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LIQUEFACTION RESISTANCE OF TWO ALLUVIAL VOLCANIC SOILS SAMPLED BY IN SITU FREEZING

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ABSTRACT

Two new improved-efficiency techniques for soil sampling by in situ freezing were used to obtain undisturbed samples of saturated alluvial volcanic soils, locally termed "Shirasu", from Kagoshima city, Japan. Undrained cyclic triaxial tests were conducted on the undisturbed alluvial Shirasu samples obtained from a depth of 5.4 to 6.0 m at site *A*, and from a depth of 8.4 to 10.5 m at site *B*, after they had been thawed. For studying the effects of sample disturbance, undrained cyclic triaxial tests were also conducted on the reconstituted specimens. The liquefaction resistance in the field was estimated based on the laboratory tests. Based on the above studies it was shown that (1) two new developments in the technique of sampling by in situ freezing were successfully achieved in the field, and (2) compared with the liquefaction resistance of the undisturbed samples, that of the reconstituted samples from site *A*, all of which had about the same density, was about 45 to 50% of the value for undisturbed samples, and that of the reconstituted samples from site *B*, with relative density 13 to 20 percentage points higher than that of undisturbed samples was about 62 to 67%.

Key words : consolidated undrained shear, liquefaction, sample disturbance, sampling, volcanic soil, shear strength, (**IGC** : D 7/C 6)

INTRODUCTION

In the southern part of Kyushu, Japan, there is a wide distribution of soil deposits which resulted from volcanic activities in the Quaternary epoch. Shirasu (a local term) is one such volcanic deposit, and it has the widest area of distribution, as shown in Fig. 1(a) in the shaded area. A previous study by Haruyama (1973) shows the details of the geological, physical, and mechanical properties of Shirasu. The unique features of Shirasu soil grain and soil deposits are : (1) the specific gravity of soil grain ranges mostly from 2.35 to 2.55, which is lower than that of ordinary sands ; (2) the soil grain has an angular shape and can be easily crushed by fingering ; and (3) the dry density is much lower than that of ordinary sands.

Alluvial Shirasu deposits with characteristics such asthose described above would usually be found to have a thickness of 100 m

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or more, although in the standard penetration test (SPT) we can find no clear increase of the N-value with the depth. These facts usually present difficulties for soil engineers who are attempting to confirm the existence of a bearing stratum to support buildings by basing their judgments only on N-values.

Although many studies (for example, by Haruyama, 1973) have been concerned with the physical properties and static mechanical properties of Shirasu, only a few studies (Oh-hara et al, 1974; Yamanouchi et al, 1976) have been presented which discuss the undrained cyclic shear strength of undisturbed Shirasu.

The object of this paper is to present the results of laboratory test of the liquefaction resistance of undisturbed samples of two Shirasu samples from Kagoshima city, Japan, obtained by in situ freezing, and to compare the results both with those of reconstituted samples and with the estimated results obtained by the presently available simplified procedures based on N-values and soil gradations.

SOIL SAMPLING BY IN SITU FREEZING AT SITE A

Sampling site A was located in Kagoshima city, southern Kyushu, as shown in Fig. 1(a). Standard penetration tests were conducted at five locations near the place of sampling as shown in Fig. 1(b). Static cone penetration tests were also performed at two locations between the SPT test locations as shown in Fig. 1(b). Fig. 2(a) shows the soil profile, the N-value of the standard penetration test by the trip monkey (tonbi) method, and the depth of the soil samples for which undrained cyclic triaxial test results are reported in this paper. Except for the silt soil layer at the depth of about 4 m from the ground surface, the soil consists of fine sand, medium coarse sand, and coarse sand with gravel or pumice, with some fines. On this site, the alluvial Shirasu deposit as opposed to fill Shirasu is considered to be below the depth of about 5 m from the ground surface. There are some variations in the N-values at each depth.

A 75-mm hole was drilled to a depth of 10 m with a rotary drilling machine using bentonite mud, and a steel pipe 73 mm in outside diameter and 6mm in wall thickness, with the lower end closed, was lowered into the hole very slowly in order to minimize the excess pore water pressure in the bore hole. After installation of the pipe, a copper pipe 15.9 mm in outside diameter and 3.5 mm in wall thickness was placed in the steel pipe, as shown in Fig. 3(1). Liquid nitrogen from a tank truck was supplied from the upper end of the copper pipe to freeze the surrounding ground outside the steel pipe, as shown in Fig. 3(1). Thermocouples for monitoring the ground tempera-

