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LIQUEFACTION RESISTANCE OF SAND DEPOSIT IMPROVED WITH SAND COMPACTION PILES

MITSU OKAMURA[i], MASANORI ISHIHARA[ii] and TAKESHI OSHITA[iii]

ABSTRACT

This paper reports results of in-situ tests and undrained cyclic shear tests on high-quality undisturbed samples obtained by the in-situ freezing method at three sites where foundation soils had been improved with sand compaction piles. Triaxial specimens obtained from the frozen samples were fully saturated in the triaxial cell and then subjected to cyclic loadings.

In the current design practice of ground improvement with the sand compaction piles, the SPT *N*-value at a mid point of a rectangular area surrounded by four adjacent sand piles, at which the *N*-value, and thus the liquefaction resistance is considered to be the smallest, is set as a target for the degree of compaction. It was found, however, that distributions of *N*-values and N_d-values in a horizontal plane at a certain depth appeared to be rather random. This suggests that the *N*-value at the center of a rectangular area does not always provide a conservative evaluation.

The relationship between liquefaction resistances and the *N*-value, which was developed based mainly on field evidences of earthquake-induced liquefaction of natural soil deposits and reclaimed lands, compared quite well with that obtained from tests on fully saturated high quality specimens and mean values of N_d at several locations in the improved sands. Degree of saturation of the frozen sample was revealed to be considerably low, in the range between about 70 and 90%. This fact indicates that the liquefaction resistances of improved sands are significantly higher than those obtained from the relationship which is available only for fully saturated soils.

Key words: cyclic shear, in-situ test, liquefaction, sand compaction pile (**IGC:** C3/D7)

INTRODUCTION

Sand compaction piles (SCP) have extensively been used to ameliorate liquefaction resistances of loose sand deposits since 1970's. The principle of this ground improvement technique is densification of foundation soils by installing sand piles in the ground. Increases in soil density as well as lateral effective stresses are considered to enhance liquefaction resistances of foundation soils.

Previously vibratory hammers were commonly utilized to construct sand compaction piles; however, there is an increasing trend of using hydraulic jacks in recent years due to the restriction of noise and ground vibration at construction sites in urban areas (Nozu et al., 1998). The former and later construction techniques may be termed as vibratory SCP and non-vibratory SCP, respectively.

In the current practice for designing SCP as a countermeasure against liquefaction, replacement ratios are often determined using an empirical chart so that SPT *N*-values at the midpoint of sand piles are higher than a target value. The empirical chart provides a relationship between replacement ratios and SPT-*N* values at midpoints of sand piles, which was established by a large number of observed *N*-values in improved grounds (Mizuno et al., 1987; Japanese Geotechnical Society, 1998). This design method is based on two major assumptions including: 1) *N*-value at a depth in improved ground and sand piles are higher than those at the midpoint of sand piles; 2) The relationship between liquefaction resistances and *N*-value (e.g. Japan Road Association, 1996), which was developed based mainly on the field evidences of earthquake-induced liquefaction of sands in natural deposits and reclaimed lands, can be applicable for estimating liquefaction resistance of soils improved with SCP.

Despite a large number of SCP that has been applied in practice, both assumptions have not yet been well examined. In particular, with regard to the assumption 2), in-situ and laboratory tests conducted by Tokimatsu et al. (1990) are the only data available in the literature. They conducted a series of cyclic shear tests on samples taken from ground improved with the vibratory SCP technique by using the in-situ freezing method. The

[i] Senior Research Engineer, Ground Vibration Team, Public Works Research Institute, 1-6 Minamihara, Tsukuba, Ibaraki Prefecture 305-8516, Japan (okamura@pwri.go.jp)
[ii] Research Engineer, ditto.
[iii] Team Leader, Construction Technology Research Team, ditto.
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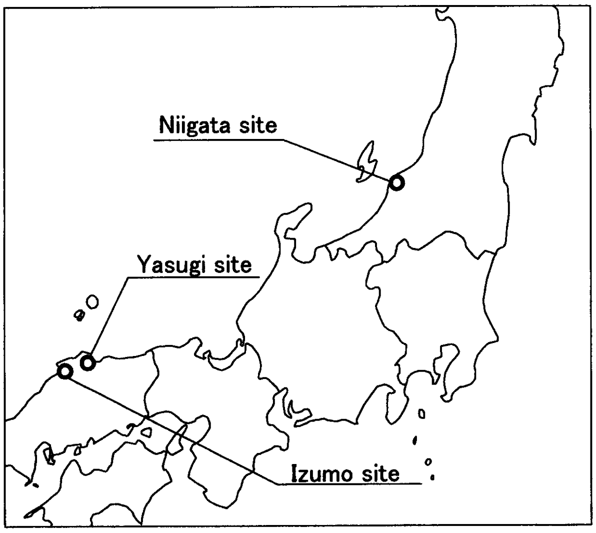


Fig. 1. Locations of sampling and in-situ test site

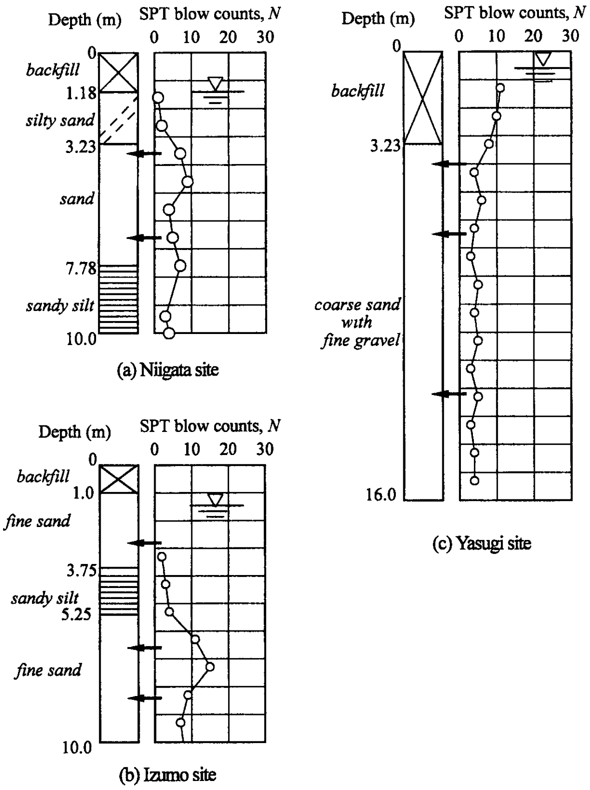


Fig. 2. Soil profile in the three sites before ground improvement (arrows indicate approximate depths where triaxial specimens were obtained)

relationship between the liquefaction resistance from the laboratory tests and N-value was reported to agree well with that of the conventional method used in practice. Accumulation of data is apparently needed to verify the validity of the design procedure and to understand the mechanical properties of improved grounds.

This paper describes in-situ tests conducted at three sites where foundation soils were improved with the vibratory or the non-vibratory SCP techniques. Spatial distributions of N-value and N_d-value obtained from the rotary ram sounding are discussed. In addition, high-quality undisturbed samples were obtained at each site by the in-situ freezing method and cyclic triaxial tests were carried out. Test results were used to verify the applicability of the conventional method for assessing liquefaction resistances of soils improved with SCP.

