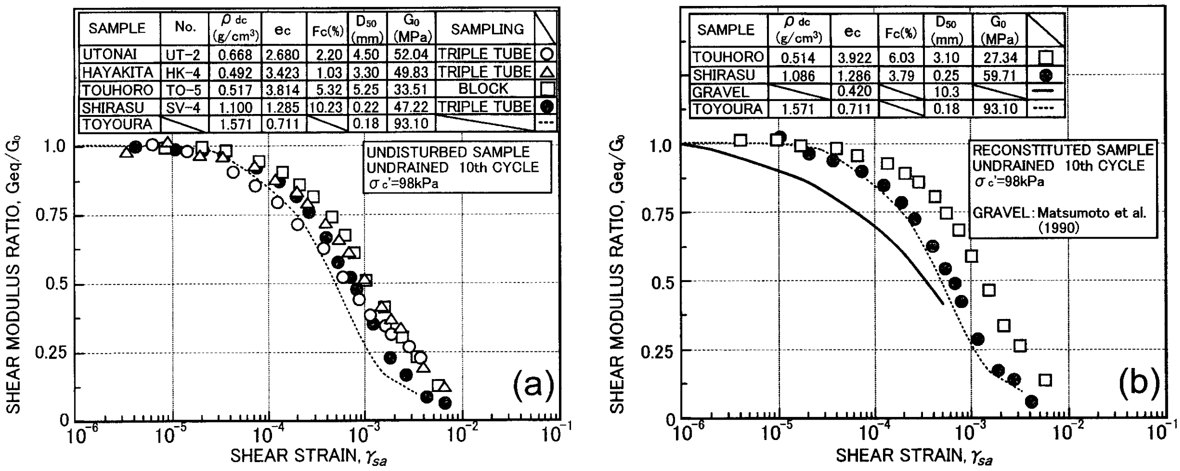

Fig. 11. Relationships between G/G_0 and γ_{sa}: (a) undisturbed samples and (b) reconstituted samples

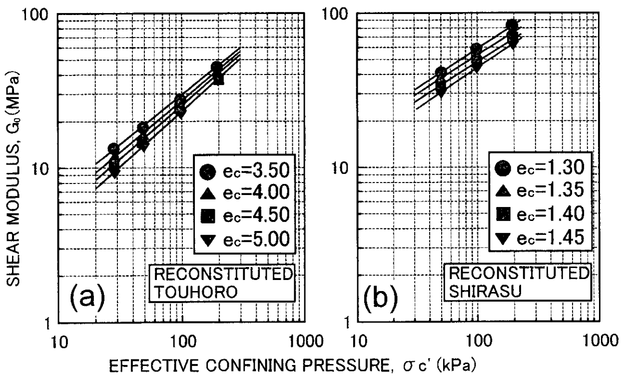

Fig. 12. Dependency of G_0 on effective confining pressure: (a) Touhoro volcanic soil and (b) Shirasu

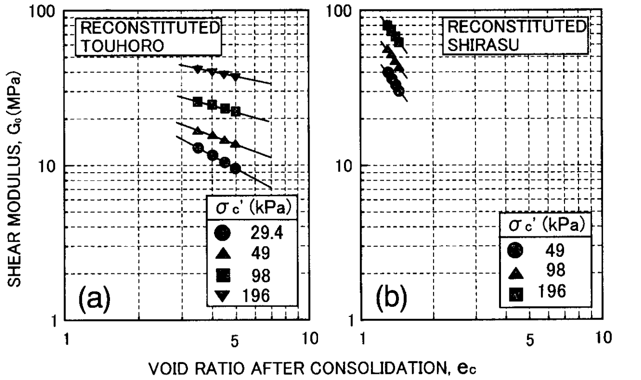

Fig. 13. Dependency of G_0 on void ratio: (a) Touhoro volcanic soil and (b) Shirasu

DEPENDENCY OF SHEAR MODULUS OF VOLCANIC SOIL ON CONFINING PRESSURE AND VOID RATIO

It is possible to express the value of G_0 as a function of σ_c' by unifying the void ratio of specimens, as shown in Figs. 12(a) and (b). In the present study, the following equations ($G_0=A\cdot F(\sigma_c')$) can be given for the volcanic soils.

$$G_0=1{,}120(\sigma_c')^{0.68} \quad \text{(in kPa, Touhoro volcanic soil with } e_c=3.50\sim5.00) \tag{2}$$

$$G_0=4{,}810(\sigma_c')^{0.52} \quad \text{(in kPa, Shirasu with } e_c=1.30\sim1.45) \tag{3}$$

The expression for Toyoura sand ($D_{rc}=80\%$, $e_c=0.711$) has been proposed by Kokusho (1980) as follows:

$$G_0=10{,}351(\sigma_c')^{0.5} \quad \text{(in kPa, Toyoura sand with } e_c=0.711). \tag{4}$$

The value n of $(\sigma_c')^n$ decreases in the order of Touhoro volcanic soil, Shirasu, Toyoura sand. It is quantitatively indicated that the effect of σ_c' on G_0 is greatly significant for the volcanic coarse-grained soils.