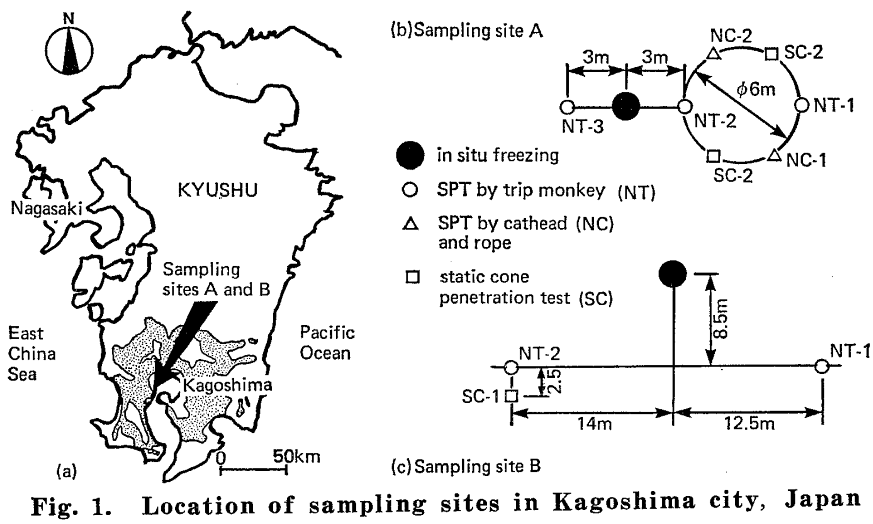


Fig. 1. Location of sampling sites in Kagoshima city, Japan

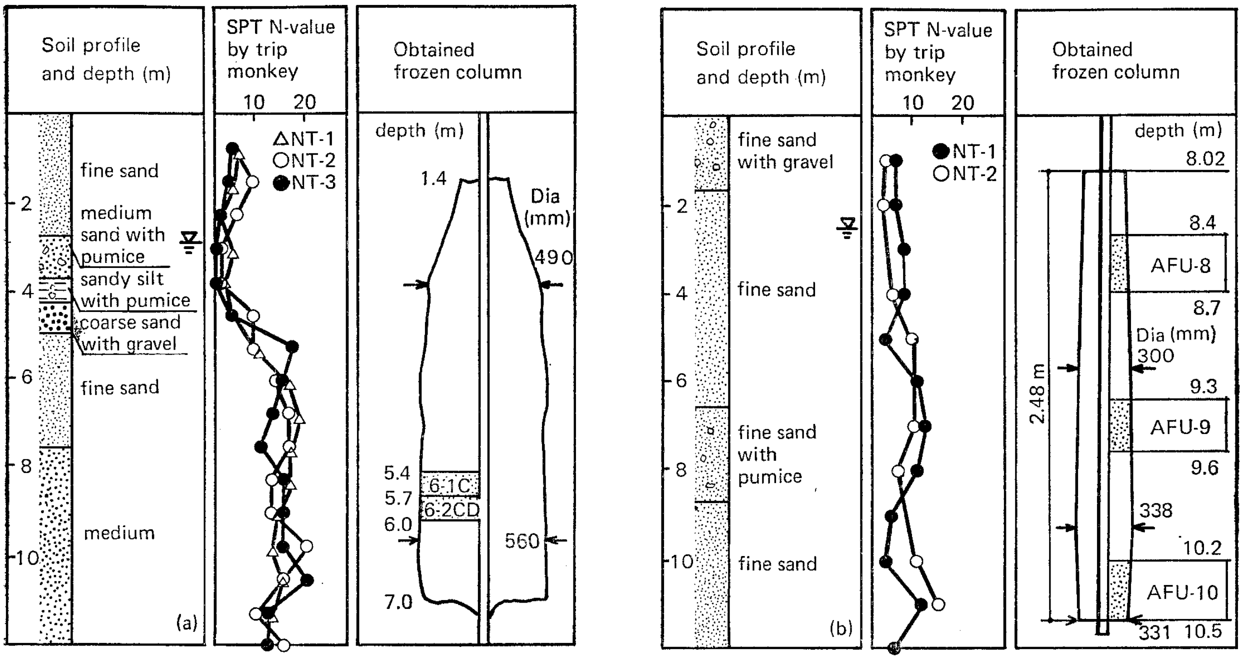


Fig. 2. Soil profiles at sampling sites

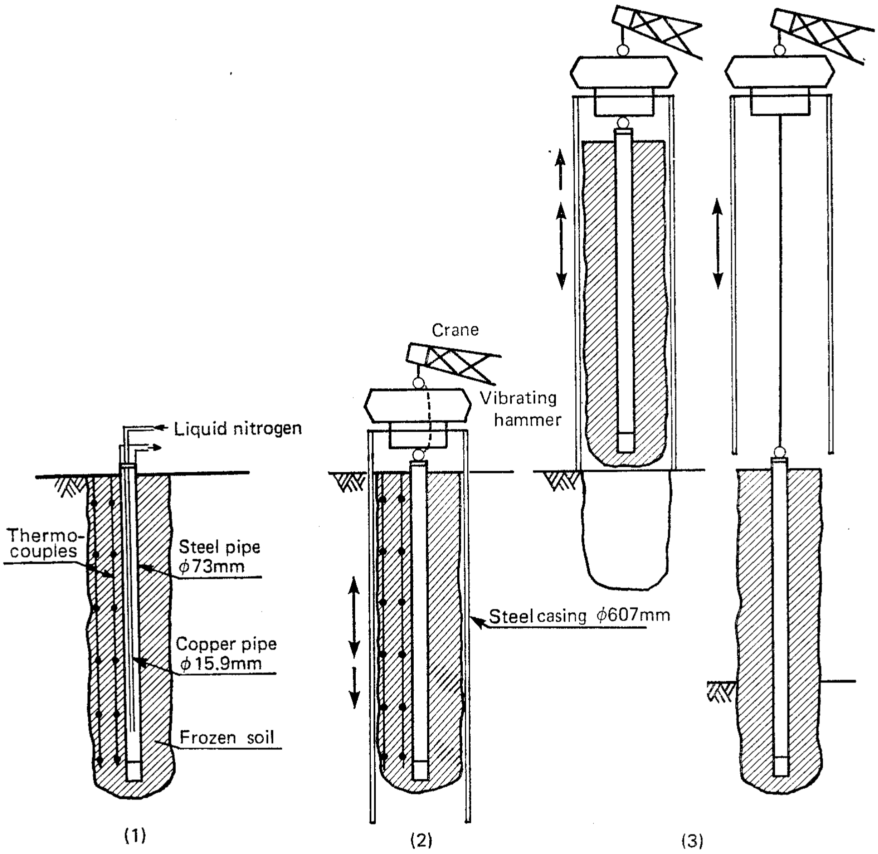


Fig. 3. In situ freezing sampling method used at site *A*

ture in order to predict the freezing front and to regulate the rate of flow of liquid nitrogen are also shown in Fig. 3(1). Continuous supplement of the liquid nitrogen for about 40 hours yielded a column of frozen sand with a diameter of about 60 cm.

A simple method was attempted here to obtain the whole frozen sample. A steel casing, 607 mm in outside diameter and 584 mm in inside diameter was lowered to a depth of 11 m by vibrating it with a 4000-type vibrating hammer as shown in Fig. 3(2). After connecting the head of the 73-mm freezing pipe to the vibrating hammer to avoid the drop of the frozen soil column, as shown in Fig. 3(3), the frozen soil was pulled out together with the steel casing by keeping the casing vibrating during lifting to reduce the friction between the steel casing and the surrounding ground.

This technique will lower the cost of sampling. However, due to the noise and the ground vibration while vibrating the steel casing, this method may be limited in the field ; it could be appropriate where the average N-value is lower than about 15 and where the sampling location is far from residential areas, unless some techniques such as water jetting are employed to reduce the noise and vibration.

The frozen sample was cut with a chain saw into blocks of convenient size and carefully wrapped and sealed in vinyl sheets to minimize sublimation of the pore ice. The blocks were transported in a refrigerated truck to the laboratory for cold storage at −20 to −30 deg. C.

SOIL SAMPLING BY IN SITU FREEZING AT SITE B

Previous studies on in situ freezing sampling by Yoshimi et al (1977, 1978, 1984) and Makihara et al (1981) showed a method of obtaining a continuous frozen soil column from the ground surface to a depth of about 10 m or less ; this was the procedure which was carried out by the authors at site A. However, it is not always necessary to obtain the full length of undisturbed sand down to the desired sampling depth below the ground surface. For practical purposes, it is useful to develop a method for obtaining, a sample from a sand layer with a limited sample length at any given depth below the ground surface ; by such a method we can obtain undisturbed sand from a much deeper soil layer, to a depth of about 30 m.

Sampling site B was also located in Kagoshima city as shown in Fig. 1(a). Standard penetration tests were conducted at two locations near the place of sampling as shown in Fig. 1(c). Fig. 2(b) shows the soil profile, the N-value of the SPT by the trip monkey method, and the depth of soil samples for which undrained cyclic test results are reported in this paper. The soil layer mainly consists of medium to fine sand with some fines content, and pumice. The N-values of SPT were found to be less than 10 to a depth of 30 m below the ground surface, although only the results to a depth of about 12 m are shown in Fig. 2(b), which shows a typical distribution of N-values of the alluvial Shirasu deposit.

The method of sampling by in situ freezing used at site B to obtain a soil sample from a depth of 8.5 to 10.5 m consists of the following steps, and is shown in Fig. 4 :

(1) A 420-mm hole was drilled to a depth of 8 m with a DH-4 type rotary boring machine as shown in Fig. 4(1), and an open-end steel pipe 406 mm in diameter and 9.5 mm in wall thickness was placed in the hole in order to prevent the collapse of the wall of the hole. A soil layer 50 cm in thickness was retained above the target sand layer to serve as a surcharge for preventing the soil expansion due to freezing and for keeping the target sand layer from melting during coring operations.

