CRITERIA OF SOIL LIQUEFACTION WITH SPT AND FINES CONTENT

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SUMMARY

A critical review of field performance of sandy soil deposits during earthquakes indicates that (1) sands containing fines have much greater resistance than clean sands with the same SPT N-value, (2) this tendency appears to become significant as the fines content increases, and (3) no soil containing more than 20% fines liquefied. On the basis of the field study, together with laboratory tests on undisturbed sands by freezing sampling, an improved empirical chart separating liquefiable and non-liquefiable conditions is presented in terms of fines content, shear strain amplitude, and SPT N-value adjusted to net energy delivered to the rod.

INTRODUCTION

Despite its limitation as to lack of rigid standardization, the standard penetration test (SPT) N-value has been extensively used as a convenient parameter to express resistance of sandy soil to liquefaction probably based on the following:

1. The SPT is an in situ test which can reflect stress history and strain history effects, soil fabric, and horizontal effective stress. All of the factors are known to influence the resistance of sands to liquefaction but are difficult to retain in most so-called "undisturbed" samples.
2. Numerous case histories of soil liquefaction with SPT N-values can reflect in situ soil characteristics under real stress conditions during earthquakes which are difficult to simulate thoroughly in the laboratory.
3. Unlike the cone penetration test that can be regarded as a drained test for sandy soils, the SPT is primarily a shear strength test under essentially undrained conditions because it is conducted under rapid strain rate.

Recent studies have shown that there is a significant variation in SPT N-value that can be related mainly to energy loss due to friction between rope and cathead. However, SPT N-values adjusted to energy delivered to the rod have scarcely been discussed in criteria of soil liquefaction. On the other hand, it has often been pointed out that the presence of fines tends to reduce SPT N-value of sands without significantly affecting shear strength. Even though many studies have been made to clarify the effects of fines on the liquefaction resistance in the laboratory, little attention has been paid to field performance of silty sands during earthquakes.

On the basis of the above considerations, a critical review of field performance of sandy soil deposits during earthquakes is reported herein. Special emphasis is placed on fines content and energy loss in SPT N-value, in order to establish a more reliable empirical chart for estimating liquefaction resistance.

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FIELD CORRELATION OF SOIL LIQUEFACTION BASED ON SPT N-VALUE AND FINES CONTENT

Many investigators have reported field evidence of soil liquefaction of which more than 70 case histories in Japan during 10 earthquakes are available as well as about 20 supplemental data outside Japan. Soil and seismic data of these case histories are summarized elsewhere (Tokimatsu and Yoshimi, 1).

Shear Stress Ratio and SPT N-value

Dynamic shear stress ratio for a given depth at a given site is defined by

$$\frac{\tau_d}{\sigma_o'} = \frac{\alpha_{max}}{g}\frac{\sigma_o}{\sigma_o'} r_d r_n \qquad (1)$$

in which τ_d = amplitude of uniform shear stress cycles equivalent to actual seismic shear stress history, α_{max} = the maximum horizontal acceleration at ground surface, σ_o = initial vertical stress, σ_o' = initial effective vertical stress, and r_d and r_n are correction factors in terms of depth and earthquake magnitude, M, respectively, as follows:

$$r_d = 1 - 0.015\,z \qquad (2)$$

$$r_n = 0.1\,(M - 1) \qquad (3)$$

in which z = depth below the ground surface in meters. To facilitate comparison of field behavior during earthquakes of different magnitudes, the factor r_n in Eq. (3) is defined so that a given number of cycles, N_ℓ, of 0.65 times the maximum shear stress amplitude can be equivalent to 15 cycles of r_n times the maximum shear stress amplitude in terms of cumulative damage concept. If M = 7.5, Eq. (3) gives r_n = 0.65 which is the same as used in currently available procedures.

On the other hand, the SPT N-value normalized for $\sigma_o' = 1$ kgf/cm^2 (98 kPa), N_1, to express soil resistance may be given approximately by

$$N_1 = C_N N = \frac{1.7\,N}{\sigma_o'(\text{kgf/cm}^2) + 0.7} \qquad (4)$$

in which C_N is a function of the effective vertical stress at the time when and at the depth where the penetration test was conducted. Eq. (4) is primarily based on the test results by Gibbs and Holtz (2) for the effective stress up to 2.8 kgf/cm^2 (275 kPa) that covers the range of general interest of soil liquefaction.