TEST SITE AND SOIL IMPROVEMENT

Niigata Site

Niigata site was located on the flood channel of Shinano river in Niigata Prefecture, about 13.5 km from the river mouth as illustrated in Fig. 1. Construction of a landing quay for emergency supply ships had been planned in this site. Figure 2(a) indicates the soil profile at the site together with the SPT N-value obtained in June 2001, just before the SCP installation work begun. The elevation of the ground surface was approximately 2.1 m above the mean Tokyo Bay sea level (T.P. +2.1 m) and the ground water table was 1.2 m below the ground surface. Except for the fine sand fill and the silty sand layer which extended down to 3.2 m from the ground surface, the foundation soil mostly consisted of a dark gray clean sand to a depth of 7.8 m. The grain size distributions and physical properties of the sand at depths of about 3.3 m and 6.7 m, from where specimens of triaxial tests were taken, are depicted in Fig. 3(a) and Table 1. The sands at the two depths were medium to fine sand with fines content less than 5%.

The sand below the water table to the depth of 7.8 m was judged liquefiable and was expected to cause flow deformation towards river when the sand liquefied. In order to reduce the deformation of the sand during an earthquake, the sand layer was improved with SCP in the area 60 m long along the side of the river and 24 m wide in the transversal direction.

Figure 4 indicates schematically the procedure of the non-vibratory SCP technique (Nozu et al., 1998) used in the site. A casing pipe with a diameter of 0.4 m was penetrated by a hydraulic jack into the ground to a depth of 8 m from the ground surface and sand was thrown in the casing from the top. Then, the casing was withdrawn 0.5 m and the sand was discharged into the bored hole with an aid of pressurized air of the order of 500 kN/m^2 supplied from the top of the casing. The sand pile was compressed vertically to increase its diameter to about 0.7 m by penetrating the casing pipe 0.3 m. The withdrawing and re-penetrating procedure was repeated until a complete compacted sand pile was formed. It was observed during the sand pile construction that large amount of air which was exhausted with sand into the ground from the tip of the casing pipe continuously spouted from everywhere on the ground surface within an area of about several meters from the casing. This is common practice in construction of the sand compaction piles. It should be noted that the noise and the ground vibration due to the sand pile construction was quite weak in this site since the casing was penetrated with a

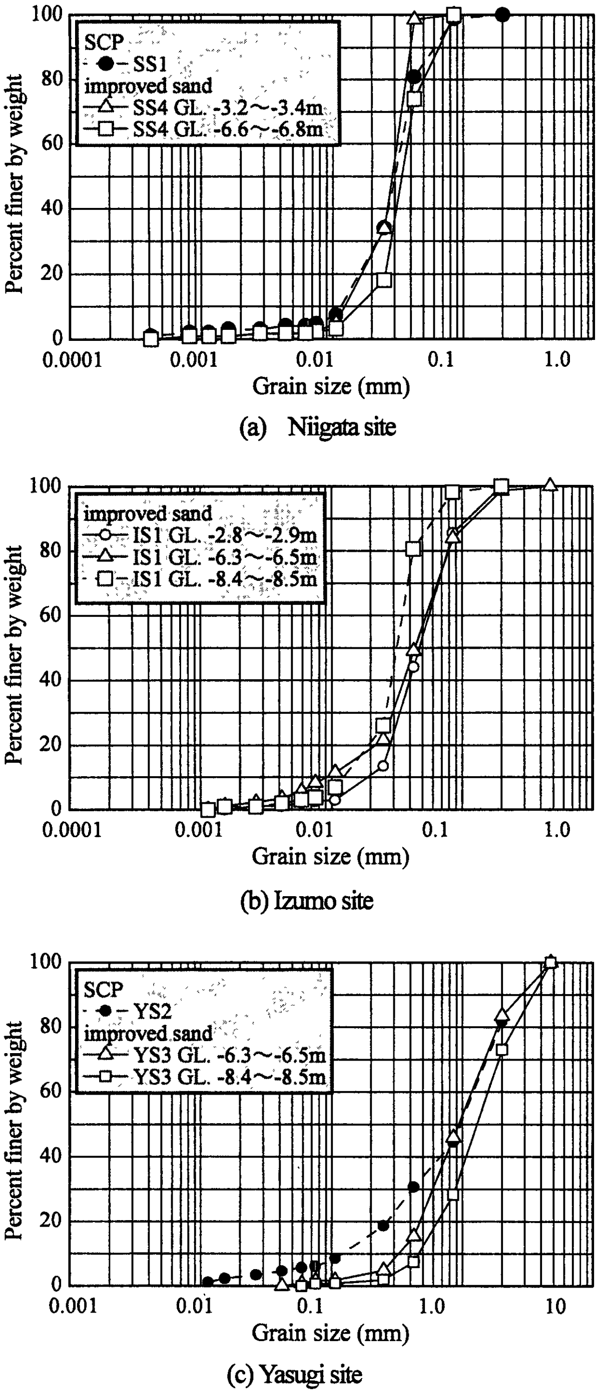


Fig. 3. Grain size distribution of sands in the three sites

hydraulic jack, instead of a vibro-hammer that used to be widely used.

The sand used to build the sand piles was dredged from Shinano river bed near the site. The grain size distribution of this sand was almost the same as the improved sands as shown in Fig. 3(a).

Figure 5(a) illustrates the plan view of the site, showing the arrangements of sand piles, in-situ test and sampling locations. From June to August 2001, a total of 850 sand piles with some 0.7 m in diameter were driven in a rectangular pattern at spacing of 1.2 m and 1.4 m, giving rise to a replacement ratio of 23%. About a month after the completion of the sand pile installation, high quality undisturbed samples were obtained by the in-situ freezing method (Yoshimi et al., 1978) and in-situ tests including the standard penetration tests (SPT) and the rotary ram sounding tests (RRS) were conducted in the vicinity of sampling locations, as depicted in the figure.

Izumo Site

The Izumo site was located near Kando river in Shimane Prefecture (Fig. 1), about 4.5 km from the river mouth and about 50 m from the river dike. In this site, construction of an abutment of a road bridge crossing the river was planned. Figure 2(b) indicates soil profile at the site together with the *N*-value obtained in August 1998, before the ground improvement work begun. The elevation of the ground surface was T.P. +3.3 m and the ground water table was 1.5 m below the ground surface. The foundation soil mostly consisted of fine sands and sandy silt to a depth of 10 m. At shallower depth in the fine sand layer, from 1.0 m to 2.1 m, pieces of wood and humic soils were found to be mixed. The grain size distributions and physical properties of the sand at depths of about 2.8 m, 6.3 m and 8.4 m, from where the triaxial specimens were taken, are depicted in Fig. 3(b) and Table 1. The soils at the three depths were medium clean sand with fines content less than 5%.

The sand below the water table to a depth of 10 m was judged liquefiable, therefore, the sand layer below the abutment was improved with SCP. The vibratory SCP technique was employed in this site since there were no houses in the vicinity of the site. The material of the sand piles was a medium clean sand with very small fines contents obtained from the river near the site.

Figure 5(b) illustrates the plan view of the site. On November and December 2001, a total of 150 sand compaction piles with 0.7 m in diameter were driven in a square pattern at a spacing of 2.2 m, giving rise to a replacement ratio of 8%. About a month after the completion of the ground improvement, high quality undisturbed samples were obtained by the in-situ freezing method and SPT and RRS were conducted at locations indicated in the figure.

Yasugi Site

During the Tottoriken-seibu earthquake of October 2000, some parts of dikes protecting the coastline of Nakaumi were damaged (Tokida et al., 2000). At the Yasugi site illustrated in Fig. 1, which is located some 50 m from the mouth of Iinashi river, a dike, about 2.1 m high, subsided as much as 1.2 m due to the liquefaction of foundation soils. Sand bags were placed on the dike after the earthquake to prevent an overflow as an emergent restoration work.