A 55-mm hole was drilled to a depth of 10.6 m with a DH-4 type rotary boring machine in the center of the 420-mm hole. A steel tube 50.8 mm in diameter and 4 mm in wall thickness, to the end of which was attached a polyvinyl chloride rod 50 mm in diameter and 300 mm in length, was installed

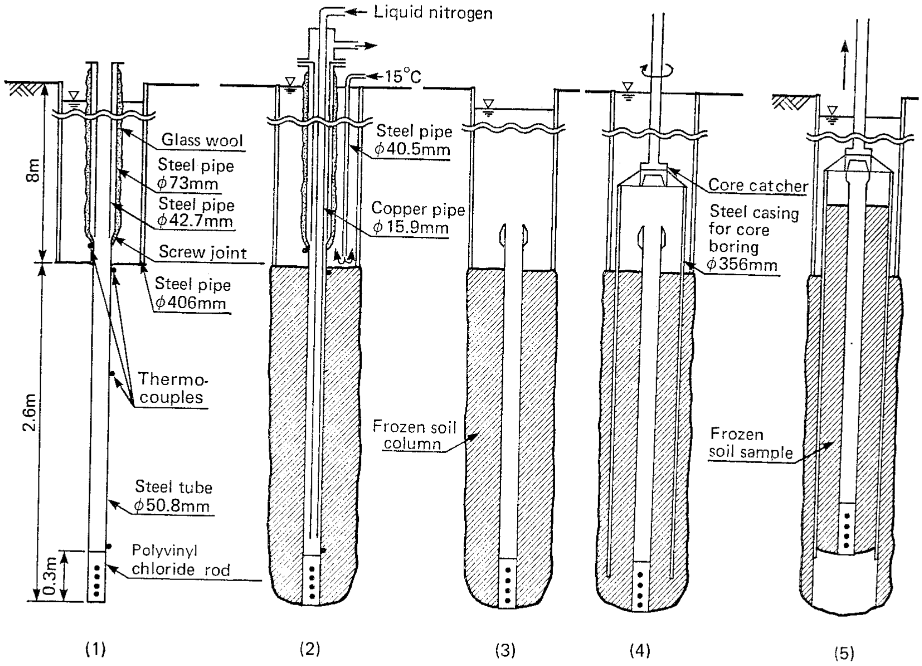


Fig. 4. In situ freezing sampling method used at site *B*

into the 55-mm hole to a depth of about 10.6 m, which is somewhat below the depth of the sand layer to be sampled, as shown in Fig. 4(1). Four thermocouples for monitoring the ground temperatures were attached on the surface of the polyvinyl chloride rod at distances of 100, 150, 200 and 250 mm from the bottom of the steel tube. The diameter of the frozen column of soil was estimated from the monitored temperatures, based on former full-scale laboratory test results which indicated that the soil below the bottom of the freezing pipe would freeze in a semispherical shape.

The upper end of the steel tube was screwed to another steel tube 73 mm in diameter and 10 mm in wall thickness. The surface of the 73-mm steel tube was wrapped with glass wool using waterproof tape, both for the purpose of conserving the liquid nitrogen and for keeping the water in the hole around the tube from freezing, in order to facilitate removal of the 73-mm tube from the 50.8-mm steel tube by unscrewing the joint after the completion of the ground freezing.

One more steel pipe 42.7 mm in diameter and 3.5 mm in wall thickness was installed in the 73-mm steel pipe as shown in Fig. 4(1), down to a depth somewhat below the place where the two steel pipes were joined ; this was done for the same heat-insulation purposes described above.

(2) A copper pipe 15.9 mm in outside diameter and 3.5 mm in wall thickness was placed in the 42.7-mm steel pipe to a depth of 10.2 m, which is 10 cm above the bottom of 50.8-mm steel tube. The liquid nitrogen supplied from the upper end of the copper pipe was allowed to rise in the annular space between the two pipes (15.9-mm pipe and 50.8-mm pipe).

Liquid nitrogen was supplied from pressurized vessels 360 kg or 720 kg for about 37.5 hours to obtain a frozen soil column about 40 cm in diameter. Due to the small container size and the time-consuming necessity of changing vessels every three to seven hours, the freezing process took considerably more time than expected ; and a total of 4000 kg of liquid nitrogen was consumed.

The temperature of the water in the hole around the screw-joined area was monitored by the thermocouples during ground freezing. In case the temperature of the water around that area dropped lower than 1°C, drilling mud with a temperature of about

15°C was supplied through a steel pipe to that area in order to keep the water from freezing, as shown in Fig. 4.

(3) After the desired thickness of frozen soil was achieved, the 15.9-mm copper tube and the 42.7-mm steel pipe were removed. The 73-mm steel pipe was removed from the 50.8-mm steel pipe by unscrewing. This process took about 65 minutes.

(4) For obtaining the frozen soil column, a steel casing 355.6 mm in diameter, 11 mm in wall thickness and 3,850 mm in length with hard metal teeth was lowered by a DH-4 type rotary boring machine using bentonite mud with a temperature of about 10 to 15°C. It took about 30 minutes to lower the steel casing from 7.5 m to 10.7 m; from a depth of 7.5 m to 8.0 m there was a layer of frozen bentonite mud, and from 10.5 m to 10.7 m was an unfrozen soil layer.

(5) At the final stage of lowering the casing, when the lowest end of the steel casing reached a depth of about 10.7 m, the core catcher built-in at the inside of the casing head (shown in Photo. 1) was screwed to the upper end of the 50.8 mm steel pipe. The successful joining of the casing and the 50.8-mm pipe can be confirmed by observing

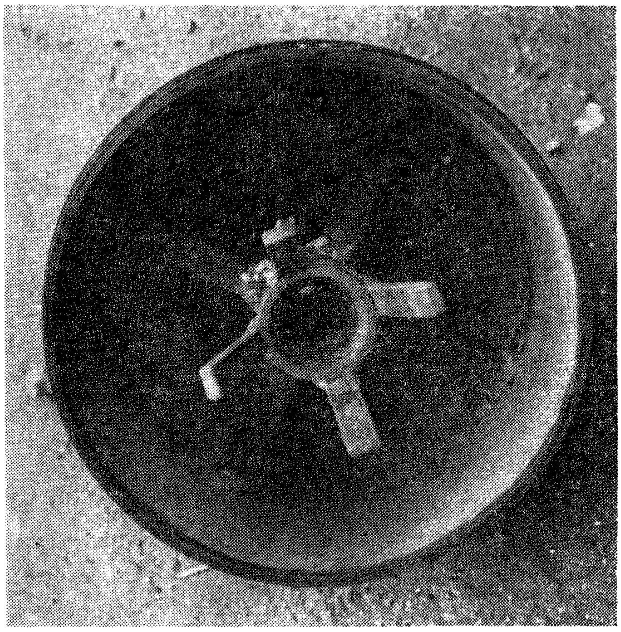


Photo. 1. Core catcher built-in at the inside of casing head

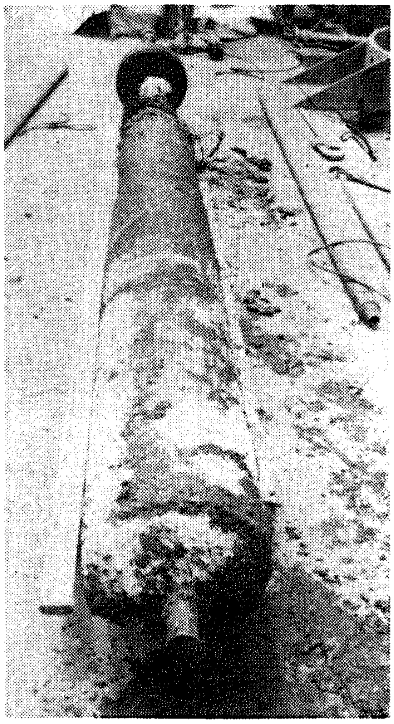


Photo. 2. The frozen soil column obtained at site *B*

the rapid rise of the pressure of the drilling mud, because if the joining is complete no more drilling mud can be circulated.

The frozen soil column inside the steel casing (Photo. 2) was pulled out by a crane; this took about 20 minutes. The procedures of cutting the frozen soil column into blocks of convenient sizes and then packing and transporting the materials to the laboratory were the same as those used at site *A*.

The same method was used successfully to obtain undisturbed Shirasu samples from a depth of 23.3 m to 25.7 m below the ground surface at another site.

