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Tsuyoshi Asonuma
Hokkaido University, Japan

Seiichi Miura
Hokkaido University, Japan

Munetaka Mae
Hokkaido University, Japan

EVALUATION OF DEFORAMATION-STRENGTH PROPERTIES OF VOLCANIC SOILS BY LABORATORY AND IN-SITU TESTINGS

Tsuyoshi ASONUMA
Graduate School of Engineering,
Hokkaido University, JAPAN

Seiichi MIURA
Graduate School of Engineering,
Hokkaido University, JAPAN

Munetaka MAE
Graduate School of Engineering,
Hokkaido University, JAPAN

ABSTRACT

The present study aims to clarify the deformation-strength properties of natural grounds consisting mainly of volcanic coarse-grained soils. Site investigations using cone penetration test (CPT), standard penetration test (SPT) and seismic cone penetration test (SCT) were performed at three sites in Hokkaido, Japan. In addition to these in-situ tests, a series of cyclic triaxial test on the undisturbed samples taken from their sites were also carried out to obtain the pseudo elastic shear modulus and damping ratio and their dependencies on strain level. From the test data, it was found that; (1) N-values obtained from SPT on crushable volcanic grounds are underestimated with the increase of the number of dynamic penetrations due to the particle breakage, (2) there are the linear relationships among q_t from CPT, shear modulus G_{sc} obtained from SCT and SPT-N value for volcanic soil grounds, and (3) small strain shear modulus obtained from the in-situ and laboratory tests is independent of void ratios of volcanic grounds.

INTRODUCTION

Natural volcanic soils have been widely distributed in, Japan. However, there are many unknown points on the control factors regarding their mechanical behaviors. Especially, it is important for evaluating the stability of the grounds to clarify the crushability and cementation possessed inherently in the volcanic coarse-grained soil. The authors have studied the effect of crushability of constituent particles on the mechanical behavior of volcanic coarse-grained soils and its evaluation method. (Miura et al.1999, Nakata et al.1998).

The objective of this study is to examine the mechanical behavior of volcanic grounds by the in-situ and laboratory tests and to quantify the deformation behaviors and their dependencies on strain level. Based on test results, a method for estimating of small strain shear modulus of volcanic coarse-grained soils is detailedly investigated.

SAMPLING SITES AND TEST MATERIALS

Volcanic soils used in this study were taken from the primary or the secondary sedimentation layers in Hayakita, Utonai and Nakashibetsu, Hokkaido, Japan (see Fig.1). Volcanic coarse-grained soils in Hayakita (①HAYAKITA) and in Utonai (② UTONAI) consist mainly of Shikotsu fall deposits (Miura et al. 1999). On the other hand, samples taken from the layers in Nakashibetsu district (③NAKASHIBETSU, ④TOHORO) belong to the layer of ejecta from Mashu volcano.

Index properties of these volcanic soils are depicted in Table 1. It must be noted in the table that the in-situ dry densities ρ_d of volcanic soils are equal to or less than 1.0g/cm^3. This reason may be that the volcanic soils are mainly composed of pumices. Furthermore, it can be found that the natural water contents of samples with low dry density are extremely high.