Figure 2(c) indicates soil profile and the *N*-value obtained in February 2001, after the earthquake. The elevation of the ground surface (on the base level of the dike before the earthquake) was approximately T.P. +0.8 m and the ground water table was 0.55 m below the

Table 1. Test conditions and physical properties of triaxial specimens

(a) Niigata site

Sample	Depth from ground surface (m)	Effective confining pressure in triaxial test (kPa)	Dry density (g/cm^3)	Relative density (%)	Minimum dry density (g/cm^3)	Maximum dry density (g/cm^3)	Specific gravity
SS1	3.6–3.8	49	1.455	56.4	1.273	1.635	2.665
	6.8–6.9	75	1.432	47.8	1.286	1.634	2.744
SS2	3.1–3.2	49	1.495	59.9	1.275	1.677	2.695
	6.6–6.8	75	1.579	77.4	1.291	1.688	2.669
SS3	3.6–3.8	49	1.466	59.7	1.291	1.622	2.765
	6.6–6.7	75	1.501	73.7	1.269	1.606	2.656
SS4	3.2–3.4	49	1.440	66.0	1.232	1.570	2.702
	6.6–6.8	75	1.440	79.4	1.206	1.516	2.645

(b) Izumo site

Sample	Depth from ground surface (m)	Effective confining pressure in triaxial test (kPa)	Dry density (g/cm^3)	Relative density (%)	Minimum dry density (g/cm^3)	Maximum dry density (g/cm^3)	Specific gravity
IS1	2.8–2.9	49	1.442	75.4	1.233	1.527	2.634
	6.8–7.0	74	1.440	81.8	1.179	1.514	2.637
	8.1–8.2	88	1.449	90.9	1.132	1.490	2.636
IS2	2.8–2.9	49	1.551	82.2	1.265	1.631	2.641
	6.3–6.5	74	1.424	81.5	1.167	1.502	2.651
	8.4–8.5	88	1.390	79.5	1.129	1.475	2.655

(c) Yasugi site

Sample	Depth from ground surface (m)	Effective confining pressure in triaxial test (kPa)	Dry density (g/cm^3)	Relative density (%)	Minimum dry density (g/cm^3)	Maximum dry density (g/cm^3)	Specific gravity
YS1	4.0–4.1	50	1.522	77.5	1.318	1.594	2.637
	6.6–6.7	70	1.491	73.0	1.280	1.587	2.630
YS2	3.8–4.0	50	1.723	80.1	1.377	1.823	2.629
	6.4–6.6	70	1.719	82.4	1.358	1.807	2.630
	12.1–12.4	120	1.542	50.0	1.329	1.649	2.635
YS3	4.0–4.1	50	1.545	87.8	1.267	1.594	2.620
	6.5–6.6	70	1.508	72.1	1.309	1.602	2.623
	12.0–12.1	120	1.542	82.6	1.295	1.588	2.624
YS4	6.5–6.6	70	1.561	73.6	1.316	1.672	2.629
	12.0–12.1	120	1.488	75.3	1.309	1.604	2.631

ground surface. Except for the fine sand fill deposited at the surface, the foundation soil mostly consisted of dark gray coarse sands with some inclusion of gravel to a depth of 16 m. The grain size distributions and physical properties of the sand at depths of about 6.4 m and 8.4 m are depicted in Fig. 3(c) and Table 1.

Figure 5(c) illustrates the plan view of the site. A total of 680 sand piles with some 0.7 m in diameter were driven by the vibratory SCP technique typically in a square pattern at spacing of 1.5 m from April to May, 2002. The replacement ratio in this site was 17%. The depth of SCP at locations of the sampling and the in-situ tests was 23 m. The sand obtained in the vicinity of the site was used as a sand pile material. The sand was a medium to coarse sand with fines contents less than 10% as indicated in Fig. 3(c). About a week after the completion of the ground improvement, high quality undisturbed samples were obtained by the in-situ freezing method and SPT and RRS were conducted at locations indicated in Fig. 5(c). On completion of the sampling and the in-situ

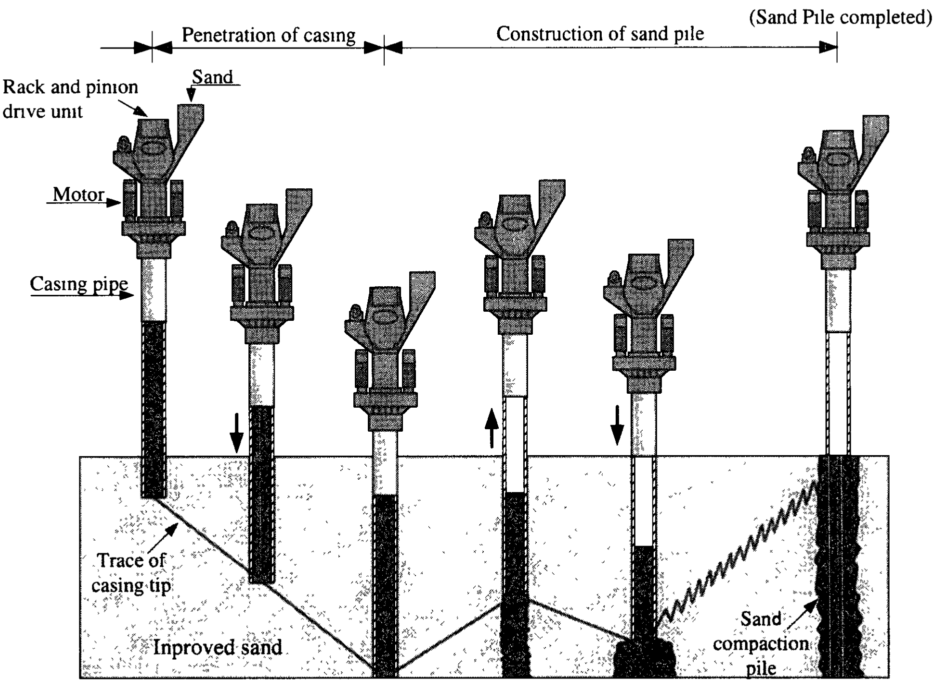


Fig. 4. Procedure of SCP installation with non-vibratory SCP technique (the procedure is just the same in the vibratory SCP technique with an exception of the use of vibro-hammer, instead of the hydraulic jack)

tests as well as ground improvement work, the dike was completely restored.

As described earlier, standard penetration tests were conducted in each site prior to the ground improvement works. The locations of SPT before ground improvement at the Niigata site, the Izumo site and the Yasugi site were very close to the sampling locations, about 3 m, 4 m and 9 m away from the sampling locations, respectively.

IN-SITU TESTS

The SPT N-value and corrected blow counts obtained by the RRS, N_d, are presented in Fig. 6. RRS is a kind of penetration test, in which a weight of 63.5 kg is dropped repeatedly from a height of 0.50 m and a blow count needed to penetrate a shaft with a 45 mm diameter cone at the tip by 0.2 m is taken as the N_{dm}-value. After every 0.2 m penetration, the torque required to rotate the shaft is measured to estimate the skin friction. In the RRS, the shaft is penetrated successively from the ground surface without using a boring hole, therefore, the N_{dm}-value can be obtained every 0.2 m easily and quickly as compared with SPT, typically half an hour for a 10 m penetration test. Allowing for the shaft friction, the N_d-value was obtained from Eq. (1) (JGS, 1995),

$$N_d = N_{dm} - 0.00041 M_v \tag{1}$$

where, M_v is the measured torque in $N \cdot$cm.