To clarify the influence of energy loss due to the friction on measured SPT N-values, comparative field tests for clean sands were conducted by Yoshimi and Tokimatsu (3), from which the following approximate relationships have been derived:

$$N_{cj,\text{with friction}} \simeq 1.2\,N_{tj} \qquad (5)$$

$$N_{cj,\text{without friction}} \simeq N_{tj} \qquad (6)$$

in which cj means the cathead and rope method in Japan, and tj means the trip monkey method in Japan. It is conceivable, therefore, that N-values measured by the cathead and rope method in an attempt to reduce the friction would produce almost the same values as those by the trip monkey method with a net energy ratio of about 0.8 to the theoretical maximum (Nishizawa et al, 4). Since the ratio of the net energy delivered to the rod in the USA was estimated to be about 0.5 to 0.6 by Kovacs and Salomone (5), the following relation is assumed for interpreting SPT N-values from the USA.

$$N_{cUS} \simeq (1.3 \sim 1.4)\,N_{tj} \qquad (7)$$

in which cUS means the cathead and rope method in the USA. In the following pages, SPT N-values are corrected to the probable net energy of Japanese trip monkey method.

Soil Classification of Liquefied Soils

A triangular classification chart for the liquefied soils is shown in Fig. 1 to identify soil type which might have liquefied. Note that none of the soils containing more than 20 percent clay liquefied. This fact is in good agreement with a study in China (Seed et al, 6). The liquefied soils are also classified in Table 1 according to the unified soil classification system. More than 50 percent of liquefied soils are so-called clean sands labelled as poorly graded sand, SP, whereas no soil classified as gravel liquefied.

Table 1 Unified soil classification for liquefied soils

Major Division		Fines Content (%)	Symbol	No. of Cases	Note
Coarse Grained Soils	Gravel		GW,GP	0	
	Sand	5	SP	26	
			SP-SM	5	
		12-15	SW-SM	3	
	Sand with Fines		SM	11	
			SM-SC	2	
		45	SM-SC/SC	1	PI=NP∼15%
			SM-SC ML-CL/ML	3	CC=8, 15%, UN
Fine Grained Soils	Silt	55	ML	1	LL=30%, NP
			ML-CL	2	CC=15%, UN
	Clay		CL	0	

CC = Clay Content, UN = Unknown

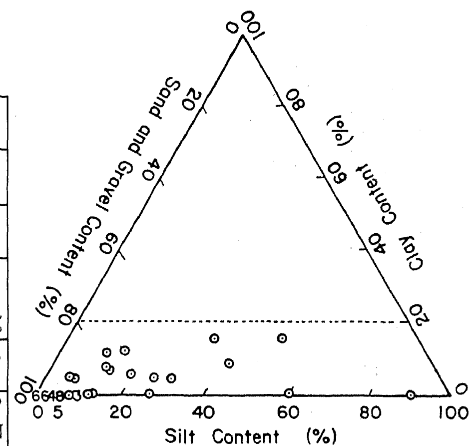

Fig. 1 Triangular classification chart for liquefied soils

Fig. 1 and Table 1 show that soils with more than about 50 percent fines hardly liquefied, except for a few cases including mine tailings that had no plasticity even though the fines content was as much as 90 percent. According to a study in China (Finn, 7), plasticity index of 10 seems to be a threshold for liquefaction. Although more field evidence is needed to clarify this point, it is conceivable that the plasticity index could be a promising parameter for estimating liquefaction resistance of silty soils, considering the fact that it is more reliable yet easier to determine than clay content.

Relationship between Shear Stress Ratio and SPT N-value

Figs. 2 and 3 show the relationship between either fines content or mean grain size and SPT N_1-value for the liquefied soils. A significant reduction in SPT N-value for silty sands with either more than 10 percent fines or mean grain size less than 0.2 mm as compared with clean sands is apparent, which would appear to confirm the fact that the presence of fines reduces SPT N_1-values without significant changes in liquefaction resistance. Data in Fig. 2 show that there is a fairly well-defined trend in which the SPT N_1-values for the liquefied soils decrease with an increase in fines content. Although the mean grain size may be used as an index to separate soil type (Tokimatsu and Yoshimi, 8), the reduction in SPT N-value with increasing fines content cannot be expressed by using mean grain size as far as the field data in Fig. 3 are concerned. Using the fines content as an index parameter for estimating liquefaction resistance has additional advantages over the mean grain size: (1) The fines content is probably better related with soil consistency which in turn is related to undisturbed shear strength of soil, and (2) The fines content can be determined more easily than the mean grain size by washing a soil sample through a 74 μm sieve.