LABORATORY TESTS TO EVALUATE SAMPLE DISTURBANCE

Based on their own studies, Yoshimi et al (1977, 1978, 1984) and Singh et al (1982) have shown that high-quality undisturbed samples of saturated sands can be obtained by in situ freezing if the sands are frozen without impeding drainage at the freezing front while an adequate confining pressure is maintained. Thus, if a sample is obtained

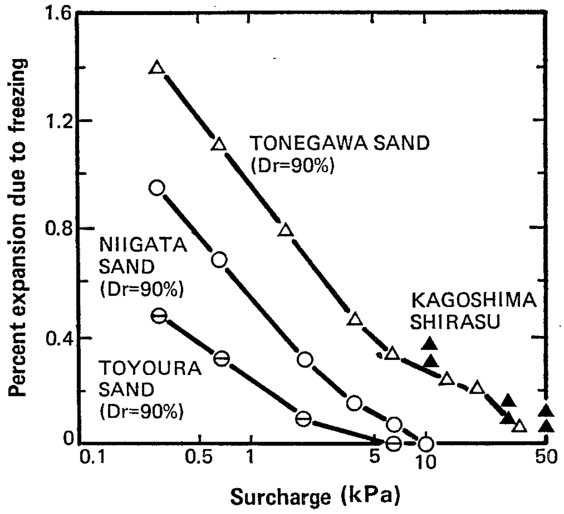


Fig. 5. Effect of surcharge on expansion of soils during unidirectional freezing

Depth 5.4m Site A

+10	+5	+7	−4	−4	+3	−2	+1	+6	6-1-C
+10	+3	+5	+4	+4	−3	−5	+2	−2	6-2-CD

6.0m (a)

← 75 mm →

Depth 8.4m Site B

freezing tube

+1	−9	−0	−1	+1				} AFU-8
+5	+4	+4	+2	−5	−0			
+40	+15	+13	+4	0	−16			
−13	+8	−17	+18	+1	+3	−4		
+9	+13	−1	+10	+3	−12			
+12	+5	0	+0	+0	−3	+2		} AFU-9
+14	+7	+5	+0	−5	0			
+3	+10	+6	−3	+2	−0	−5		
+5	+1	−2	−0	+1	+1	+1	−0	AFU-10

10.5m (b)

← 40~50 mm →

Fig. 6. Deviation of dry density of frozen samples

far enough away from the freezing pipe, it may be considered to be of high quality.

Unidirectional Freezing Test

A series of unidirectional freezing tests similar to those by Yoshimi et al (1978, 1984) were conducted to see if effective overburden pressure was high enough to prevent the volume expansion due to freezing. Fig. 5 show the present test results on Shirasu sample from the surface soil layer at site *A* compared with those reported by Yoshimi et al (1984) on other kinds of sand. Fig. 5 shows that the surcharge required to prevent expansion due to freezing is about 50 kPa, which is equivalent to the overburden pressure at a depth of about 4 m at the test site when the ground water table is at a depth of about 2.7 m.

Density Distribution

The radial distribution of dry density, ρ_d, was determined by the same method used by Yoshimi et al (1977, 1984) to assess the zone of possible disturbance of the frozen soil column near the freezing pipe.

Based on their studies in both laboratory tests and field tests, Yoshimi et al (1977, 1978, 1984) and Makihara et al (1981) have pointed out that the zone of possible disturbance due to drilling a hole for inserting the freezing pipe is limited to an area within one diameter of the freezing pipe from the pipe surface. The percentage deviation of dry densities from the average value, $\bar{\rho}_d$, at each depth is shown in Fig. 6, i. e.,

$$\frac{\rho_d-\bar{\rho}_d}{\bar{\rho}_d}\times 100\% \qquad (1)$$

in which $\bar{\rho}_d$ shows the average dry density of the recovered frozen sand outside the heavy lines (which are located about one diameter of the freezing pipes used at site *A* and site *B*, respectively, away from the surface of freezing pipe in the horizontal direction) at each depth. Unlike the results shown in the previous studies by Yoshimi et al (1977, 1978, 1984), it was not possible to clearly distinguish between the disturbed and the undisturbed part only by judging from the variation of dry density as shown in Fig. 6 for both sites. Relatively large deviations of dry density even in the supposedly undisturbed zone can be attributed to the fact that among the various specimens the content of pumice and gravel fragments is non-uniform.

Based on the studies by Yoshimi et al (1977, 1978, 1984) and Makihara et al (1981), it was decided, in the present study, to use only the part of the sample at least 7.5 cm or 5 cm (about one diameter of the freezing pipes used at sites *A* and *B*, respectively) away from the freezing pipe.

PHYSICAL PROPERTIES OF THE SHIRASU SAMPLES TESTED

Fig. 7 and Table 1 show the physical properties of the Shirasu soil for which undrained cyclic test results are reported in this paper. Judging from the curves of grain size distribution, the Shirasu soil may reasonably be classified as sand or silty sand as a whole. Fairly large differences in grain size distribution can be seen for samples at site *B*, even though they were obtained from a soil layer only two meters or less in thickness. These facts are considered to reflect the fairly large deviations of dry density in the horizontal direction even in the undisturbed zone of the frozen sample, as mentioned previously. As shown in Table 1, the Shirasu deposits have a lower value of G_s and an extremely low value of ρ_d compared with the ordinary sands ; this is essentially consistent with the results reported by Haruyama (1973).

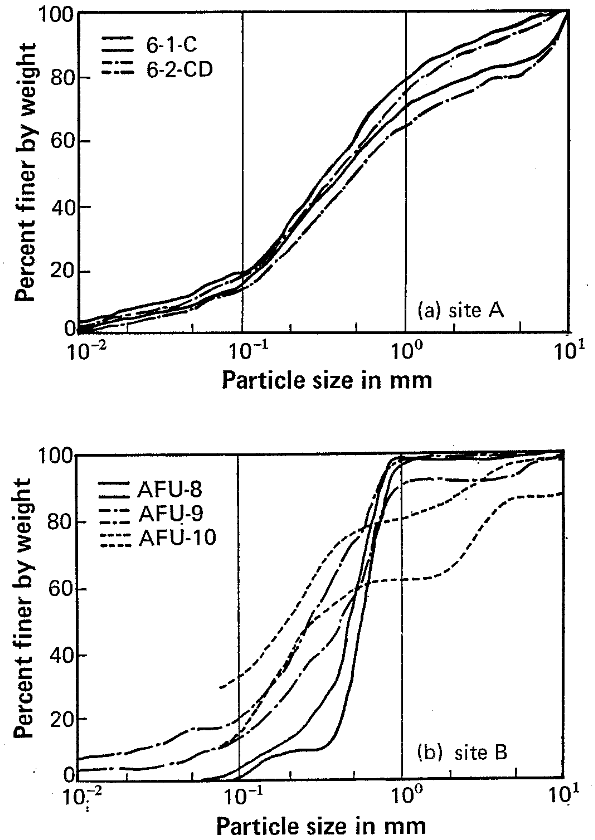


Fig. 7. Grain size distribution of the undisturbed samples used for undrained cyclic triaxial tests

TEST APPARATUS AND PREPARATION OF SHIRASU SPECIMENS

Triaxial Apparatus and Test Procedure

All of the undrained cyclic tests were conducted using triaxial test apparatus for which the static and cyclic loads were applied pneumatically. The load cell (100 kgf capacity) and gap sensors (Bison soil strain gauges) are placed inside the cell to determine the stress-strain relationship reliably at low strain levels. Also the LVDT used for measuring large strain near the stage of liquefaction is installed outside the cell.