Figure 6(a) compares N-values and N_d-values of the same sand pile in the Niigata site (SPT2 and RRS3), indicating that these values virtually coincided. Figures 6(c), (f) and (i) depict SPT-N values and N_d-values of the improved grounds at the three sites. N-values after ground improvement at any depth and at any site were found to be in the range between the upper and the lower bound of N_d-values. These observations are consistent with the previous report (Sato and Iwasaki, 1980) that RRS N_d-value was almost the same as SPT N-value.

It can also be seen in Fig. 6 that N-values of sand layers in the improved ground at each site were apparently higher than those before the SCP installations for a depth deeper than three or four meters, indicating significant effects of the ground improvement work. The increase in N-value due to the SCP installations, however, was not significant near the ground surface. The threshold depth, below which the effects of ground improvement or the increase in N-value are significant, may be about 3 or 4 meters irrespective of the replacement ratio and the type of SCP construction technique, that is, the vibratory and the non-vibratory SCP. Another type of ground improvement method may be needed to improve soils at a shallower depth.

It has often been considered that the N-value, and thus the liquefaction resistances of sand piles are higher than the surrounding improved sand (e.g. Oobayashi et al., 1998). This is the case for the Yasugi site and Izumo site where the vibratory SCP technique was adopted. In these sites N-values and N_d-values in the sand piles are almost the same or somewhat higher than those in the surrounding improved sand. While in the Niigata site, in which the non-vibratory SCP technique was utilized, however, N-values and N_d-values of the sand pile (SPT2 and RRS3) were even smaller than those of the improved sand (SPT1, RRS1, 2, 4 and 5). A possible reason for the differences in the N-value of SCP between the two sites is

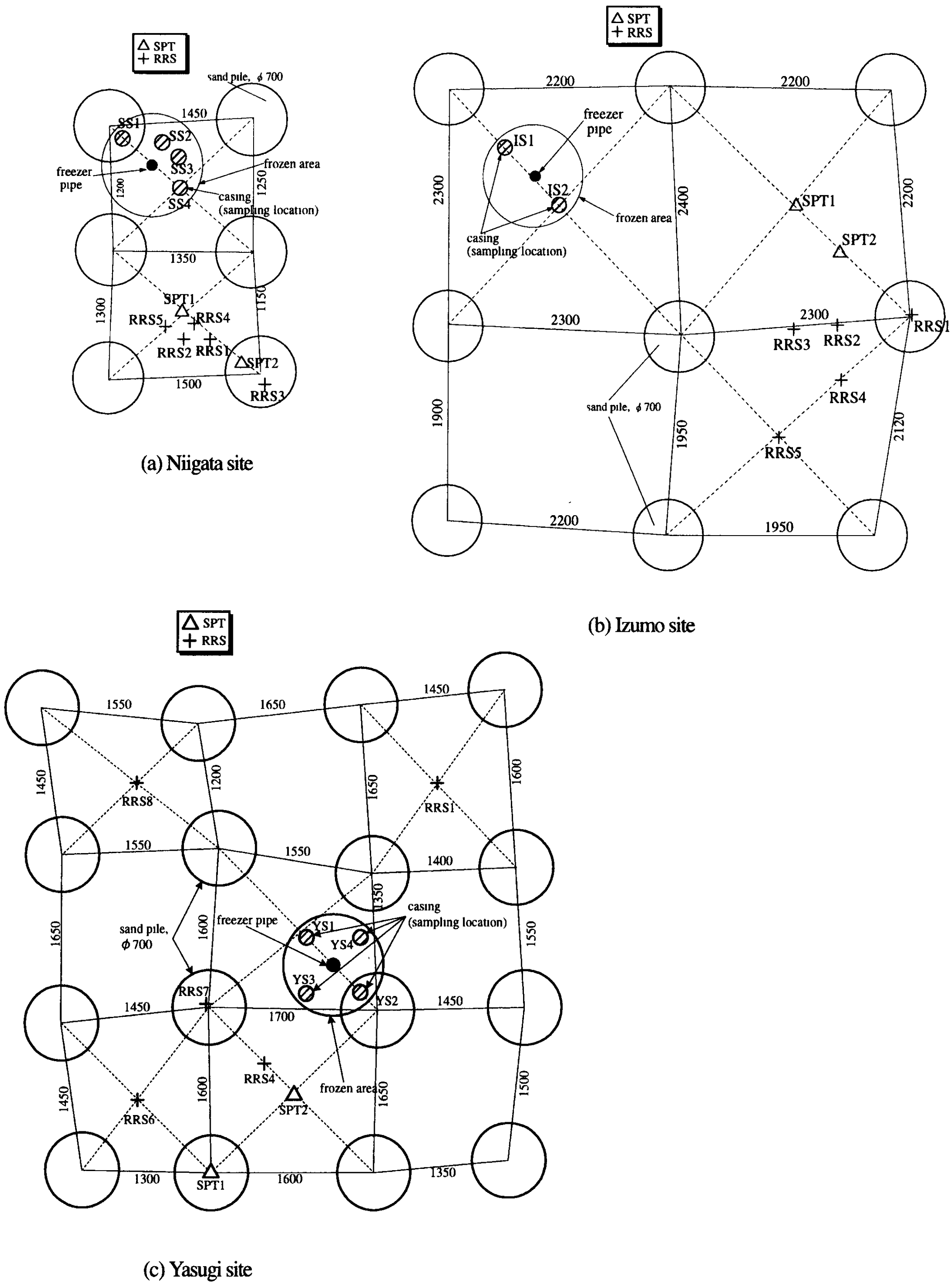


Fig. 5. Plan view of sampling and in-situ test in the three sites

the use of different SCP construction techniques; the strong vibration during the SCP installation caused by the vibro-hammer may have contributed to increase the density of sand piles. Relative density of the sand pile at the two depths in the Niigata site was in a narrow range of 47.8–56.4%, as shown in Table 1, and the N_1 value was more or less uniform with depth, indicating that the sand pile was in a medium dense condition irrespective of depth, where N_1 is the normalized blow count for the effective vertical stress of 98 kPa given by Eq. (2),

$$N_1 = \frac{N \times 1.7}{\sigma_v'/98 + 0.7} \tag{2}$$

The N_d-values at several depths of each site are plotted in Fig. 7 against horizontal distance from the center of the nearest sand pile. It should be mentioned in Fig. 5 that RRS in the Niigata site and the Izumo site were conducted at several locations in the same quadrangular area surrounded by the same four sand piles, except for RRS performed in sand piles. While in the Yasugi site, four RRS were carried out at around midpoints of different

