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LIQUEFACTION RESISTANCES AND DEGREE OF SATURATION OF SAND IMPROVED WITH SAND COMPACTION PILES

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SUMMARY

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 the sand compaction pile. The relationship between liquefaction resistances and *N*-value, which was established based mainly on the field evidences of earthquake-induced liquefaction of natural soil deposits and reclaimed lands, compared quite well with that obtained from tests on fully saturated improved sand specimens. 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. Undrained cyclic shear tests were also carried out on unsaturated specimens and effects of desaturating sand on undrained cyclic shear strength and deformation characteristics were studied.

INTRODUCTION

The sand compaction pile (SCP) has been widely used to ameliorate liquefaction resistance of loose sand deposits. In the current practice for designing SCP as a liquefaction countermeasure, the relationship between liquefaction resistances and *N*-value, which was developed based mainly on the field evidences of earthquake-induced liquefaction of sands in natural deposits and reclaimed lands, are used without validation of its applicability to SCP improved ground. In this study, in order to verify the validity of the relationship, a series of undrained cyclic shear tests were performed on high-quality undisturbed samples obtained by the in-situ freezing method from foundation soils improved with SCP.
It is usually considered that soils below the ground water table are 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 the case for this because a large amount of air exhausted from casing pile may desaturate soils in an improved area. Degree of saturation of improved ground was investigated using the frozen sample. Effects of desaturation of improved sands on undrained shear strength and deformation characteristics were studied through undrained cyclic shear tests on unsaturated specimens.

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IN SITU TESTS AND SAMPLING SITES

Standard penetration tests and sampling of high quality undisturbed samples were conducted at three sites where foundation soils had been improved with sand compaction piles.

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**. **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 extended down to 3.2 m from the ground surface, the foundation soil was mostly consisted of a dark gray clean sand to the depth of 7.2 m. The sand below the water table to the depth of 7.5 m was judged liquefiable and expected to cause flow deformation towards river when the sand liquefy. In order to reduce the deformation of the sand during an earthquake, the sand layer was improved with SCP.

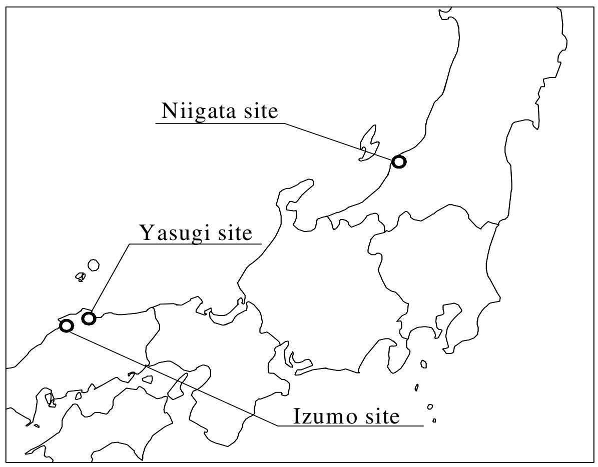

Fig. 1 Locations of sampling and in-situ test site

Figure 3 indicates schematically the procedure of the SCP installation used in the site. A casing pipe with a diameter of 0.4 m was penetrated by a hydraulic jack into the ground to the 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 of the ground surface within the area about several meters from the casing. This is the common practice in construction of the sand compaction piles. The sand used to build the sand piles was dredged from Shinano River bed near the site.

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 1.2m and 1.4m, giving rise to the replacement ratio of 23 %. About a month after the completion of the sand pile installation, high quality undisturbed samples were obtained from both sand pile and improved sand (sand between sand piles) by the in-situ freezing method (Yoshimi et al. [1]) and the standard penetration tests (SPTs) were conducted at locations within 3 m from sampling locations.