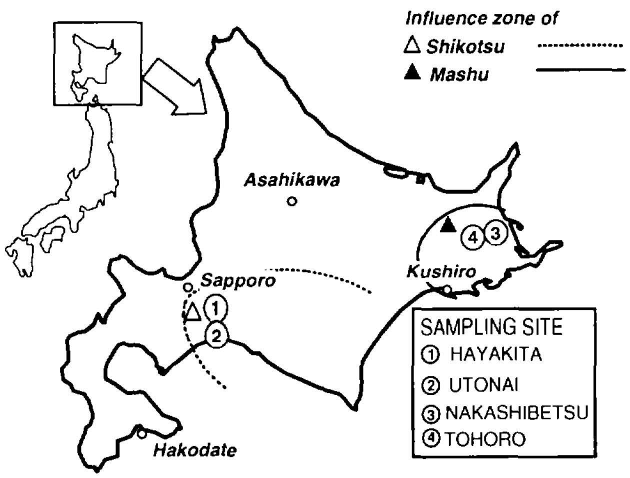


Fig. 1. Sampling sites in Hokkaido, Japan

Table 1. Index properties of volcanic soils

SITE		DEPTH (m)	σ_v' (kPa)	Gs	ρ_d (g/cm^3)	e_0	ω_0 (%)	D_{50} (mm)	Uc	Fc (%)	SAMPLING METHOD
①HAYAKITA	VOLCANIC SAND	2.9	39.2	2.80	1.003	1.792	52.2	0.75	10.00	1.79	TRIPLE TUBE
	PUMICE WITH	6.7	78.4	2.32	0.556	3.165	76.7	1.36	4.52	0.81	
	VOLCANIC SAND	8.1	88.2	2.27	0.562	3.037	96.5	2.40	5.65	2.95	
		9.4	98.0	2.18	0.487	3.476	132.6	3.30	5.68	1.03	
②UTONAI	PUMICE WITH	15.8	98.0	2.33	0.511	3.435	106.1	3.00	5.88	2.6	TRIPLE TUBE
	VOLCANIC ASH	19.0	99.0	2.46	0.662	3.078	95.0	4.50	7.12	2.2	
③NAKASHIBETSU	PUMICE	1.8	29.4	2.42	0.405	4.978	128.1	4.20	6.82	2.66	THIN WALL TUBE
		2.5	29.4	2.61	0.432	5.051	157.5	4.76	28.18	5.17	
		3.2	39.2	2.42	0.382	4.708	136.0	3.90	6.00	1.67	
	VOLCANIC SAND	5.5	58.8	2.59	1.012	1.563	39.2	0.12	——	28.94	TRIPLE TUBE
	VOLCANIC ASH	5.5	58.8	2.52	0.809	2.120	73.2	0.29	——	31.60	
④TOHORO	PUMICE	2.0	29.4	2.49	0.404	5.163	140.9	6.36	36.36	4.38	BLOCK

σ_v': Effective overburden pressure, Gs: Specific gravity, ρ_d: Dry density, e_0: Void ratio, ω_0: Natural water content, D_{50}: Mean grain size, Uc: Uniformity coefficient, Fc: Finer content

TESTING METHOD

In this site investigation, the standard penetration test (SPT), the electric cone penetration test (CPT) and the seismic cone test (SCT) were performed on the volcanic soil grounds at three sites (Hayakita, Utonai and Nakashibetsu districts). The methods of in-situ test have been described detailedly by Tanaka et al. (1998).

Undisturbed specimens used in this study were taken according to the sampling methods using triple tube, thin wall tube and block sampler (JGS 1995). After sampling, undisturbed specimens were frozen in the insulated boxes, and transported to a freezer (about -25℃) in the laboratory. Thereafter, a frozen specimen was trimmed to 70 mm in diameter, 170 mm in height.

To compare the mechanical behavior of volcanic soils with that of sands, specimens of Toyoura sand (T-sand) are prepared according to MSP method proposed by Miura et al. (1982). The relative density of specimens after consolidation D_{rc} is 80%. The variation in D_{rc} of T-sand is limited to the extent within ±3%.

After each specimen was set up in the cell, a confining pressure was raised up to 19.6kPa. A frozen specimen was melted under the confining pressure 19.6kPa for 2 hours. These specimens were saturated using the methods proposed by Rad et al. (1984) and JGS (2000). By these procedures, Bishop's B value is equal to or larger than 0.96.

After consolidating for 12 hours under isotropic consolidation pressure σ_c' which corresponds to the in-situ effective overburden pressure, a series of cyclic triaxial tests were performed under stress controlled condition with a frequency of 0.1Hz for Utonai volcanic soil.

On the other hand, deformation properties for Hayakita, Nakashibetsu and Tohoro volcanic soils were investigated by using the cyclic triaxial apparatus developed by Asonuma et al. (2000) under strain controlled condition with a frequency of 0.1Hz.