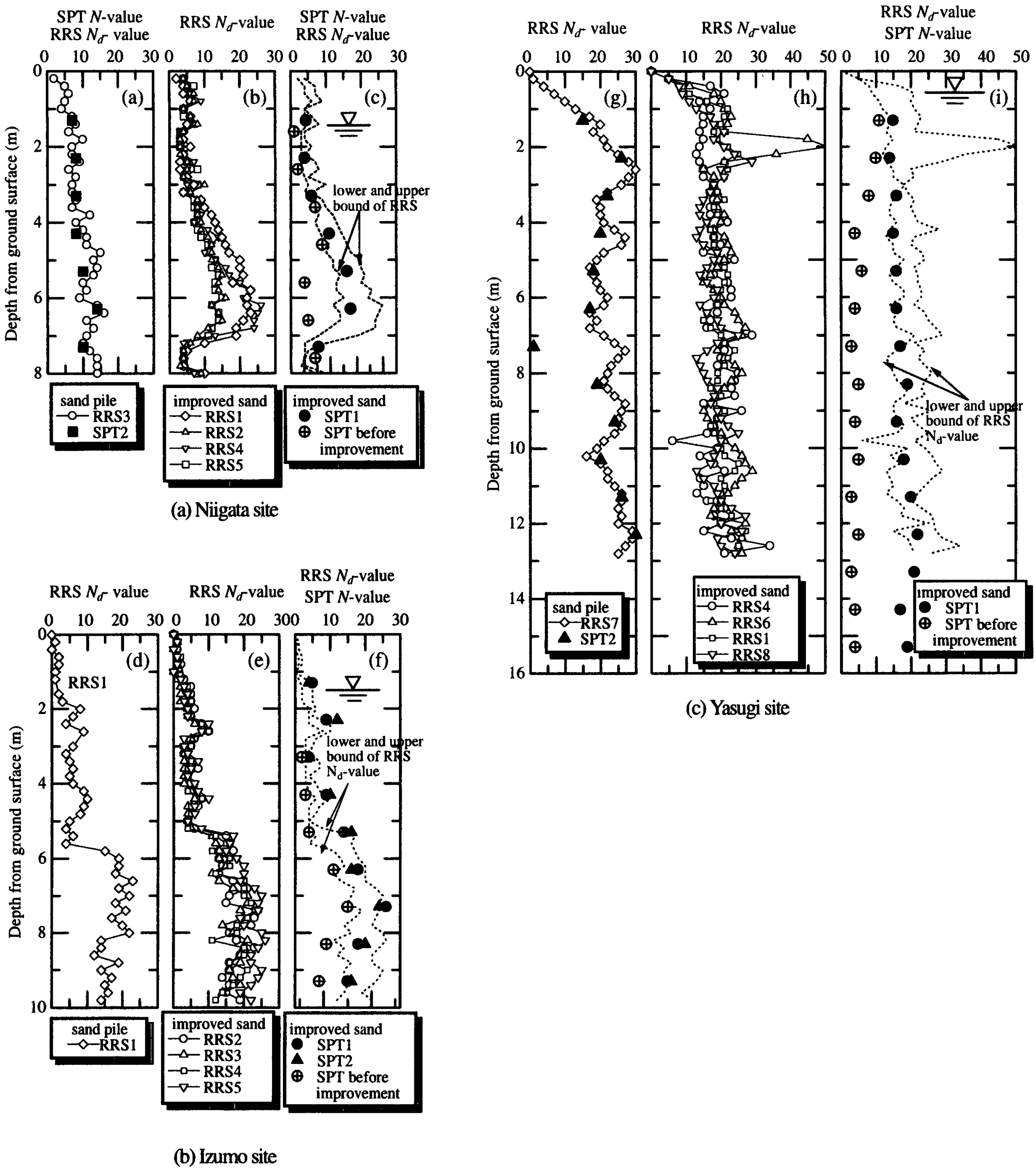


Fig. 6. Profile of N value and N_d value in the three sites

quadrangular areas. Therefore, Figs. 7(a) and (b) demonstrate variations with the distance from the same sand piles. It is often assumed that the N-value in the improved ground is largest in sand piles and decreases with horizontal distance toward the midpoint of sand piles (Ohbayashi et al., 1998). In Figs. 7(a) and (b), however, N_d-values at any depth show violent fluctuation and the clear tendency of a decrease in N_d-value with an increase in the horizontal distance cannot be detected. In Fig. 7(c), in which RRS in the Yasugi site conducted in different quadrangular areas are depicted, a relatively large scatter of N_d-value is also found. These observations imply that the improved sand is highly heterogeneous and randomly distributed in a horizontal plane. In situ tests at more than several locations may be needed to estimate the properties of improved grounds. For the purpose of execution management, RRS may be recommended owing to its easiness to perform and equivalency of results to the SPT N-value.

IN SITU FREEZING, SAMPLING OF FROZEN SAND AND UNDRAINED CYCLIC SHEAR TEST

Freezing and Sampling of Sand

High quality undisturbed samples were obtained by the in-situ freezing method using a single freezing pile