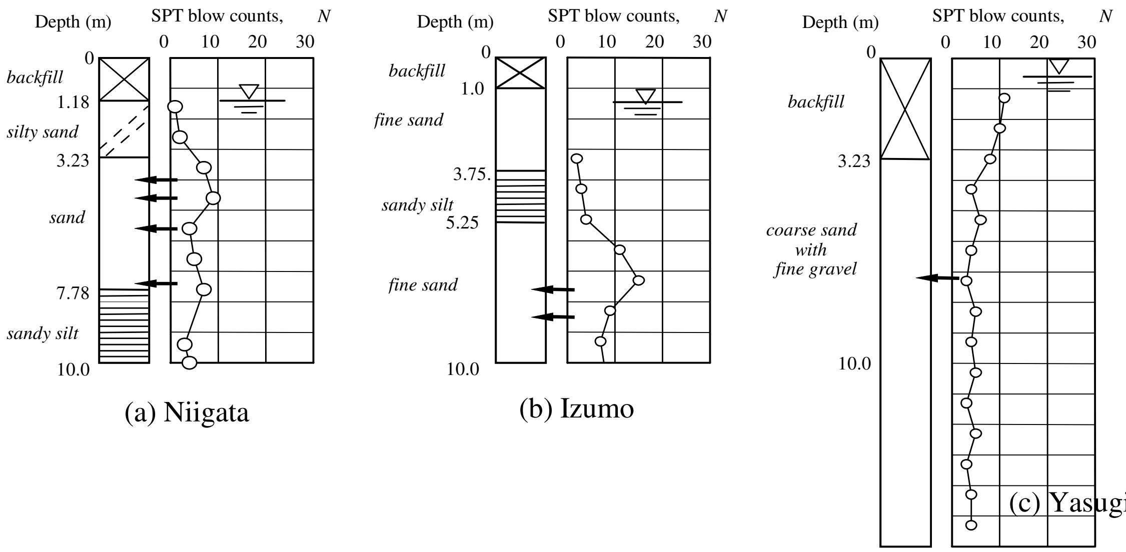

Fig. 2 Soil profiles in three sites before ground improvement
(Arrows indicate approximate depths where test specimens were obtained)

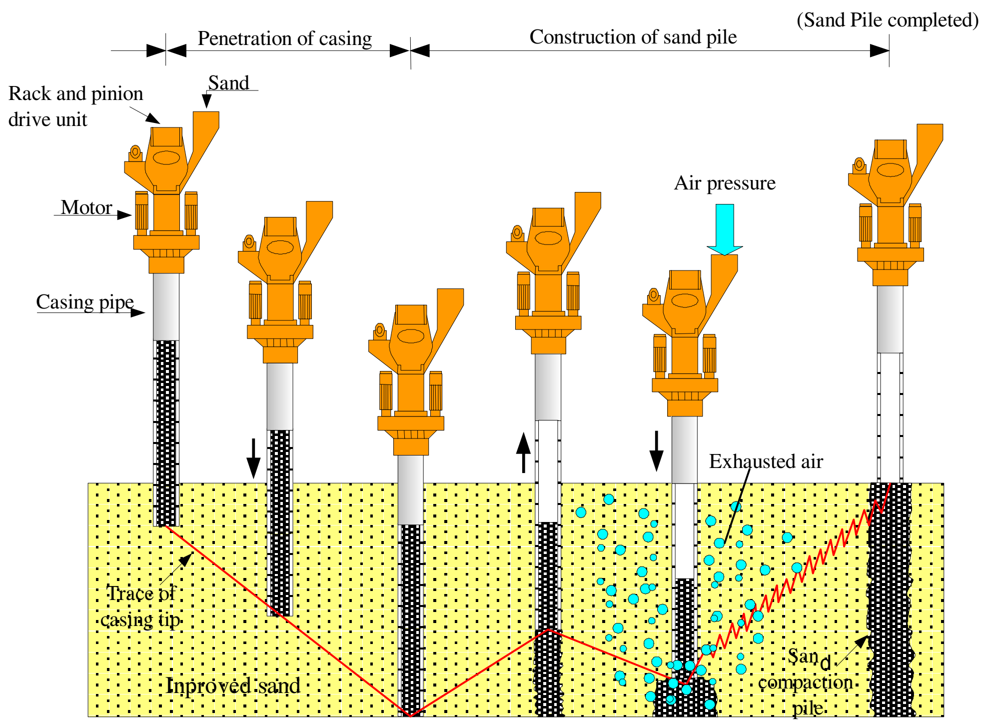

Fig.3 Schematic illustration of SCP installation procedure

Izumo site

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 planed. **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 was mostly consisted of fine sand and sandy silt to the depth of 10 m. In the fine sand layer extended from 1.0 m to 2.1 m wood pieces and humic soils were found to be mixed.
The sand below the water table to the depth of 10 m was judged liquefiable, therefore, the sand layer below the abutment was improved with SCP. The material of the sand piles was a medium clean sand with very small fines contents obtained from the river near 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 the replacement ratio of 8 %. About a month after the completion of the ground improvement, high quality undisturbed samples were obtained from improved sand by the in-situ freezing method and SPTs were conducted at locations within 5 m from the sampling locations.