All of the undrained cyclic tests were performed by applying uniform sinusoidal cycles of deviator stresses at a frequency of 0.5 Hz. The so-called "stress-down" of cyclic deviator stress was found in the present study, and the shear strength is corrected

Table 1. Physical properties of Shirasu tested

	Site *A*		Site *B*		
	No. 6-1-C	No. 6-2-CD	No. AFU-8	No. AFU-9	No. AFU-10
Depth (m)	5.4—5.7	5.7—6.0	8.4—8.7	9.3—9.6	10.2—10.5
G_s	2.43—2.45	2.41—2.50	2.49—2.54	2.42—2.58	2.43—2.46
D_{50} (mm)	0.38—0.51	0.34—0.40	0.49—0.56	0.28—0.43	0.2—0.3
D_{10} (mm)	0.05—0.07	0.04—0.056	0.16—0.21	0.024—0.065	——
U_c	11.6—13.8	8.9—16.3	2.91—3.84	8.2—15.3	——
Fines (%)	11.1—14.4	12.8—15.9	0.4—2.1	11.1—17.4	10.7—26.2
$\rho_{d\max}$ (g/cm³)	1.21—1.27	1.24—1.29	1.15—1.28	0.96—1.08	0.98—1.03
$\rho_{d\min}$ (g/cm³)	0.88—0.92	0.90—0.99	0.87—1.00	0.70—0.79	0.70—0.74
D_r (%)	54—72	31—79	56—77	69—80	55—84

as an average value of shear strees applied at the first cycle and at the cycle where the samples liquefied.

The electrical output from the load cell, displacement transducer, and pore pressure transducer was amplified and then recorded simultaneously in a magnetic disk of a System-7 IBM computer.

Preparation of Undisturbed Specimens from Frozen Samples

Cylindrical specimens of Shirasu soil about 5 cm in diameter were obtained from the frozen blocks in the undisturbed zone and were cut to approximately the desired length with a cutting tube 52 mm in diameter and 1 mm in wall thickness with hard metal teeth by the use of a hand-feed boring machine, as shown in Photo. 3. The specimens were then cut to the final length of about 125 mm by a power saw fitted with diamond teeth.

After the frozen specimen was set in the triaxial cell, covered with membrane and sealed with O-rings, the cell was filled with water to a level somewhat higher than the top of the specimen cap. Then the all round pressure of about 20 kPa or less was applied to the specimen and the specimen was allowed to thaw at room temperature under drained conditions.

Although consolidation pressures ranging from 5 to 98 kPa during thawing of a dense sand from Niigata (which had an average SPT N-value of about 30) had no significant effect on the undrained shear strength, as reported by Yoshimi et al (1984), no data have yet been presented to confirm this for alluvial Shirasu deposits with an SPT N-value of about 10. A relatively low confining pressure of about 20 kPa was adopted in the present study. After the specimen thawed completely after about 12 hours, it was saturated by letting de-aired water flow upwards under a small water head of about 30 cm for about three hours, with an applied back pressure of 300 kPa. The pore pressure coefficient B-value was 0.95 or more for all specimens.

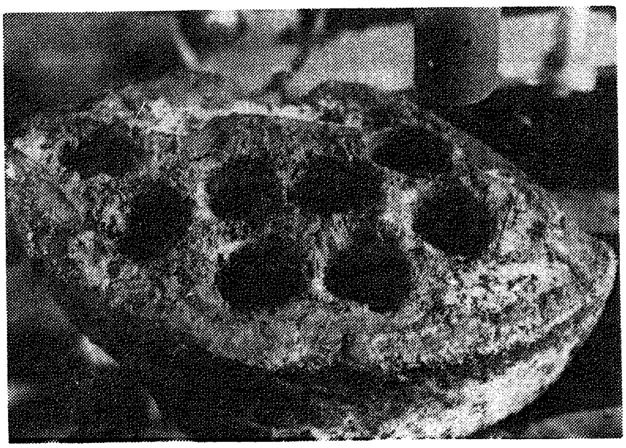


Photo. 3. Obtaining a cylindrical frozen sample from a frozen block by hand-feed coring machine

After that, the specimen was consolidated for about three hours before undergoing the cyclic test under an isotropic stress about 1.1 times the effective vertical stress of the sample in situ. Based on the findings by Ishihara et al (1973) that for soils containing some fines, over-consolidation of the sample will increase the undrained cyclic shear strength, and also on the findings by Yoshimi et al (1984) that the undrained shear strength of the undisturbed sample is markedly influenced by the consolidation pressure particularly where the consolidation pressure, σ_c, is lower than the effective overburden stress in situ, the authers adopted for this study a confining pressure value about 1.1 times the effective vertical stress of the sample in situ, to avoid overestimating the undrained shear strength of samples in situ.

Preparation of Reconstituted Specimens

Reconstituted specimens were prepared from the remaining portion of the frozen sample in order to study the effect of sample disturbance on the undrained cyclic strength. The remainder of the frozen sample (shown in Photo. 3) was thawed and then broken down structurally by fingering, with extreme care taken not to crush the soil particles. The saturated disturbed Shirasu was poured into a water-filled specimen mold in about 10 layers and compacted by striking the mold with a wooden bar 30 mm in diameter to reproduce the density of the undisturbed samples in situ.

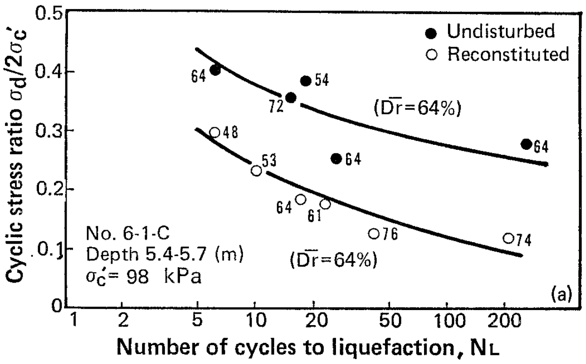
Determination of the maximum and minimum dry densities was done by the JSSMFE Standard Method of Test for the Maximum and Minimum Densities of sand, JSF standard T 26-81 T (JSSMFE, 1979). Because many of the specimens contained some pumice particles which were too large in size for the container (60 mm in inside diameter) which was used for measuring the limiting densities, for all specimens only the components under 5 mm were used for determining the maximum and minimum densities.

For the soils obtained at site *B*, it is very difficult to reconstitute specimens which are insufficiently good shape for testing and which have the same relative density as the samples in situ. As a result, the reconstituted specimens from blocks Nos. AFU-8 and AFU-9 have a relative density about 15 percentage points higher than that in situ. It was not possible to prepare a satisfactory reconstituted specimen from block No. AFU-10.

The methods of saturation and consolidation were carried out in the same way as for the undisturbed specimens described in the preceding section, except for the process of thawing the frozen specimen.