TEST RESULTS AND DISCUSSIONS

Correlation between SPT and CPT for Volcanic Grounds

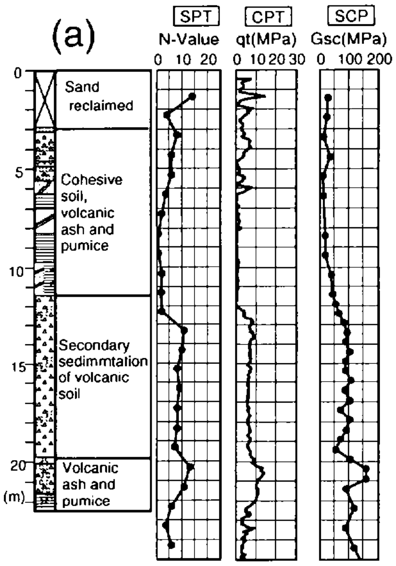
Figure 2 shows the results of in-situ tests (SPT, CPT and SCT) and soil profiles on each site. The pumice layers deposit from a depth of 12 to 20m in Utonai, from 2 to 10m in Hayakita and from 1 to 5m in Nakashibetsu, respectively. From these figures, it can be found that SPT-N value for the pumice layers are less than 10, and q_t obtained from CPT are lower values than other soil layers. The reason may be that the density of pumice layer is smaller than that of a sand layer or other layers. Therefore, no significant effect on overburden pressure is observed in SPT-N value and q_t value.

Figure 3 depicts SPT-N value vs. q_t relationships. The relationships for sandy soils obtained by Tanaka et al. (1998) are similar to Meyerhof's (1956) results in which q_c is obtained from dutch double-tube cone penetration test. In the volcanic grounds, it was found (Miura et al. 1999) that the values of q_c are close to those of q_t. As shown in this figure, there are the linear relationships between SPT-N value and q_t value for volcanic grounds. It should be noted that rate in increase of SPT-N value with q_t value for volcanic soils is extremely lower than that for sandy soils. These facts indicate that the effect of particle breakage of volcanic grounds on the penetration resistance is significant.

These q_t or q_c vs. N-value relationships can be expressed as follows;

q_t=0.8N (in MPa) (Volcanic soils) (1)
q_c=0.4N (in MPa) (Meyerhof) (2)

It is found that N-value obtained from SPT on volcanic soil grounds is underestimated with the increase of the number of



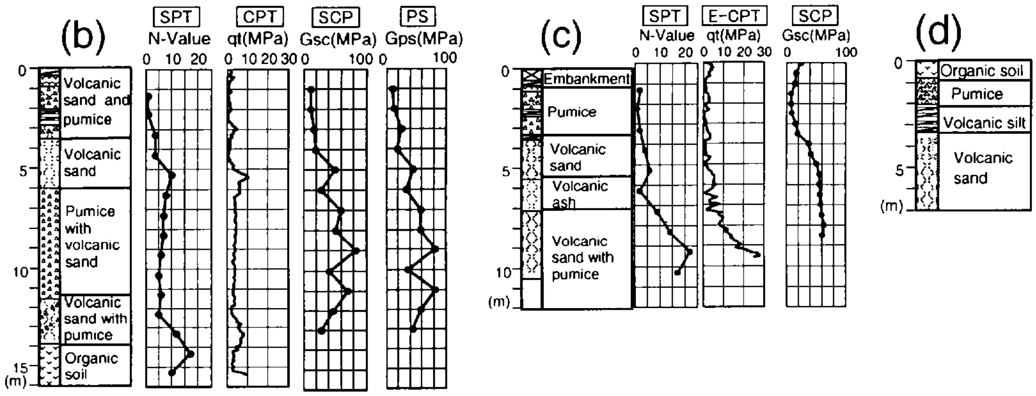


Fig. 2. Soil profiles for volcanic grounds; (a)Utonai, (b)Hayakita, (c) Nakashibetsu and(d)Tohoro