Yasugi site
During Tottoriken-seibu earthquake of October 2000, some parts of dikes protecting the coastline of Nakaumi were damaged (Tokida et al. [2]). At Yasugi site, which located some 50 m from the mouth of Iinashi river, about 2.1 m high dike subsided as much as 1.2 m due to the liquefaction of foundation soils. **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 ground surface. Except for the fine sand fill deposited at the surface, the foundation soil was mostly consisted of dark gray coarse sands with some inclusion of gravel to the depth of 16 m.
A total of 680 sand piles with some 0.7m 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. About a week after the completion of the ground improvement, high quality undisturbed samples were obtained by the in-situ freezing method from a sand pile and improved sand, and SPTs were conducted at locations within 5 m from the sampling locations. On completion of the sampling and the in-situ tests as well as ground improvement work, the dike was completely restored.
In prior to the ground improvement works, SPTs were conducted in each site at locations within several meters from the sampling locations. More detailed information about the in-situ tests can be found elsewhere (Okamura et al. [3])

IN-SITU TEST RESULTS

Figure 4 compares *N*-values obtained before and after the ground improvement. It can be seen that *N*-values of sand layers in the improved ground at each site were apparently higher than those before the SCP installations for the 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, seems to be about 3 or 4 meters irrespective of the replacement ratio. Another type of ground improvement method may be needed to improve soils at a shallower depth.

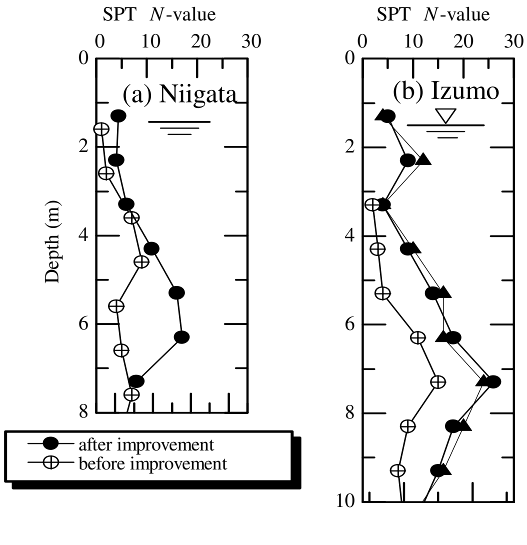

Fig.4 Profiles of *N*-value before and after SCP installation

UNDRAINED CYCLIC SHEAR TESTS ON FULLY SATURATED SPECIMENS

Test conditions

The frozen samples were cut out and trimmed in the laboratory with a steel straight edge to hollow cylindrical specimens with 100 mm and 60 mm in outer and inner diameter respectively and 100 mm high. The specimens were thawed in a test cell under a confining pressure of 20 kPa and saturated with deaired water until the pore pressure coefficient *B* value exceeded 0.95.

The arrows in **Fig.2** show the approximate depths where the 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 tests