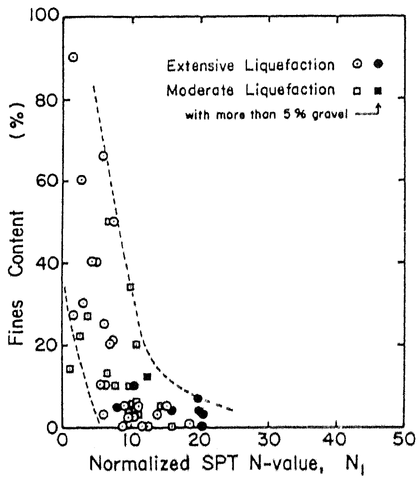

Fig. 2 Relationship between fines content and SPT N_1-value for liquefied soils

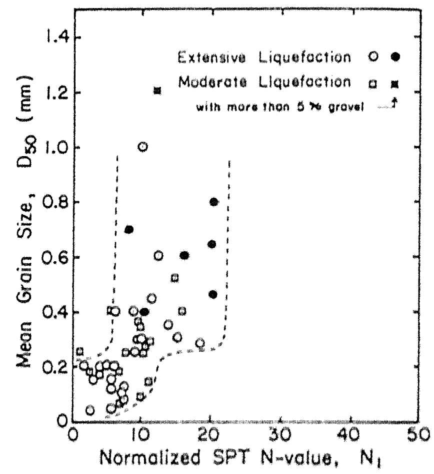

Fig. 3 Relationship between mean grain size and SPT N_1-value for liquefied soils

It is also noted in Fig. 2 that SPT N-values for sands containing some gravels tend to become larger than those for clean sands without gravels. This fact probably reflects an increase in penetration resistance due to the presence of the large particles.

On the basis of the above findings, field correlations between shear stress ratio and normalized SPT N_1-value are shown in Figs. 4 to 6 for clean sands with less than 5 percent fines, for silty sands with more than 10 percent fines, and for silty soils with about 50 percent fines, respectively. It is apparent from these figures that soils containing fines have much greater resistance than clean sands with the same SPT N-values, and that this tendency appears to become significant as fines content increases. It seems also that there is a critical SPT N-value above which liquefaction may not occur. This value is about 20 to 25 for clean sands and about 15 to 20 for silty sands.

RELATION BETWEEN FIELD PERFORMANCE AND LABORATORY TESTS

Comparison with Laboratory Tests

In order to assure the presence of the critical SPT N-value, undisturbed samples of dense sand were extracted recently in Niigata city by means of an in situ freezing method, and extensive cyclic triaxial tests were conducted on the samples (Yoshimi et al, 9). Detailed procedure and results of the in situ test and the laboratory test will be presented elsewhere by the authors. For comparison purposes, the data from the freezing sampling is plotted in Fig. 4. The data by the in situ freezing sampling seems to be consistent with the field observation, indicating that there exists a critical SPT N-value above which liquefaction would not occur. Also shown in Fig. 4 is a test result for dense sands obtained by a conventional tube sampling. The data appears to show a significantly lower strength than those from the field observation, which suggests the possibility that considerable underestimation of undrained strength for dense sands would be expected if criteria are based on laboratory test on samples by a conventional method.

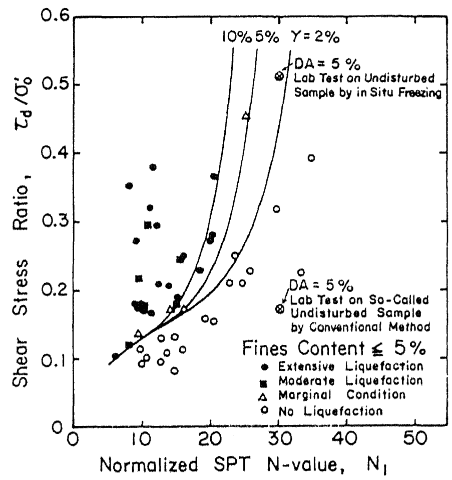

Fig. 4 Field correlation between shear stress ratio and SPT N_1-value for clean sands