UNDRAINED CYCLIC SHEAR STRENGTH OF THE ALLUVIAL SHIRASU

The results of undrained cyclic triaxial tests, both on samples obtained by the in situ freezing method and on reconstituted sam-



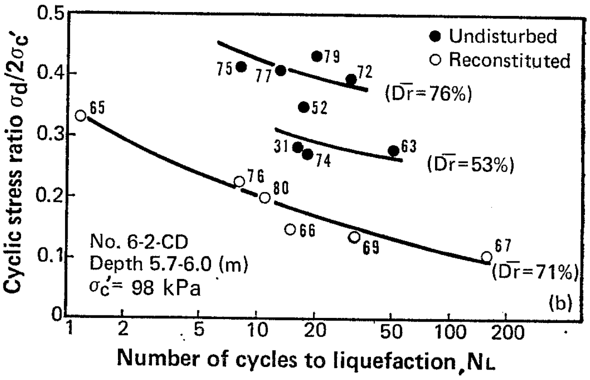
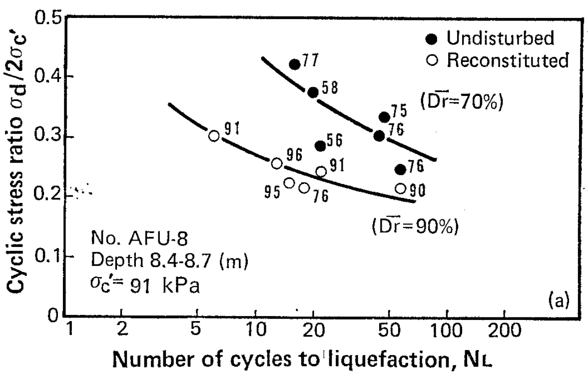
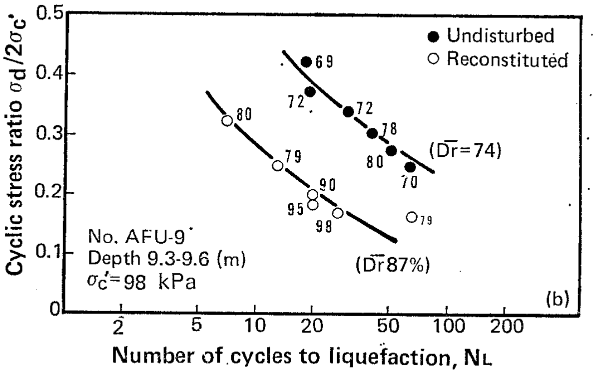


Fig. 8. **Shear stress ratio required to cause zero effective stress for samples from site *A***

ples, are presented in this section.

Figs. 8 and 9 show the relationship between the shear stress ratio and the number of cycles to cause liquefaction (defined as a stage where the excess pore water pressure equals the initial effective stress) both for undisturbed samples and for reconstituted samples obtained at site *A* (Fig. 8) and site *B*(Fig. 9). The solid and hollow circles show the results of undisturbed sample and reconstituted samples, respectively. The numbers





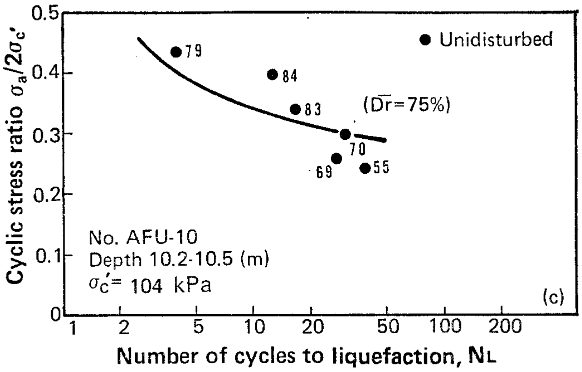


Fig. 9. Shear Stress ratio required to cause zero effective stress for samples from site *B*

near the circles show the relative density (D_r) of each sample. $\bar{D}_r$ is the average relative density of the samples tested. From Figs. 8 and 9, the following can be pointed out :

(1) For the samples from site A, the cyclic stress ratio required to cause liquefaction in 20 cycles on undisturbed samples, R_{20}, is about 1.7 to 2.2 times that required for reconstituted samples, although both of them have about the same relative density.

(2) For the samples from site B, the cyclic stress ratio required to cause liquefaction, R_{20}, on undisturbed samples is about 1.5 to 1.8 times that required for reconstituted samples, even though the former has a relative density about 13 to 20 percentage points lower than the latter.

(3) A fairly large deviation of relative density for undisturbed samples can be seen, which is considered due to the fact that pumice particles larger than 5 mm in diameter are not uniformy contained in all of the samples ; this deviation is consistent with the results which indicate a large deviation of dry density even in the undisturbed zone of frozen samples, as described previously.

Fig. 10 shows the relationship between the shear stress ratio and the number of cycles required to reach double amplitude axial strains of 1, 5.0, and 7.5%, both for undisturbed samples (solid symbols) and for reconstituted samples (hollow symbols) from site A. Fig. 11 shows these relationships for the samples from site B. From Figs. 10

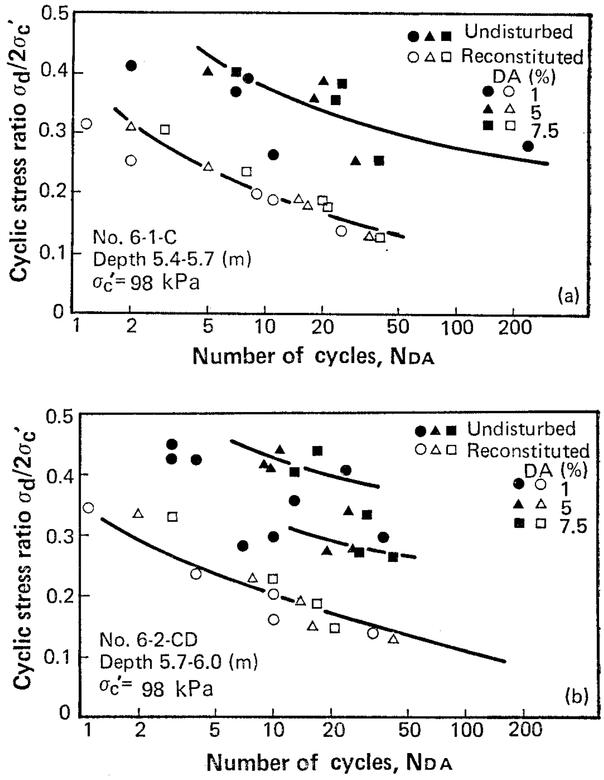


Fig. 10. Shear stress ratio required to cause double amplitude axial strains of 1, 5 and 7.5% for samples from site *A*

and 11, the following can be shown.

(1) For undisturbed samples obtained from both site A and site B, the cyclic stress ratio R_{DA20} required to cause double amplitude axial strains of 5.0% in 20 cycles was almost the same as R_{20}, defined in terms of zero effective stress. The ratios of cyclic stress ratio R_{DA20} between undisturbed samples and reconstituted samples were 2.0 to 2.2 and 1.5 to 1.6 for the samples from sites A and B, respectively ; these ratios are similar to those based on R_{20}.

(2) The number of cycles required to increase the double amplitude of axial strain from 1% to 5% for undisturbed samples was more than that required for reconstituted samples ; in other words, undisturbed samples are much more ductile than reconstituted samples.

It is interesting to note that, although some sample disturbance was expected at site A due to the vibration of the casing (as was discussed in the stage of planning the field