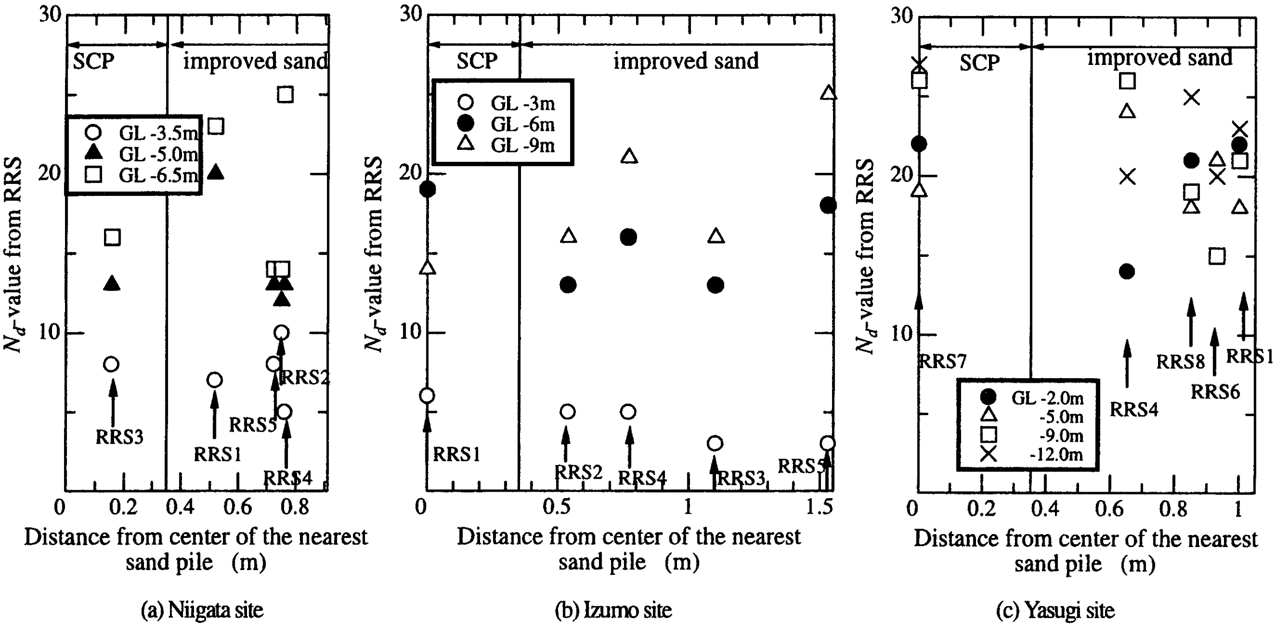


Fig. 7. Horizontal variation of N_d values

(Yoshimi et al., 1978). It has been reported that disturbance of samples due to expansive strains during freezing became significant as an increase in fines content of sands (Yoshimi et al., 1978, 1994). However, the quality of the samples obtained at the three sites was expected to be high since the sands contained small amount of fines, mostly less than 7%. The procedures of the in-situ freezing and the sampling of frozen sands are schematically illustrated in Fig. 8. Steel casings of 240 mm in diameter were set in excavated holes. The casing was used as a guide for a core barrel when sand samples were cored. The casings were equipped with thermocouples so that the position of the freezing front in the ground could be monitored. A bore hole was drilled to the predetermined depth and a freezer pipe with a diameter of 100 mm was set. Liquid nitrogen (−196°C) was poured into the pipe until the diameter of the frozen zone became 1200 mm. At the Niigata site, for instance, it took 74 hours and about 20 kN of liquid nitrogen until the diameter of the frozen zone grew to 1200 mm. Then, frozen sand samples of 150 mm in diameter were obtained by using a 150 mm core barrel. The coring was done at several different locations in each site as indicated in Fig. 5, avoiding the zone 70 mm from the surface of the freezer pile where the sands could be disturbed by the borehole excavation for the freezer pile installation (Yoshimi et al., 1984). The locations of the sampling in the Niigata site and the Yasugi site included a sand pile, a midpoint of sand piles in the quadrangular area and two more locations between these two locations, while samples were taken from two locations in the improved soil in the Izumo site. The frozen samples were wrapped with plastic sheets to minimize sublimation of the pore ice and carried to the laboratory by a refrigerator car. More detailed information about the in-situ freezing method and quality of samples has been reported by Yoshimi et al. (1984, 1994).

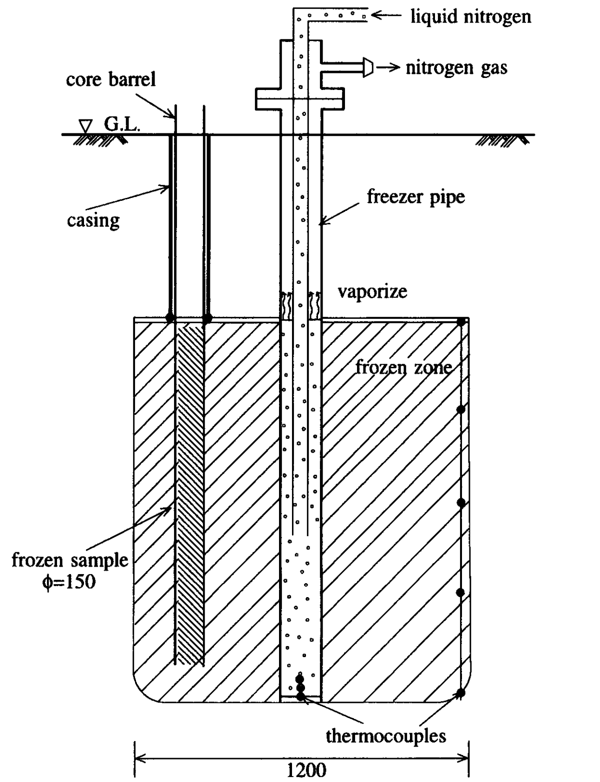


Fig. 8. Schematic illustration of in-situ freezing and sampling of frozen sand

Test Conditions of Undrained Cyclic Triaxial Test

With regard to the Niigata site, the sample SS1 obtained from the sand pile was fully composed of the dredged sand from the river bed, while the samples SS3 and SS4 from a depth deeper than 3.38 m, that had been in the vicinity of the midpoint of the sand piles in the quadrangular area, were filled with dark gray sand, which is the originally deposited sand at the location. However, the sample SS2, which had been assumed to be located outside the sand pile, consisted of a zone of the originally deposited sand and dredged sand. Possible reasons are that the diameter of the sand piles was larger than the design value of 700 mm and the sand pile was inclined during installation of adjacent sand piles. In fact, lateral displacement of the sand pile head of an order of 0.2 m was commonly observed; sand piles at each site were installed precisely in the predetermined locations in a rectangular arrangement but observed locations of the sand piles after the ground improvement show apparent displacement, as can be seen in Fig. 5.

The frozen samples were cut out and trimmed in the laboratory with a steel straight edge to specimens 50 mm in diameter and 100 mm high. The large diameter of the samples (150 mm) made it possible to prepare four specimens at the same depth from a sample with an exception of the sample SS2 at a depth between 3.9 and 7.8 m. Triaxial specimens were carefully trimmed from the originally deposited sand zone at a depth of 6.6–6.8 m from the sample SS2.

The specimens were thawed in a triaxial cell under a confining pressure of 20 kPa and saturated with deaired water until the pore pressure coefficient B value exceeded 0.95.

Table 1 summarizes test conditions. For the samples obtained from the Niigata site (SS1–SS4), the tested specimens were taken mostly from two depths, from 3.10 m to 3.36 m and 6.55 m to 6.90 m. Also, for the samples from the Izumo site (IS1 and IS2) and the Yasugi site (YS1–YS4), undrained cyclic triaxial tests were performed on specimens taken from two or three depths.

The arrows in Fig. 2 show the approximate depths where the triaxial specimens were taken. Detailed inspection of the frozen specimens revealed that very thin clay layers and silt layers were sandwiched in the sand layers at many depths. These layers were carefully avoided to obtain uniform specimens. All the undrained cyclic triaxial tests reported in this paper were performed under initial confining pressures approximately equal to the effective overburden pressures in-situ. The back pressure applied to the specimens was 196 kPa to achieve a higher B value, despite the fact that in-situ hydrostatic pressures were lower than this.

RELATIONSHIP BETWEEN LIQUEFACTION RESISTANCE AND PENETRATION RESISTANCE

Variations of the number of cycles required to cause double amplitude of axial strain (DA) of 5% with the cyclic shear stress ratio are given in Fig. 9. It should be noted here that dense sands often generate large negative pore pressures during undrained cyclic shearing. The absolute value of the possible maximum negative excess pore pressure in-situ is hydrostatic pressure plus 98 kPa, while back pressure plus 98 kPa in triaxial tests. The use of a back pressure higher than the in-situ hydrostatic pressure, which allows generation of larger negative excess pore pressure, may result in an overestimation of liquefaction resistances. However, all the triaxial tests conducted in this study were not this case. The absolute value of the maximum negative excess pore pressure measured in the tests was lower than 98 kPa. Figure 10 depicts the variation of shear stress ratios to cause DA = 5% in 20 cycles with the distance from the center of the nearest sand pile. It can be observed that liquefaction resistances are randomly distributed in the improved sand at any depth, consistent with the observations discussed in Fig. 7.

Figure 11 shows cyclic shear stress ratios for DA = 5% at 20 cycles obtained from triaxial tests plotted against the normalized blow count for an effective vertical stress of 98 kPa. The relationship used in current practice (Japan Road Association, 1996) for sandy soils with fines content less than 10% is also demonstrated in the figure. In this study, the normalized blow count was calculated by Eq. (3) with a use of RRS N_d-values as a substitute for SPT N-values, since RRS conducted at several locations in each site may well represent the penetration resistances of the grounds with a heterogeneous nature,

$$N_{1d} = \frac{N_d \times 1.7}{\sigma_v'/98 + 0.7} \tag{3}$$

where σ_v' = effective vertical stress in kPa. The range of N_{1d} at the corresponding depth, which was computed using the lower and upper bound of N_d shown in Fig. 6, is indicated in Fig. 11. It appears that there are large scatter both in N_{1d} and the cyclic shear stress ratio of the improved sand; however, the relationship in the current practice incorporation with the mean value of N_{1d} appears to estimate the cyclic shear stress ratio of the improved sand reasonably well.