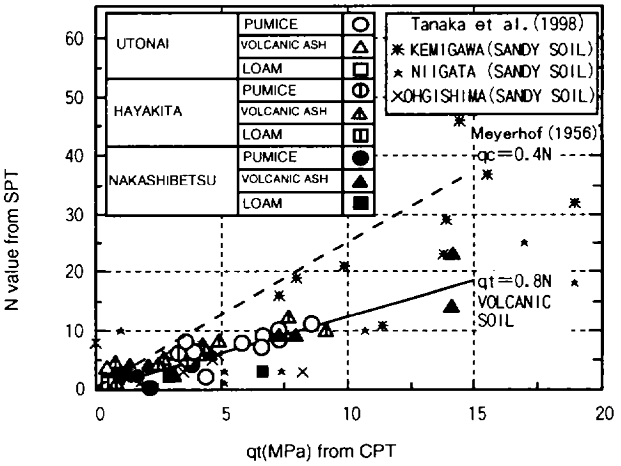


Fig. 3. Relationships between q_t and N-value

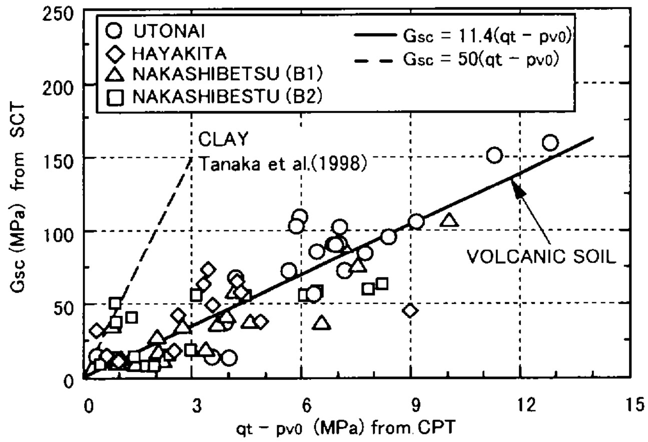


Fig. 4. Pseudo elastic shear modulus from SCT related to (q_t-p_{v0})

dynamic penetrations. In any case, these relationships are useful for estimating the strength property of in-situ volcanic grounds and its dependency on crushability of constituent particles.

Small Strain Shear Modulus Obtained from In-situ and Laboratory Tests

In the present study, in-situ shear modulus G_{sc} which were obtained from SCT are compared with q_t given by CPT (see Fig.4). The shear strain corresponding shear modulus is less than 10^{-6}. Tanaka et al. (1998) has presented the relationships between (q_t-p_{v0}) and G_{sc} for the clay as shown in Equation (3);

$$G_{sc} = 50(q_t - p_{v0}) \qquad (\text{in MPa}) \qquad (3)$$

where p_{v0} is the overburden pressure at total stress. For volcanic soils, the shear modulus similarly increases with the increase of (q_t-p_{v0}). The relationships between (q_t-p_{v0}) and G for natural volcanic grounds can be also expressed as follows;

$$G_{sc} = 11.4(q_t - p_{v0}) \qquad (\text{in MPa}) \qquad (4)$$

Furthermore, if the effect of overburden pressure p_{v0} on strength behavior of volcanic grounds is disregarded as mentioned above, the following relationship is obtained,

$$G_{sc} = 11.2 q_t \qquad (\text{in MPa}) \qquad (5)$$

In these expressions, it must be noted for volcanic soil grounds that there are the proportional relationships among N-value, G_{sc} and q_t which are similar to the sandy grounds. Therefore, it may be possible to estimate the small strain shear behavior of in-situ volcanic soil grounds from in-situ tests.

Figure 5 exhibits the relationships among equivalent young's modulus E_{eq}, damping ratio h and single amplitude axial strain $(\varepsilon_a)_{SA}$ in cyclic triaxial tests (CTX) on undisturbed samples. This figure also indicates the comparison between cyclic deformation properties for volcanic soils and that for T-sand (D_{rc}=80%). It is obvious from these figures that cyclic deformation behavior of volcanic soils depends strongly on the magnitude of the axial strain. The equivalent young's modulus E_{eq0} at $(\varepsilon_a)_{SA} = 1\times10^{-5}$ for undisturbed volcanic soils is lower values due to that the confining pressure and the specimen density of volcanic soils are smaller than those of T-sand.