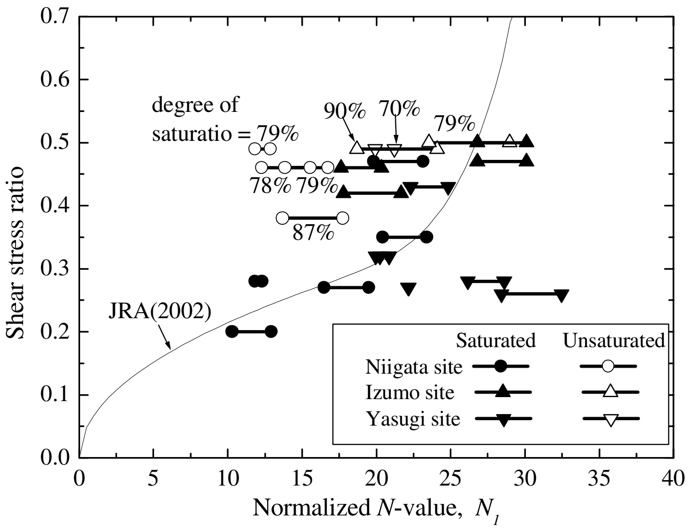

Fig. 5 Relationship between liquefaction resistance and normalized *N*-value

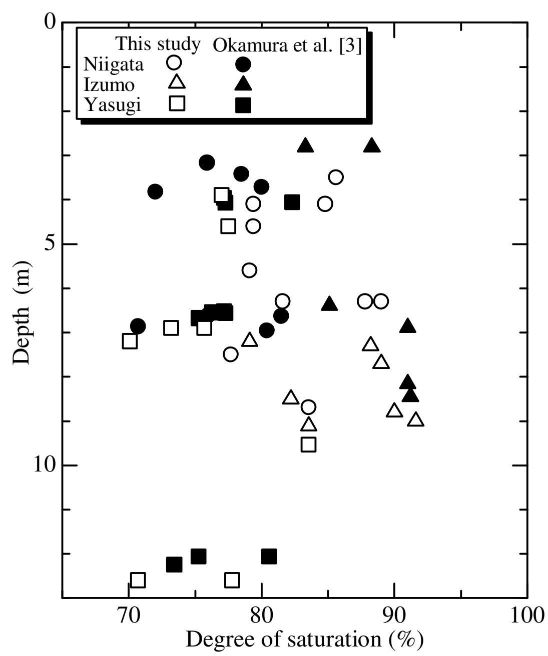

Fig.6 Degree of saturation

reported in this paper were performed under the initial confining pressures approximately equal to the effective overburden pressures in-situ.

Relationship between Liquefaction Resistance and Penetration Resistance

Liquefaction resistances in terms of cyclic shear stress ratios for double amplitude shear strain of 7.5 % in 20 cycles are plotted against the normalize blow count in **Fig. 5**. The relationship used in current practice (Japan Road Association [4]) for sandy soils with fines content less than 10 % is also presented in the figure. The normalized blow count for effective vertical stress of 98 kPa, N_1, was calculated by equation (3),

$$N_1 = \frac{N \times 1.7}{\sigma_v'/98 + 0.7} \qquad (3)$$

where σ_v' = effective vertical stress in kPa. A frozen sample with 0.5 m long was needed to determine a cyclic stress ratio at a depth from a set of tests on four specimens. The range of N_1 of each data point in **Fig. 5** corresponds to the lowest and the highest N_1 value at the depth where a set of specimens were taken from. Although some scatter of the data points exist, the relationship in the current practice incorporation with the mean value of N_1 is appeared 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 the case for this because a large amount of air exhausted from casing pile may desaturate soils in an improved area. In fact, Tokimatsu and Yoshimi [5] reported that primary wave velocities observed in a SCP improved ground were unusually low, indicating that the soil was desaturated. Since degree of saturation of soils, S_r, has a significant effect on the liquefaction resistance (e.g. Sherif et al. [6]) it is important to investigate degree of saturation of grounds improved with SCP. The primary wave velocity has successfully been employed to evaluate degree of saturation of almost fully saturated sands (e.g. Ishihara et al. [7]). But it is diffucult to use V_p to determine 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. Alternative method to evaluate S_r of partially saturated soils is the use of undisturbed samples obtained by the ground freezing. In this study, the specific gravity, the water content and the unit weight of each frozen specimen were measured and degree of saturation was calculated.

Degree of saturation of specimens is plotted against depth in **Fig.6**. In this figure, degree of saturation of triaxial specimens reported by Okamura et al. [3] is also included. It is apparent that the improved ground as well as the sand pile contained considerable amount of air. The degree of saturation was in a range between 70 % and 91 %. 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.

UNDRAINED CYCLIC SHEAR TESTS ON UNSATURATED SPECIMENS

Test procedure

In this study undrained torsional cyclic tests on the partially saturated specimen were also conducted. Attention was paid to minimize the change of S_r through the course of specimen preparation and the test. A hollow cylindrical specimen taken from a frozen sample was set in the torsional shear apparatus and a vacuum of –93 kPa was applied in the specimen for a couple of minutes to suck air in the connecting tubes, porous stones and space between the membrane and the specimen. Then deaired water was introduced through the bottom porous stone. The water was flowed from the bottom up for about a minute. At this moment, the region of the specimen only about 1 to 2 mm from the surface was thawed. The imparted water flow expelled air in the system and specimen in the thawed zone, but the air entrapped in the still frozen zone was thought to be intact. Then, the specimen was allowed to thaw completely in the drained condition under the effective confining pressure of 20 kPa. Finally, undrained cyclic torsional shear was applied to the specimen.