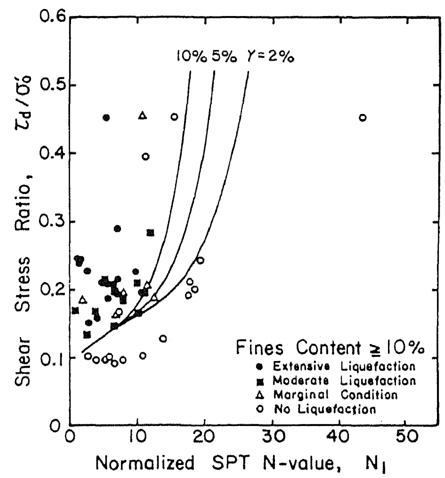

Fig. 5 Field correlation between shear stress ratio and SPT N_1-value for soils with more than 10% fines

Also shown in Figs. 4 to 6 are the boundary curves in terms of the maximum cyclic shear strain to be developed during earthquakes, tentatively drawn based on the field performance data together with the test data for dense sands by freezing sampling, as well as previously published data for medium to loose sands (Tokimatsu and Yoshimi, 1). The boundaries separating liquefaction and no liquefaction conditions seem to be well defined in Fig. 4 for clean sands and can be used as a reliable basis for estimating liquefaction potential for practical purposes.

On the other hand, because the data scarcely include silty soils with more than about 50 percent fines, application of the boundary curves to these soils may be considered less reliable. Since FC = 50% is exactly the same as D_{50} = 74 μm, the proposed criteria can be directly compared with the criteria by Iwasaki et al (10) that is expressed in terms of the mean grain size on the basis of laboratory tests on samples by conventional sampling methods. Fig. 6 shows that the proposed criteria are in good accord with their boundary curve for SPT N-value less than 10, where sample disturbance during sampling, handling and reconsolidation may not have a significant influence on the test results or may counterbalance them. For higher SPT N-values, however, the criteria tend to diverge from the other, showing a rapid increase in shear strength. Considering the possibility of significant reduction in soil resistance due to sample disturbance in the laboratory for the higher SPT N-values, the criteria presented herein appear to be reasonable for silty soils as well. However, more field behavior studies of silty soils are needed for definite confirmation.

Shear Stress Developed in the Field

The boundary curves shown in Figs. 4 to 6 may be defined by the following equation for the cyclic shear stress ratio up to 0.5:

$$\frac{\tau_\ell}{\sigma_o'} = a\,C_r\left[\frac{16\sqrt{N_a}}{100} + \left(\frac{16\sqrt{N_a}}{C_s}\right)^n\right] \qquad (8)$$

in which a = 0.45, C_r = 0.57, n = 14, C_s is a parameter depending on shear strain as listed in Table 2, and N_a is an adjusted SPT N-value defined by

$$N_a = N_1 + \Delta N_f \qquad (9)$$

where ΔN_f is a variable depending on the fines content as shown in Table 3, which was assumed from Figs. 2, 4, 5 and 6.

Fig. 7(a) summarizes field correlation in terms of the SPT N_a-value for various soils except for gravelly soils containing more than 25% gravel particles. As might be expected, liquefaction condition appears to be uniquely determined irrespective of fines content.

Table 2

Shear Strain (%)	C_s
2	88
5	81
10	75
25	60

Table 3

Fines Content FC (%)	ΔN_f
0 - 5	0
5 - 10	Interpolate
10 -	0.1 FC + 4

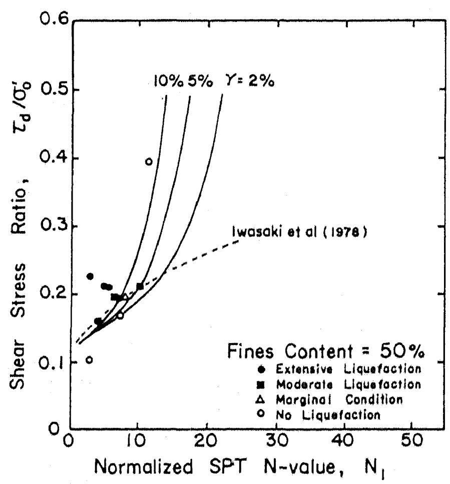

Fig. 6 Field correlation between shear stress ratio and SPT N_1-value for soils with about 50% fines