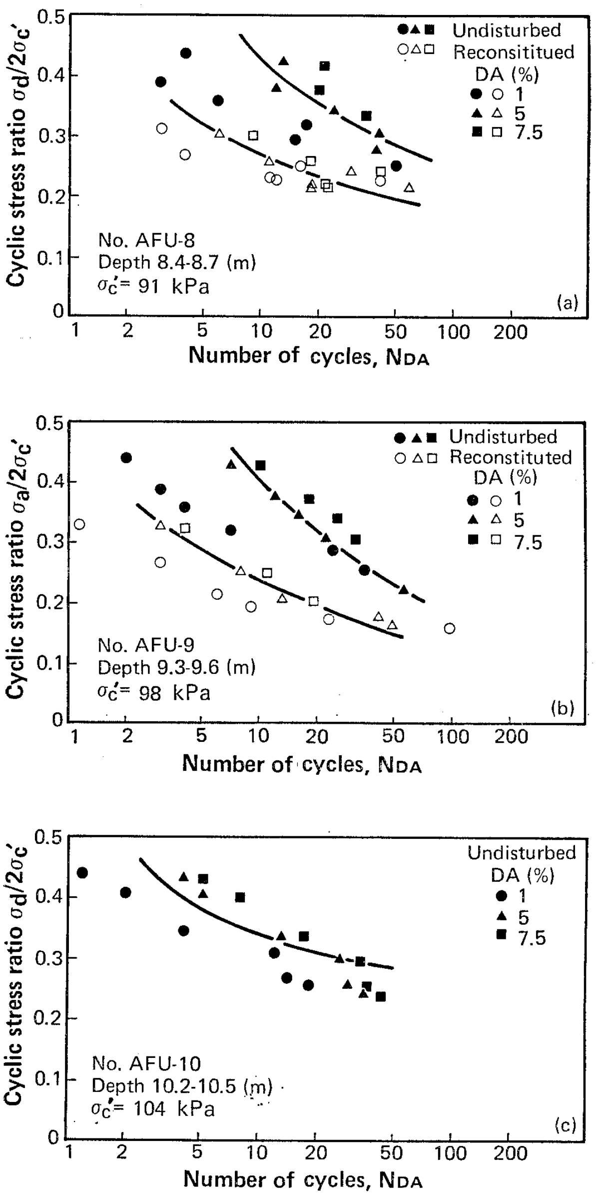


Fig. 11. Shear stress ratio required to cause double amplitude axial strains of 1,5 and 7.5% for samples from site *B*

test), no clear evidence of sample disturbance (such as a cracking) was found either in the field or in the laboratory, for undisturbed samples the liqueraction resistance was much higher than that of reconstituted samples.

COMPARISON OF UNDRAINED CYCLIC TRIAXIAL STRENGTH WITH THAT EVALUATED FROM THREE SIMPLIFIED PROCEDURES

In this section the results of the undrained cyclic triaxial tests on undisturbed samples described in the preceding section are compared with estimated values based on N-value, mean particle size and fines content of soils from three simplified procedures proposed by Seed (1979), Tatsuoka et al (1978), and Tokimatsu and Yoshimi (1983).

The comparison will be made for the following set of parameters : earthquake magnitude=8.0, maximum horizontal acceleration at ground surface=150 gal, and number of equivalent stress cycles=20.

In comparison, the in situ undrained cyclic strength considered in the method proposed by Seed (1979), Tatsuoka et al (1978) and Tokimatsu and Yoshimi (1983) is expressed by Eqs. (2) through (4), respectively, based on the previous studies on reconstituted sand by De Alba et al (1976).

$$\frac{\tau_l}{\sigma_c'}=0.57\left(\frac{\sigma_d}{2\sigma_c'}\right)\text{triaxial}\ (\Delta u=\sigma_c')=0.57\times R_{20} \qquad (2)$$

$$\frac{\tau_l}{\sigma_c'}=\left(\frac{\sigma_d}{2\sigma_c'}\right)\text{triaxial}\ (DA=5\%)=R_{DA20} \qquad (3)$$

$$\frac{\tau_l}{\sigma_c'}=0.57\left(\frac{\sigma_d}{2\sigma_c'}\right)\text{triaxial}\ (DA=5\%)=0.57\times R_{DA20} \qquad (4)$$

The results of comparison are shown in Fig. 12 and Fig. 13 for site A and site B, respectively. Although similar trends in SPT results were found in all locations, to avoid complications in the figures only the N-values of SPT performed at the location nearest the place for freezing sampling at each site are used in the comparison.

In Figs. 12 and 13, the hollow symbols show the values of factor of safety against liquefaction, F_l, as calculated by the three simplified procedures based on N-value and soil gradation. For the solid symbols, in calculating the value of F_l, the undrained cyclic strength of soils was not estimated from N-value and soil gradation but calculated from the measured strength of undisturbed Shirasu, as shown in Eqs. (2) through (4).

From Figs. 12 and 13 the following can be

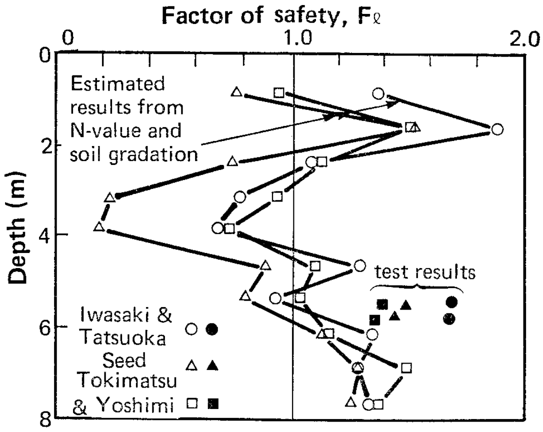


Fig. 12. Undrained cyclic strength of undisturbed samples from site *A* compared with that evaluated from simplified procedures

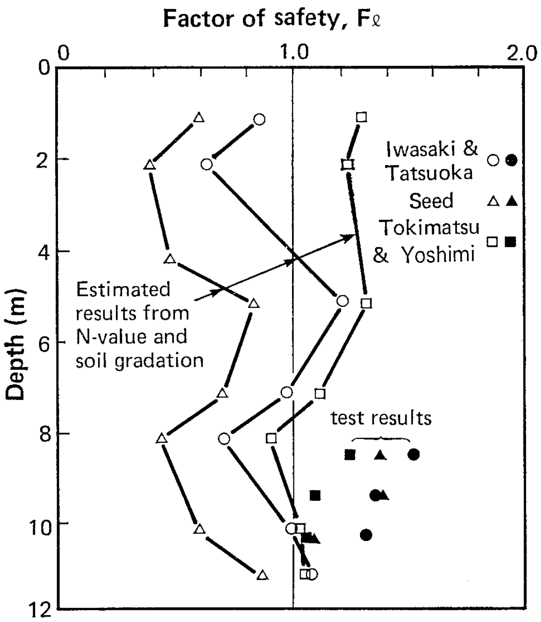


Fig. 13. Undrained cyclic strength of undisturbed samples from site *B* compared with that evaluated from simplified procedures

shown :

(1) As a whole, the in situ undrained cyclic strength evaluated from any of the above three simplified procedures is lower than the cyclic triaxial test results on the samples obtained by in situ freezing. This suggests the high possibility of underestimating the in situ undrained cyclic strength of alluvial Shirasu from N-values and soil gradations by the use of the existing sim-

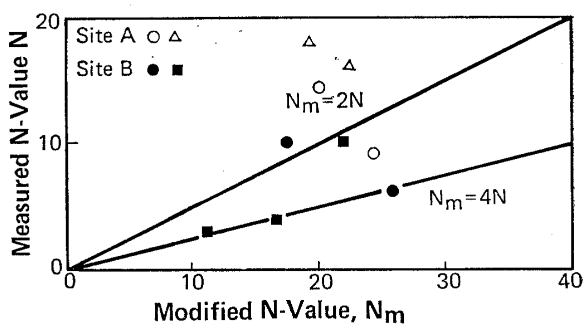


Fig. 14. Relationship between modified *N*-value, N_m, and measured *N*-value, *N*

plified procedures.

(2) Among the three simplified procedures, the method proposed by Tokimatsu and Yoshimi (1983) gave the highest value of factor of safety against liquefaction.