Results and discussions

In **Fig. 5**, the shear stress ratios of unsaturated specimens required to induce double amplitude shear strain of 7.5 % in 20 cycles are compared with those of saturated specimens. The liquefaction resistances of the unsaturated specimens were found to be considerably higher than saturated specimens. The difference in the liquefaction resistances is more apparent for lower N_1-value. It was reported by Yoshimi et al.[8] that the lower the degree of saturation, the higher the liquefaction resistance. However, such a trend is not clearly observed in this study. The liquefaction resistances of unsaturated specimens are in the relatively small range between 0.4 and 0.5 irrespective of N_1-value and the degree of saturation. This indicates that the liquefaction resistance a loose sand, say with N_1-value of 15, is almost doubled by lowering the degree of saturation, while the stress ratio of dense sand, say N_1-value of 25, do not change significantly.

Shown in **Fig. 7** is evolution of shear strain amplitude with number of cycles for both saturated and unsaturated specimens with N_1-vlaue of about 12, 22 or 25. Each test indicated in this figure is subjected to different cyclic stress ratios (τ_a/σ_0') but caused roughly the same double amplitude shear strain of 7.5% in 20 cycles. For cases of loose sand with $N_1 = 12$, both unsaturated and saturated samples shows similar trend of the shear strain evolution, which is typical of loose sand; the shear strain amplitude increased markedly for number of cycles larger than about 20. While for sand with N_1 higher than 22, the shear strain amplitude increased more or less in a constant rate, which is typical of medium dense and dense sand. From these observation, it can be said that shear stress ratio of loose sand can be enhanced to about 0.5 by desaturating the sand, however, deformation characteristics are not improved since the soil skelton is unchanged.

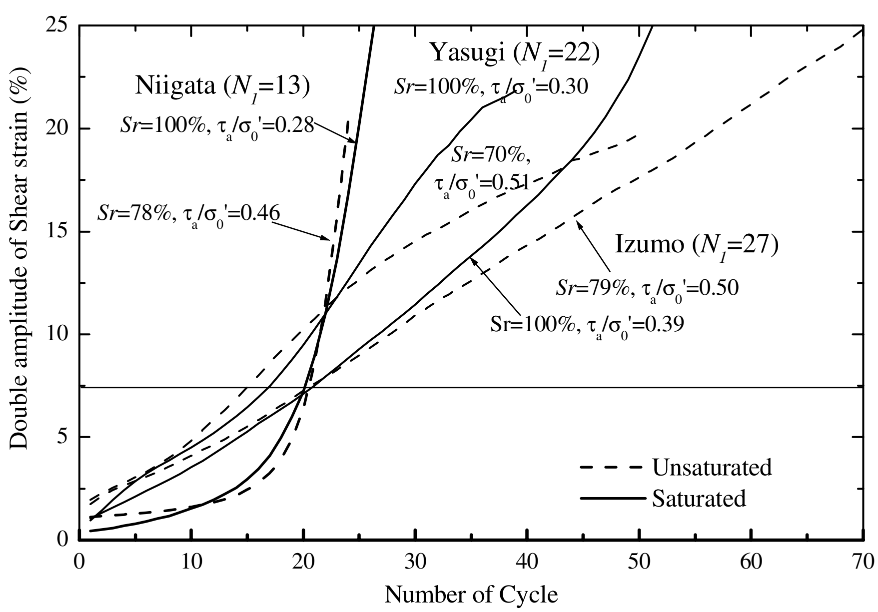

Fig. 7 Evolution of shear strain amplitude during cyclic shearing

CONCLUSIONS

High quality undisturbed samples were obtained at three site by the in-situ freezing method, where foundation soils were improved with SCP, and cyclic tortional shear tests were carried out. From tests on fully saturated specimens, it was found that there is a good correlation between the liquefaction resistance and SPT *N*-value. It was confirmed that the conventional method to assess the liquefaction resistance of sands adopted in the current practice incorporation with the mean value of *N* yielded the liquefaction resistance of the improved sand reasonably well as long as the sand is fully saturated. But it was revealed that degree of saturation, S_r, of specimen was in the range between 70 % and 91 %.
Additional undrained torsional cyclic tests were conducted on the partially saturated specimen. It was found that shear liquefaction resistance of loose sand can be enhanced to about 0.5 by desaturating the sand, however, deformation characteristics are not improved since the soil skelton is unchanged in a loose state.

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