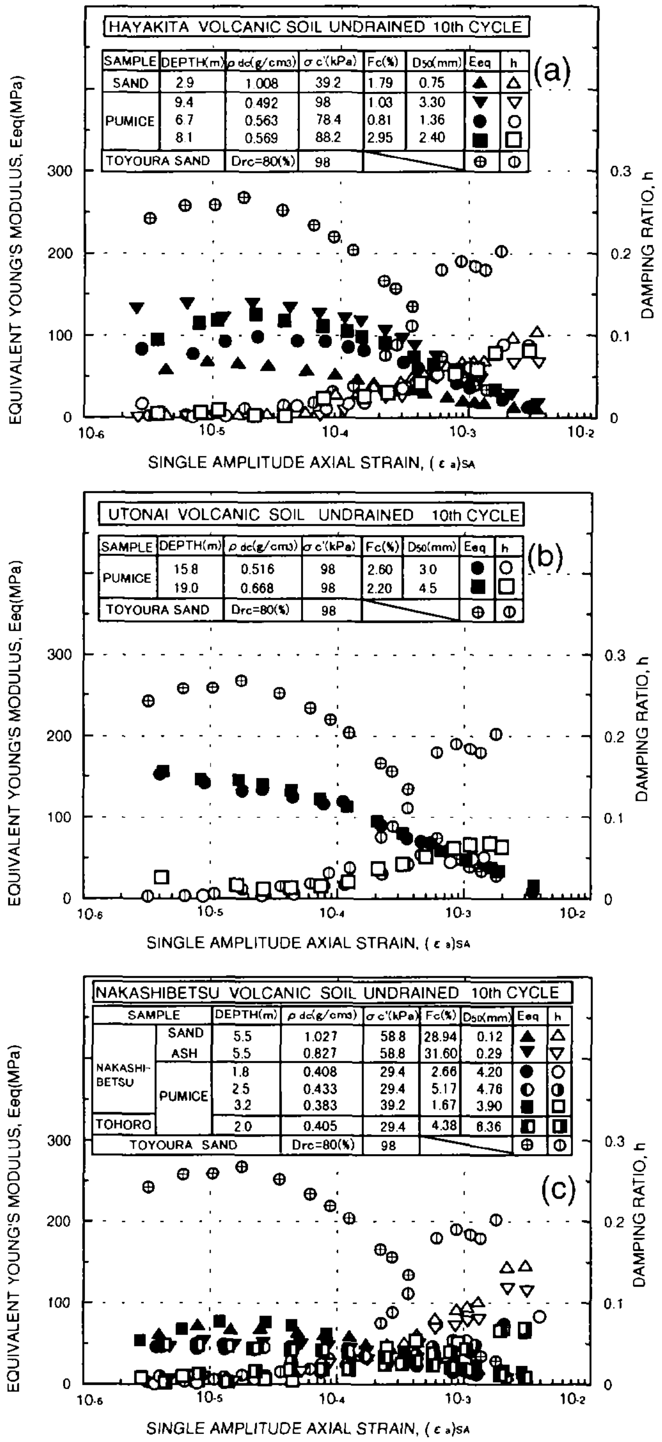


Fig.5. Cyclic deformation properties for volcanic soils; (a)Hayakita, (b)Utonai and (c)Nakashibetsu

Furthermore, the damping ratio h at $(\varepsilon_a)_{SA}=1\times10^{-3}$ of volcanic soils become also lower values in comparison with that of dense T-sand. Therefore, the effect of cementation among constituent particles on the cyclic deformation behaviors at small strain levels of in-situ volcanic soils is extremely important.

Figure 6 is arranged in terms of the shear modulus ratio G/G_0 vs. $\log\gamma$ based on Fig.5. G is defined as $G=E_{eq}/3$ and G_0 is the shear modulus at $\gamma=1\times10^{-6}$. In these figures, G/G_0 - γ relationships for T-sand are also depicted.