DEGREE OF SATURATION

Soils below the ground water table are usually considered to be fully or nearly saturated. A large number of observed primary wave velocities (V_p) in such grounds have confirmed this. However, improved ground by SCP may not be a case of this; a large amount of air exhausted from the casing pipe may desaturate soils in an improved area. In fact, Tokimatsu and Yoshimi (1990) reported that primary wave velocities observed in a SCP improved ground were unusually low, indicating that the soil was unsaturated. Since degree of saturation of soils, S_r, has a significant effect on the liquefaction resistance (Sherif et al., 1977; Martin et al., 1978; Yoshimi et al., 1989, Ishihara et al., 1998; Tsukamoto et al., 2002), it is important to investigate degree of saturation of grounds improved with SCP. Cyclic shear tests conducted by Yoshimi et al. (1989) have indicated that a 10% decrease in S_r doubled the liquefaction resistance of loose to medium dense sand specimens. Similar results were obtained by Ishihara et al. (1998), Tsukamoto et al. (2002) and Goto and Shamoto (2002).

The primary wave velocity has successfully been employed to evaluate degree of saturation of almost fully saturated sands (Ishihara et al., 1998; Tamura et al., 2002; Yang, 2002). But it is difficult to use V_p to deter-

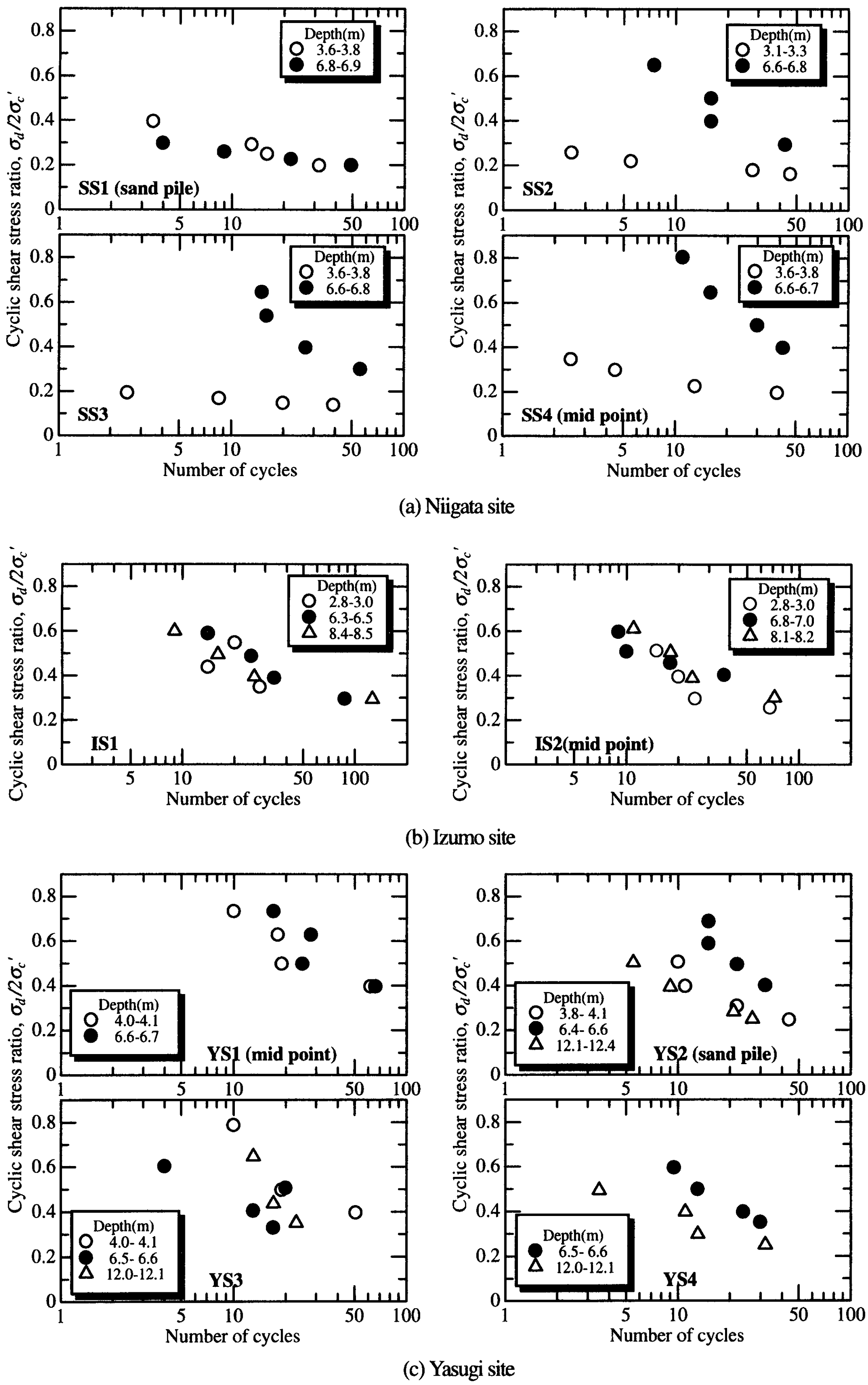
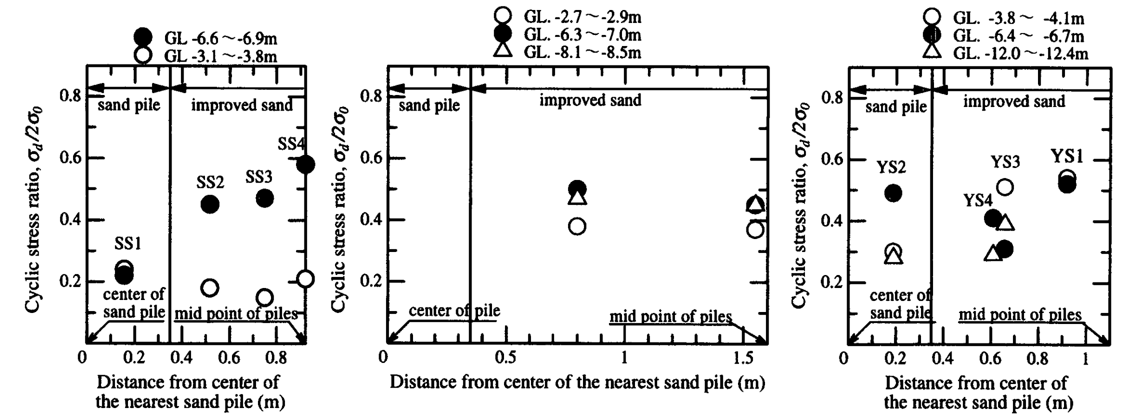