Figure 13 shows the relationship between G_0 and void ratio e_c for reconstituted Touhoro volcanic soil and Shirasu. G_0 can be expressed easily as a function of e_c, as indicated in the figure. The unique relations between G_0 and e_c are formulated for Touhoro volcanic soil and Shirasu by Eqs. (5) and (6), respectively.

$$G_0=45{,}400e_c^{-0.56} \quad \text{(in kPa, Touhoro volcanic soil for } \sigma_c'=29.4\sim196\text{ kPa)} \tag{5}$$

$$G_0=111{,}000e_c^{-2.46} \quad \text{(in kPa, Shirasu for } \sigma_c'=49\sim196\text{ kPa)} \tag{6}$$

Figure 14 shows the relationship between shear modulus G_{max} ($=G_0$) and void ratio e at $\sigma_v'=\sigma_c'=98$ kPa, according to the equations for sandy soils proposed previously (Hardin and Richart (1963), Shibata and Soelarno (1975), Iwasaki and Tatsuoka (1977), Kokusho (1980), Lo Presti et al. (1997)). The data of reconstituted Touhoro volcanic soil and Shirasu obtained from laboratory test in the present study is also depicted in the figure. The relations between shear modulus and void ratio which have been proposed by the researchers (Hardin and Richart (1963), Shibata and Soelarno (1975), Kokusho (1980)) are expressed as the following type of equation.

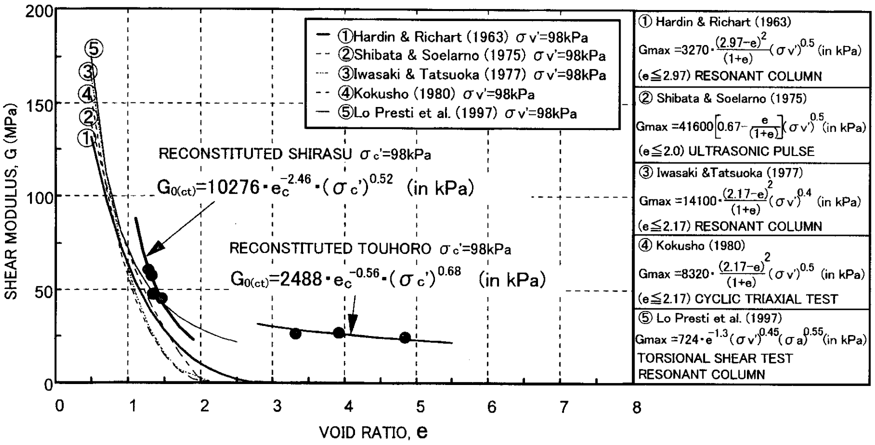

Fig. 14. Small strain shear modulus *G* related to void ratio

$$G_0(ct)=A\cdot e_c^b\cdot(\sigma_c')^c \tag{7}$$

where $G_0(ct)$ denotes shear modulus obtained from cyclic triaxial test (CT). A is a constant value. The power of b for Touhoro volcanic soil and Shirasu is obtained by Eqs. (5) and (6). The power of c also is obtained from the value of Eqs. (2) and (3).

As can be seen in Fig. 14, the range of void ratio for Shirasu is narrow and equal to the usual sandy soils. Therefore, the *G-e* relation of Shirasu seems to resemble closely most sandy soils. It should be noted that the value of void ratio for Touhoro volcanic soil is remarkably high, since the intra-particle voids may be contained in constituent particles. Such peculiarity for the volcanic coarse-grained soils consisting of crushable particles may lead to a shear modulus that is independent of the void ratio.

Wesley (2001) has reported that conventional methods of measuring specific gravity using liquid displacement procedures to determine particle volume, with or without vacuum extraction, are likely to include part of the internal void space of the particles, and therefore, would give an incorrect value for the effective volume of the particles. The resulting specific gravity is likely to be higher than the true value, leading to an underestimation of solid volume and an overestimation of void ratio. Although this study does not focus on how to determine the void ratio correctly, further research on this point should be conducted to estimate a complete intra-particle void.

EFFECT OF PARTICLE CRUSHING ON IN-SITU TEST RESULTS

The relationship between G_{sc} from SCP and N value is shown in Fig. 15. In addition, the results of diluvial and alluvial sands by Imai (1977) are also illustrated in the figure. Although the results for Shirasu and Touhoro volcanic grounds agree with that for the alluvial sand, the G_{sc} value for Utonai and Hayakita volcanic soils is higher than the alluvial sand. This phenomenon is considered to be attributed to the reduction of N value, which is induced by particle crushing caused by intrusion of the sampler of SPT. It is considered that the estimation of shear modulus by N-value for ground composed of crushable particulates such as volcanic soils is not useful without evaluation of the effect of particle crushing by results of dynamic penetration test.