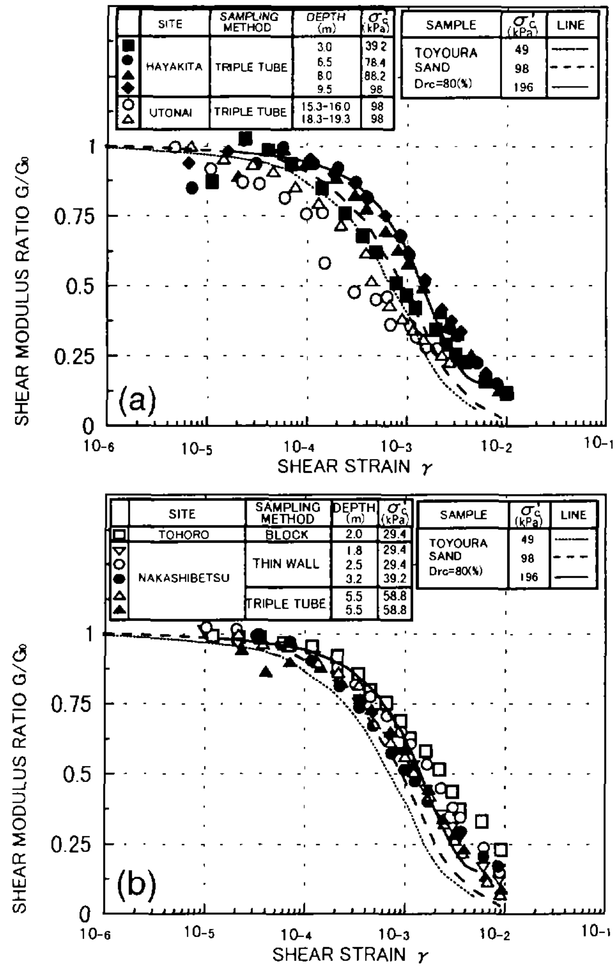


Fig.6. Relationships between shear modulus ratio, G/G_0 and $\log\gamma$; (a)Hayakita and Utonai Volcanic soils and (b)Tohoro and Nakashibetsu volcanic soils

From Fig. 6(a), it can be seen that G/G_0 behavior for Hayakita volcanic soil has the similar tendency of that of T-sand. But variations of G/G_0 in Utonai volcanic soil exists notably. Therefore, it is pointed out that the estimation of sample disturbance is essential for volcanic soil.

The results of Tohoro and Nakashibetsu volcanic soils are illustrated in Fig. 6(b). G/G_0 for the volcanic soil grounds are higher value than that of sand nevertheless the effective confining pressure is low. It may be considered that the cementation of constituent particles develops strongly in these volcanic grounds. Changes in G/G_0 due to the difference in methods of sampling such as block and thin wall sampling exist on the pumice layer. From these results, it needs to quantify the change in deformation-strength properties of volcanic soils attributed to the difference of sampling methods.

Figure 7 exhibits the variations in the small strain shear modulus with depth. In this figure, the shear modulus G_{sc}, G_{ps} obtained from PS logging and G_{ct} estimated from cyclic triaxial test (CTX) are plotted. As shown in Fig.7 (a), there are variations of the distribution of G_{sc} from a depth of 12 to 20m for Utonai and from