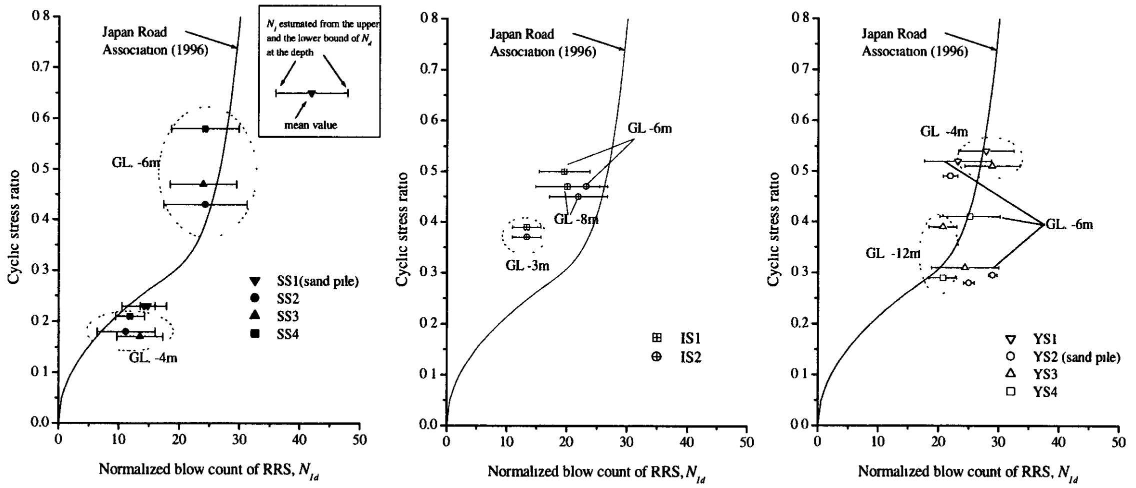
Fig. 9. Relationships between cyclic stress ratio and number of cycles to cause DA = 5%

mine S_r of partially saturated soils with S_r lower than a certain value, say about 98%, because V_p becomes essentially insensitive to a change in S_r for soils with lower S_r. An alternative method to evaluate S_r of partially saturated soils is the use of undisturbed samples obtained by ground freezing. In this study, the specific gravity, the water content and the unit weight of each frozen specimen were measured and the degree of saturation was calculated. It should be noted here that the increase in the solubility of air to water due to the fall of water tempera-



(a) Niigata site (b) Izumo site (c) Yasugi site

Fig. 10. Relationship between cyclic stress ratio obtained from triaxial tests and distance from center of the nearest sand pile



(a) Niigata site (b) Izumo site (c) Yasugi site

Fig. 11. Relationship between liquefaction resistance and normalized tip resistance

ture during the ground freezing process may result in the increase in the degree of saturation. In the Niigata site, for instance, the temperature of the soil at G.L. -8 m changed from 14 degree centigrade at the beginning of ground freezing to below zero, corresponding to the change in the solubility for water of one cubic centimeter from 0.021 cm^3 to 0.029 cm^3. Also, the volume of the air decreased in proportion to the absolute temperature in the ground freezing process. Considering these effects, the frozen samples may overestimate S_r slightly, at most 2.5% for the samples in this study.

Degree of saturation of specimens is plotted against depth in Fig. 12. It is apparent that the improved ground as well as the sand pile contained a considerable amount of air. The degree of saturation was lower than 77% for the sand pile (SS1 and YS2) and 91% for the improved sand. There is a trend where S_r decreases as the distance from the sand pile increases, but no clear trend of variation of S_r with depth can be detected. This fact implies that the liquefaction resistances of the improved sand are considerably higher than those obtained from the N-value based conventional method which is only available for fully saturated soils. Field evidences also support this presumption (Matsuo et al., 1997); a SCP improved sand in a reclaimed land did not liquefy during the 1995 Hyogoken-Nambu earthquake. The cyclic shear stress that the sand was subjected during the earthquake was considerably stronger than that estimated from the N-value based practical method.

The question to be addressed here is that how long the unsaturated condition lasts after the completion of SCP constructions. Provided that a considerable amount of air remains in SCP improved grounds for a long time and the degree of saturation is kept low enough, the effect of desaturation on the liquefaction resistance could be taken into account in design practice. The authors continue investigations of a secular change in S_r and results will be reported elsewhere in the near future.

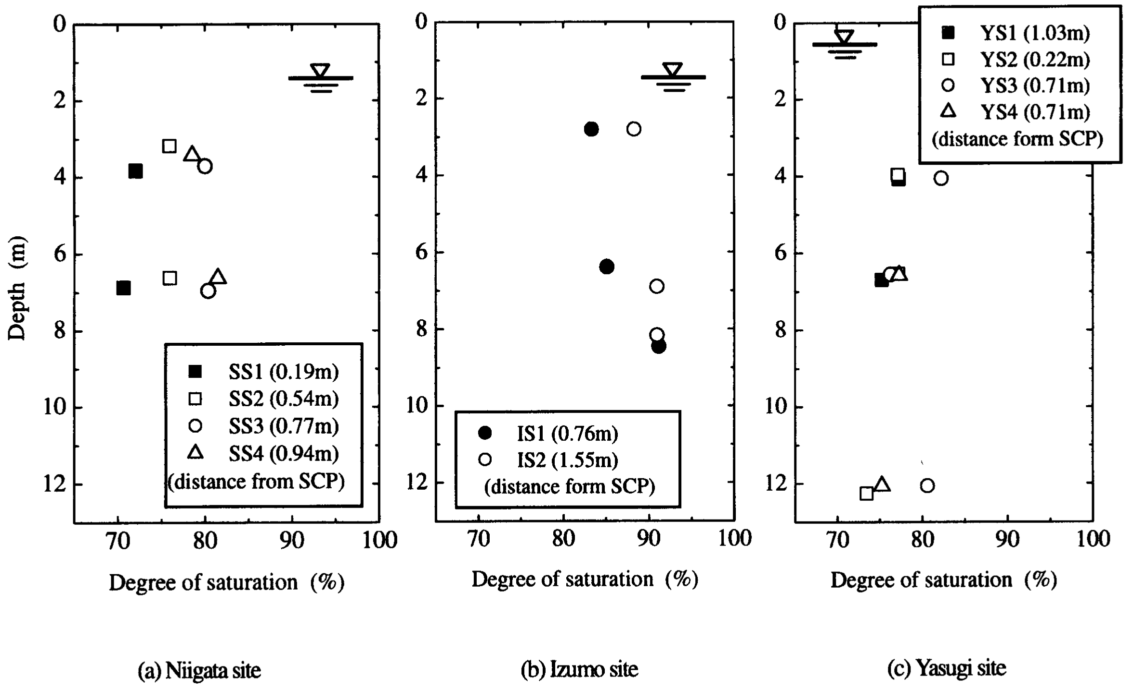


Fig. 12. Variation of degree of saturation with depth

CONCLUSIONS

In-situ tests conducted at three sites where foundation soils were improved with the vibratory or the non-vibratory SCP techniques revealed that penetration resistances are highly heterogeneous and randomly distributed in a horizontal plane at any depth. The SPT *N*-value at the center of the rectangular area (the midpoint of sand piles) does not always provide a conservative evaluation of the liquefaction resistance of improved sand. It is recommended that in-situ tests be conducted at more than several locations for the evaluation of mechanical properties of improved ground and execution management. It was also found that both the vibratory and the non-vibratory SCP techniques could not increase significantly the liquefaction resistance of soils near the ground surface, from ground surface to a depth of about three or four meters.

High quality undisturbed samples were obtained at each site by the in-situ freezing method and cyclic triaxial tests were carried out. All the specimens were saturated in a triaxial cell after thawing. It was found that there is a good correlation between the liquefaction resistance and mean value of N_d obtained at several locations. It was confirmed that the conventional method to assess the liquefaction resistance of sands adopted in the current practice incorporated with the mean value of N_d yielded the cyclic shear stress ratio of the improved sand reasonably well, as long as the sand is fully saturated. It was revealed that degree of saturation, S_r, of specimen was in the range between 70% and 91%. This fact implies that the liquefaction resistance of the improved sand is considerably higher than those obtained from the *N*-value based conventional method which is only available for fully saturated soils.

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