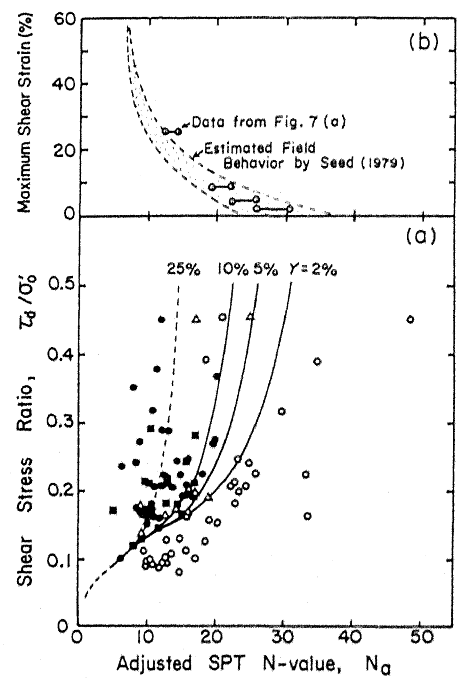

Fig. 7 (a) Relation between shear stress ratio and SPT N_a-value for various soils (b) Maximum shear strain potential

The maximum cyclic shear strains to be developed for cyclic shear stress ratios of 0.3 and 0.5 from Fig. 7(a) are plotted in Fig. 7(b) with limiting shear strains estimated by Seed (11) for natural deposits, since a careful review on his study shows that what is called as "limiting shear strain" seems to be the maximum cyclic shear strain under such stress conditions. In the figure, the corrected SPT N_1-value for the USA is adjusted to the Japanese one by using Eq. (7). The maximum cyclic shear strains from Fig. 7(a) are in good agreement with those estimated by Seed (11) which indicates the possibility that the curves defined in Fig. 7(a) would serve as a basis for estimating the maximum cyclic shear strains to be developed for either any shear stress level or any SPT N_1-value.

The boundary curves in Fig. 7 suggest that if the cyclic shear stress slightly exceeds soil resistance of loose sand with SPT N_a-value of 5, more than 25% shear strain may develop. However, the liquefaction for dense sands is the cyclic mobility where shear strain is relatively insensitive to a little change in the cyclic stress. This fact indicates that one should adopt a fairly large factor of safety, shch as 1.5 or more, for loose sands as compared with dense sands.

Pore Pressure Ratio in Sands

Even though the pore pressure does not reach a value equal to the initial effective stress, sometimes it may be desirable to estimate the degree of pore pressure buildup due to a given earthquake. Seed et al (12) showed that the pore pressure ratio in saturated sand under undrained conditions could be expressed by

$$r_u = \frac{1}{2} + \frac{\sin^{-1}(2\,r_N^{\frac{1}{\alpha}} - 1)}{\pi} \qquad (10)$$

in which $r_u = u/\sigma_0'$, (u = excess pore water pressure), r_N = ratio of the number of cycles, N_i, to N_ℓ that corresponds to $r_u = 1$, and α is an empirical constant. Assuming that a relationship between shear stress amplitude and number of cycles to cause liquefaction is straight with a slope of $-\beta$ on a log-log plot, an equation in terms of safety factor, F_ℓ, can be drawn:

$$r_u = \frac{1}{2} + \frac{\sin^{-1}(2\,F_\ell^{\frac{1}{\alpha\beta}} - 1)}{\pi} \qquad (11)$$

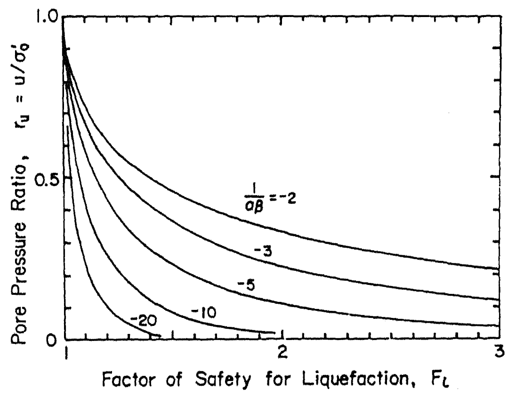

Fig. 8 Relationship between pore pressure ratio and safety factor

Typical relationships between r_u and F_ℓ are shown in Fig. 8 in terms of $1/\alpha\beta$. Because a represetative value of $1/\alpha\beta$ is about -5 to - 10 for loose to medium dense sands, pore pressure ratio may not exceeds 0.2 if F_ℓ is greater than 1.5.