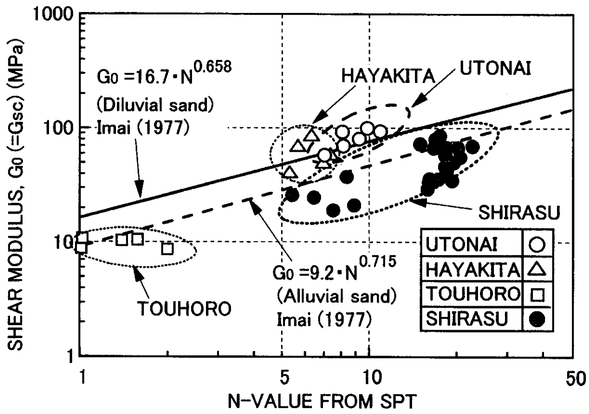

Fig. 15. Relationships between shear modulus G_0 ($=G_{sc}$) and N value from SPT

Figures 16(a) and (b) depict SPT N-value vs. q_c and q_t relationships. It can be found that the typical relationships for sandy soils obtained by Tanaka and Tanaka (1998) are similar to the results of Meyerhof (1956). Data of q_c in the figure is obtained from the Dutch double-tube cone penetration test. It has been known that q_t and q_c values are almost the same. As shown in the figure, there are linear relationships between SPT N-value and q_t value for volcanic soils and sandy soils. It should be noted that rate of increase of SPT-N value with q_t value for volcanic soils is apparently lower than that for sandy soils. These facts seem to indicate that the effect of particle crushing of volcanic soil grounds on the penetration resistance is extremely significant. q_t or q_c vs. N-value relationships for volcanic soils obtained in the present study can be expressed as follows:

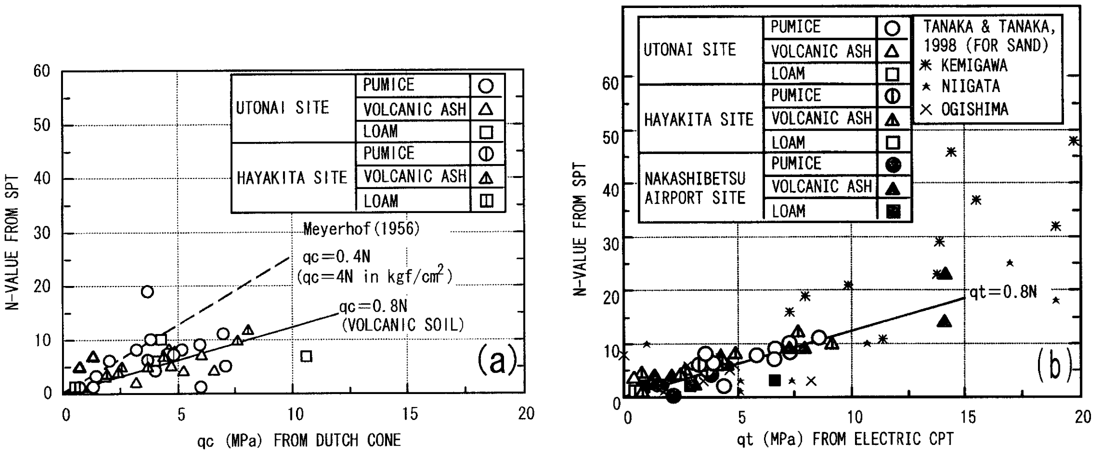

Fig. 16. **Relationships between N value and cone penetration test results: (a) q_c vs. N and (b) q_t vs. N**

$$q_t \text{ or } q_c = 0.8N \text{ (in MPa) (Volcanic soils)} \qquad (8)$$

In summary, it can be seen that N value obtained from SPT on volcanic soil grounds is underestimated with the increase of the number of dynamic penetrations. Equation (8) is useful for estimating the strength property of in-situ volcanic grounds and its dependency on crushability of constituent particles.

CONCLUSIONS

On the basis of a limited number of static and cyclic laboratory tests and in-situ tests on several kinds of volcanic soils in Japan, the following conclusions were derived.

1) In the test on volcanic coarse-grained soils where the effective mean principal stress increases during shear, the amount of particle crushing becomes large. With the increase of confining pressure, shear strength for crushable volcanic soils decreases remarkably due to effect of particle crushing.
2) Strong contractive behavior in the crushable volcanic soils is mainly attributed to particle crushing due to shear. Owing to particle crushing, the volume contraction of specimens which consist of weak particles is notable in the tests under the drained condition.
3) Strong anisotropy exists in the mechanical properties of volcanic soil grounds. Such anisotropy in mechanical properties is due to the fabric anisotropy of the in-situ ground formed during deposition.
4) Shear modulus of volcanic soils can be expressed as a function of confining pressure and void ratio as for the case of usual sand. However, compared to Toyoura sand and Shirasu, influence of void ratio on shear modulus in volcanic coarse-grained soils is not outstanding due to the existence of many intra-particle voids.
5) In grounds composed of crushable volcanic soils, particle crushing is induced due to the intrusion of the sampler at the SPT. This leads to the underestimation of N value for crushable grounds.

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