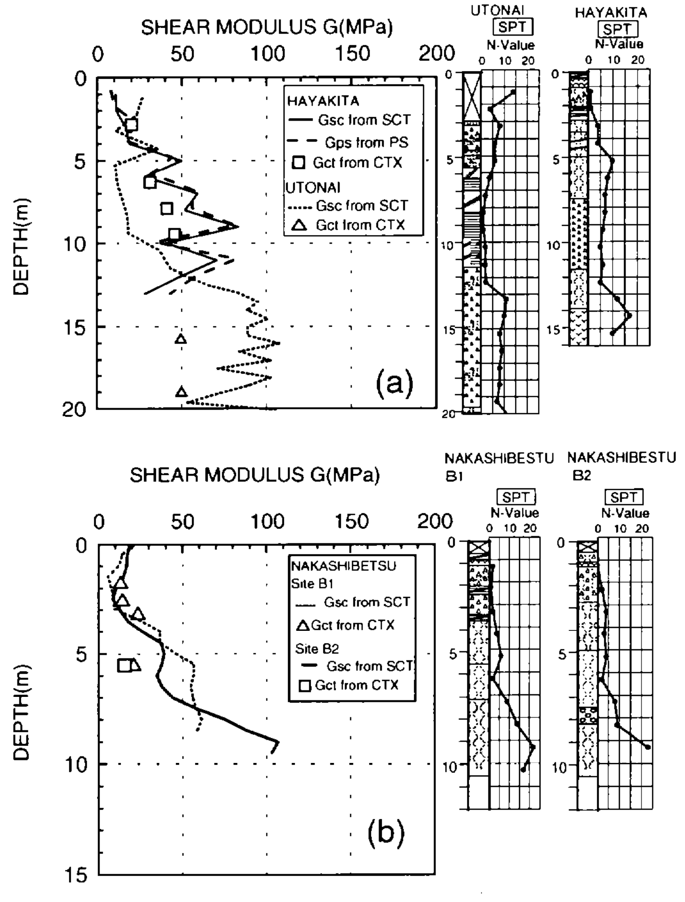


Fig.7. Comparison of pseudo elastic shear modulus between in- situ and laboratory tests; (a)Hayakita and Utonai volcanic soils and (b)Nakashibetsu volcanic soils

7 to 12m for Hayakita irrespective of the uniform layers. Hayakita and Utonai volcanic soils belong to the same eruption sources (Shikotsu eruption) and the distance between two sites is about 8km. Comparison among G_{ct} estimated from laboratory tests, G_{sc} and G_{ps} indicates that the small strain shear modulus is almost coincident to G_{sc} from in-situ tests value for Hayakita volcanic soil. For Utonai site, on the other hand, G_{ct} value becomes smaller than G_{sc} value in depth of 15 to 20m. Similarly, Fig. 7(b) illustrates the test result for Nakashibetsu. The in-situ tests are performed in two sites (B1 and B2) whose distance is about 50m. From this figure, it can be found that G_{sc} value corresponds well with G_{ct} value in a depth of 0 to 3m. This indicates that the effect of sample disturbance on the small strain shear behavior of volcanic soil is dependent on sampling method.

Variation in Small Strain Shear Modulus due to Sample Disturbance

In order to evaluate the influence of sample disturbance on cyclic deformation behavior, variation of small strain shear modulus due to the difference in sampling methods is also examined. As mentioned in above, undisturbed volcanic specimens are taken according to triple tube sampling method at the sites in Hayakita and Utonai. On the other hand, undisturbed Nakashibetsu

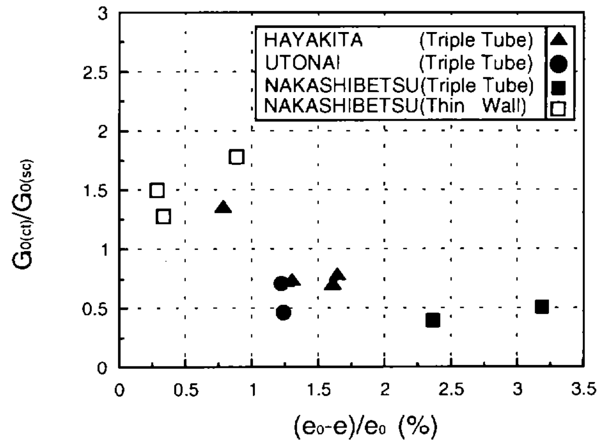


Fig.8. Influence of sample disturbance on shear modulus of volcanic soil

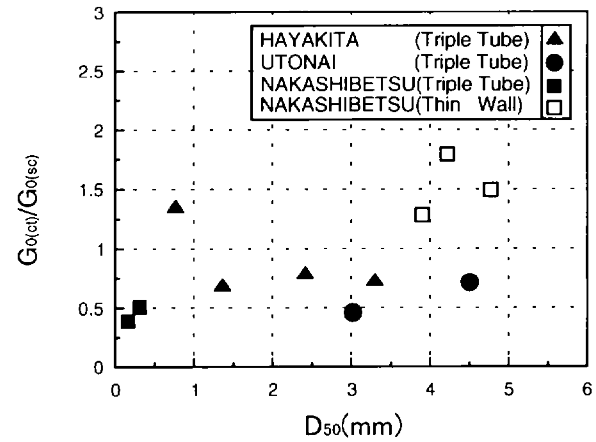


Fig.9. Comparison of the disorder on specimen by the sampling method

volcanic soils were given according to thin wall and triple tube sampling methods.