CONCLUSIONS

The following conclusions may be made on the basis of a critical review of field performance of sandy deposits during several earthquakes together with laboratory tests on undisturbed samples of sands by freezing sampling:

(1) Sands with fines have much greater liquefaction resistance than clean sands with the same SPT N_1-value and this tendency increases with an increase in fines content.
(2) Liquefaction with catastrophic failure would not occur for clean sands with SPT N_1-value greater than 25, and this critical value decreases with increasing fines content.
(3) Soils containing more than 20 percent clay (finer than 5 μm) would hardly liquefy unless their plasticity indexes are low.
(4) Sands containing gravel particles seem to have less resistance to liquefaction than clean sands without gravel having the same SPT N-values, while soils classified as gravel appear to have behaved well during earthquakes.

Finally the following procedure for estimating soil liquefaction potential of level ground may be proposed based on the above conclusions:

(1) If clay content is greater than 20 percent, consider that the soil does not liquefy unless its plasticity index is low.
(2) Assume equivalent dynamic shear stress within a given site by Eq. (1).
(3) Correct N-values in terms of net energy delivered to the rod, if necessary.
(4) Estimate adjusted SPT N_a-values from Eqs. (4) and (9), and Table 3. The equations must be cautiously applied to soil containing gravels because the presence of gravels tends to increse SPT N-values without significant change in liquefaction resistance.
(5) Estimate soil liquefaction resistance by Fig. 7(a) or by Eq. (8).
(6) Compute the factor of safety against liquefaction by
$$F_\ell = (\tau_\ell/\sigma_o')/(\tau_d/\sigma_o')$$
(7) If the safety factor is greater than 1 and the dynamic shear stress is greater than the threshold shear stress under which no pore pressure can be generated in the soil, the maximum pore pressure may be estimated by Fig. 8.

REFERENCES

(1) Tokimatsu, K. and Yoshimi, Y. (1983): "Empirical correlation of soil liquefaction based on SPT N-value and fines content," to be published in Soils and Foundations.

(2) Gibbs, H. J. and Holtz, W. G. (1957): "Research on determining the density of sands by spoon penetration testing," Proc., 4th ICSMFE, London, Vol. 1, pp. 35-39.

(3) Yoshimi, Y. and Tokimatsu, K. (1983): "SPT practice survey and comparative tests," Soils and Foundations, Vol. 23, No. 3, pp. 105-111.

(4) Nishizawa, Y., et al (1980): "In situ tests and efficiency concerning SPT," Proc., Sounding Symp., JSSMFE, pp. 83-92 (in Japanese).

(5) Kovacs, W. D. and Salomone, L. A. (1982): "SPT hammer energy measurments," Jour. Geotech. Eng. Div., ASCE, Vol. 108, No. GT4, pp. 599-620.

(6) Seed, H. B., Idriss, I. M. and Arango, I. (1983): "Evaluation of liquefaction potential using field performance data," Jour. Geotech. Eng. Div., ASCE, Vol. 109, No. GT3, pp. 458-482.

(7) Finn, W. D. L. (1982): "Soil liquefaction studies in the People's Republic of China," Soil Mechanics - Transient and Cyclic Loads, Ch. 22, pp. 609-626, John Wiley & Sons, Ltd.

(8) Tokimatsu, K. and Yoshimi, Y (1981): "Field correlation of soil liquefaction with SPT and grain size," Proc., Int. Conf. on Recent Advances in Geotech. Earthquake Eng. and Soil Dynamics, St. Louis, Mo., Vol. 1, pp. 203-208.

(9) Yoshimi, Y., Kaneko, O. and Tokimatsu, K. (1983): "Cyclic shear characteristics of a dense sand sampled by in situ freezing," Proc. 18th Annual Meeting, JSSMFE, pp. 559-560 (in Japanese).

(10) Iwasaki, T., et al (1978): "A practical method for assessing soil liquefaction potential based on case studies at various sites in Japan," Proc., 2nd Int. Conf. on Microzonation, San Francisco, Vol. 2, pp. 885-896.

(11) Seed, H. B. (1979): "Liquefaction and cyclic mobility evaluation for level ground during earthquakes," Jour. Geotech. Eng. Div., ASCE, Vol. 105, No. GT2, pp. 201-255.

(12) Seed, H. B., Martin, P. P. and Lysmer, J. (1976): "Pore-water pressure changes during soil liquefaction," Jour. Geotech. Eng. Div., ASCE, Vol. 102, No. GT4, pp. 323-246.