Fig. 14 shows the relationship between the N-value of SPT and the value of N_m, where N_m is a value calculated by substituting the undrained cyclic triaxial strength of undisturbed samples into Eq. (5), as presented by Tatsuoka et al (1978) in their simplified procedure.

$$N_m=\left(\frac{R_{DA20}-0.225\log_{10}\dfrac{0.35}{D_{50}}}{0.0882}\right)^2\times(\sigma_v'+0.7)$$

$$(0.04\text{ mm}\leqq D_{50}\leqq 0.6\text{ mm}) \qquad (5)$$

$$N_m=\left(\frac{R_{DA20}+0.05}{0.0882}\right)^2\times(\sigma_v'+0.7)$$

$$(0.6\text{ mm}<D_{50}\leqq 1.5\text{ mm}) \qquad (6)$$

Where D_{50} is the mean particle size of soil grains in mm and σ_v' is the effective vertical stress in kgf/cm^2 (1 kgf/cm^2=98 kPa). Four test results of SPT by the trip-monkey method (borings No. NT-2 and NT-3 at site *A* and borings No. NT-1 and NT-2 at site *B*) were used in comparison. As shown in Fig. 14, generally speaking the modified N-value N_m is larger than the measured N-value N, and when the measured N-value N is lower than about 10, the data are distri buted almost entirely in the area enclosed by the two lines of $N_m=2N$ and $N_m=4N$.

It may be useful, for practical purposes, to extend the applicability of the existing method to some sandy soils with special geo-

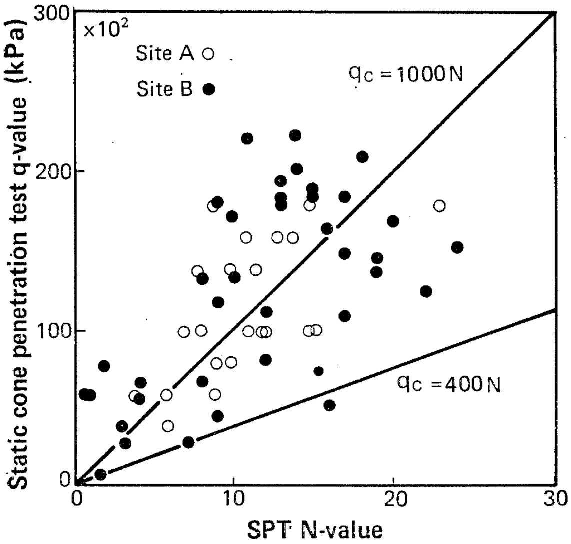


Fig. 15. Relationship between N-value of SPT and q_c value of static cone penetration test obtained from site A and site B

logic conditions by modifying the N-value. The use of N_m described above may be an example of one such way to do so.

Fig. 15 shows the relation between the N-value of SPT and the q_c value in kPa of the static cone penetration test. The hollow circles show the results at site A and the solid circles show the results at site B. Although the data are scattered considerably, on the average, for Shirasu a relation of $q_c=1000\,N$ can be seen, rather than the relation of $q_c=400\,N$ which is typical for ordinary sands. It is of interest to note that the relation between $q_c=1000\,N$ and $q_c=400\,N$ almost corresponds to the previously described results of $N_m=2\,N$ to $4\,N$.

CONCLUSIONS

The conclusions, which are based on the undrained cyclic triaxial tests on two undisturbed samples of alluvial Shirasu soil from Kagoshima, Japan, obtained by in situ freezing, as well as on reconstituted samples with densities nearly equal to or larger than those of undisturbed samples are summarized as follows :

(1) Two new developments in the technique of sampling by in situ freezing were successfully achieved in the field tests.

The method performed at site A by vibrating a steel casing will shorten the time required for obtaining the frozen sand column ; Because of this the cost of obtaining frozen soil columns can be reduced. However, due to the noise and the problem of ground vibration, this method should be limited to field situations in which the average N-value is lower than about 15 and the location is sufficiently far away from residential areas.

Another method conducted at site B, termed "partial freeze core sampling," offers an important advantage in the technique of obtaining an undisturbed samples of a sand layer with a certain thickness from a depth of about 30 m below the ground surface, although this paper presents the results of field tests for obtaining undisturbed samples only from a depth of about 10 m.

(2) Compared with the liquefaction resistance of the undisturbed samples obtained by the two methods of in situ freezing described above, that of the reconstituted samples from site A, which had about the same density, was about 45 to 50% of the value for the undisturbed sample from this site, and that of the reconstituted samples from site B, with relative density which was 13 to 20 percentage points higher than that of undisturbed samples, was about 62 to 67% of the value for the undisturbed sample from site B ; these conclusions assume that the strength is defined as the shear stress ratio required to cause a double amplitude axial strain of 5% in 20 stress cycles.

(3) The two alluvial Shirasu soils studied in this paper were found to have higher undrained cyclic strength than that which was estimated by the three simplified procedures based on N-value and soil gradation. As far as the authors know, however, specific data on such volcanic soils as Shirasu were not used to any great extent in the process of establishing these three simplified procedures. Among three simplified procedures, the results obtained by the method proposed by Tokimatsu and Yoshimi (1983)

are nearest to the test results on the undisturbed samples.

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REFERECES

1) De Alba, P., Seed, H. B. and Chen, C. K. (1976) : "Sand liquefaction in large-scale simple shear test," Jour. of the Geotechnical Engineering Division, ASCE, Vol. 102, No. GT 9, pp. 909-927.
2) Haruyama, M. (1973) : "Geological, physical, and mechanical properties of 'Shirasu' and its engineering classification," Soils and Foundations, Vol. 13, No. 3, pp. 45-60.
3) Ishihara, K. and Sodekawa, M. (1973) : "Liquefaction characteristics of saturated sand with fines content," Proc. of 8th Annual Meeting of JSSMFE, pp. 323-326.
4) JSSMFE (1979) : Soil Testing Manual, 2nd Revised Edition, pp. 172-188.
5) Makihara, Y., Oh-hara, J., Hatanaka, M. and Suzuki, Y. (1981) : "Undisturbed sampling of saturated sand by in situ freezing," Proc. of Symposium on Sampling, JSSMFE, pp. 65-70 (in Japanese).
6) Oh-hara, S., Yasunaga, F. and Fujii, N. (1974) : "Dynamic properties of undisturbed Shirasu," Jour. of JSSMFE, Vol. 14, No. 4, pp. 107-114 (in Japanese).
7) Seed, H. B. (1979) : "Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes," J. GED, ASCE, Vol. 105, No. GT 2, pp. 201-255.
8) Singh, S., Seed, H. B. and Chan, C. K. (1982) : "Undisturbed sampling of saturated sands by freezing," J. GED, ASCE, Vol. 108, No. GT 2, pp. 247-264.
9) Tatsuoka, F., Iwasaki, T., Tokida, K., Yasuda, S., Hirose, M., Imai, T. and Kon-no, M. (1978) : "A method for estimating undrained cyclic strength of sandy soils using standard penetration resistances," Soils and Foundations, Vol. 18, No. 3, pp. 43-58.
10) Tokimatsu, K. and Yoshimi, Y. (1983) : "Empirical correlation of soil liquefaction based on SPT *N*-value and fines content," Soils and Foundations, Vol. 23, No. 4, pp. 56-74.
11) Yamanouchi, T., Koreeda, K. and Sakaguchi, O. (1976) : "Alluvial Shirasu deposits in Kagoshima-its geotechnical properties," Tsuchi-to-Kiso, JSSMFE, Vol. 24, No. 7, pp. 25-32 (in Japanese).
12) Yoshimi, Y., Hatanaka, M. and Oh-oka, H. (1977) : "A simple method for undisturbed sampling by freezing," Proc. of Specialty Session 2 on Soil Sampling, 9th Int. Conf. on Soil Mech. and Found. Eng. Tokyo, pp. 23-28.
13) Yoshimi, Y., Hatanaka, M. and Oh-oka, H. (1978) : "Undisturbed sampling of saturated sands by freezing," Soils and Foundations, Vol. 18, No. 3, pp. 59-73.
14) Yoshimi, Y., Tokimatsu, K., Kaneko, O. and Makihara, Y. (1984) : "Undrained cyclic shear strength of a dense Niigata sand," Soils and Foundations, Vol. 24, No. 4, pp. 131-145.