The relationships between shear modulus ratio $G_{0(ct)}/G_{0(sc)}$ and change in void ratio due to the isotropic consolidation are shown in Fig. 8. In the figure, e_0 and e denote the in-situ void ratio of a volcanic specimen and the void ratio of the specimen after isotropic consolidation (Shiwakoti et al. 2000), respectively. $G_{0(ct)}/G_{0(sc)}$ decreases apparently with the increase in void ratio.

Furthermore, it must be also noted that variation of $(e_0-e)/e_0$ due to the difference in sampling methods is remarkable. For example, $(e_0-e)/e_0$ value is less than 1(%) for thin wall sampling and is equal to or more than 1(%) for triple tube sampling. From this result, it is said that triple tube sampling exerts an influence on the sample quality.

To evaluate the influence of the particle size on in-situ dynamic behaviors of volcanic soil grounds, the relationships between D_{50} and $G_{0(ct)}/G_{0(sc)}$ are examined in Fig.9. $G_{0(ct)}/G_{0(sc)}$ rather fluctuates with the change in D_{50}. Therefore, the effect of particle size on the sampling method seems to be vague.

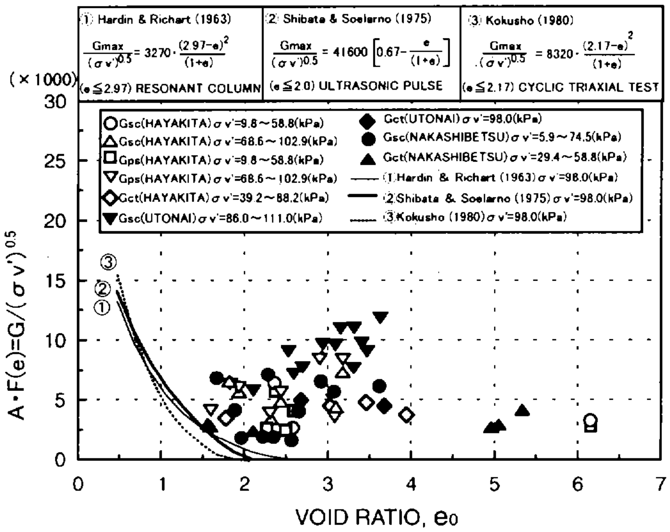


Fig.10. Small strain shear modulus G related to void ratio for volcanic soils

<u>Relationships between Shear Modulus and Void Ratio Obtained by Laboratory and In-situ Testing</u>

Variations of the shear modulus due to the difference of void ratio is investigated. The normalized shear modulus $G/(\sigma_v')^{0.5}$ vs. initial void ratio e_0 relationships is shown in Fig.10. σ_v' means the effective vertical stress. $G/(\sigma_v')^{0.5}$-e_0 relations are arranged based on the results of SCT, PS logging and CTX. The correlations for sandy soils (σ_c'= 98kPa) are also depicted in Fig.10. Many researchers (e.q. Hardin et al.(1963), Shibata et al.(1975) and Kokusho (1980)) have proposed the correlations as follows;

$$A \cdot F(e) = G_{max} / ((\sigma_v')^n \cdot (\sigma_a')^{n-1}) \tag{6}$$

where A, F(e), σ_a' and n are a constant value, a function of void ratio, effective mean principal stress and an exponent, respectively.

It can be seen that the value of $G/(\sigma_v')^{0.5}$ of volcanic soils is extremely greater than that of sandy soils. There are no unique relationships between $A \cdot F(e)$ and e_0. Such peculiarity for volcanic soil consisting of crushable particles may lead that shear moduli from in-situ and laboratory tests are independent of the void ratio. At any rate, it may be said that at least e_0 value for volcanic coarse-grained soils is not significant for evaluating the small strain shear behavior.

CONCLUSIONS

From a series of cyclic deformation tests and in-situ tests on volcanic coarse-grained soil grounds, the following conclusions were derived;

1. N-values obtained from SPT on crushable volcanic grounds are underestimated with the increase of the number of dynamic penetrations due to the particle breakage.
2. There are a good correlation among q_t, shear modulus G_{sc} obtained from SCT and SPT-N value for volcanic soil grounds, which is similar to the linear relation for sandy grounds.
3. For volcanic coarse-grained soils, the shear modulus estimated from in-situ and laboratory tests is independent of void ratios.

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