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A METHOD FOR ESTIMATING UNDRAINED CYCLIC STRENGTH OF SANDY SOILS USING STANDARD PENETRATION RESISTANCES

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ABSTRACT

In evaluating liquefaction potential of saturated sandy deposits, it is necessary to estimate undrained cyclic shear strength of soils. On many occassions in actual design procedures, it is very convenient if engineers can evaluate undrained cyclic shear strength of sandy soils with use of ordinary engineering properties of soils and grounds such as grading properties of disturbed samples and blow counts (N-values) by standard penetration tests. For this purpose, available data of sand sampling, dynamic triaxial tests on undisturbed specimens, N-values by standard penetration tests and gradings were analysed and a correlation among dynamic shear strength, mean diameter D_{50} and $D_r^*=21\sqrt{N/(\sigma_v'+0.7)}$, in which N is the blow counts by standard penetration tests and σ_v' is the in situ effective overburden pressure in kg/cm^2, was obtained. This correlation is represented by a simplified equation. Using this equation, approximate dynamic shear strengths of reclaimed and alluvial sandy deposits can be easily estimated from N-values, σ_v' and D_{50}.

Key words: dynamic, earthquake resistant, grain size, laboratory test, liquefaction, penetration test, repeated load, sampling, sandy soil, sounding
IGC: D6/C3/C6

INTRODUCTION

In actual practical procedures for evaluation of liquefaction potential of sandy soil deposits, blow counts or N-values by standard penetration tests have often been utilized. Since standard penetration test is easy to conduct in situ, it is widely utilized in common engineering practices in Japan. At present, in designing most of civil engineering structures this test is generally conducted. Therefore, procedures to estimate liquefaction potential with use of N-values are convenient to practical engineers. However, it is also well-known that this test is rather crude to estimate precise soil properties.

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Therefore, for recent important structures to be constructed on liquefaction susceptible deposits, more sophisticated geological surveys are often conducted where undisturbed samples are secured and dynamic triaxial tests on undisturbed specimens are performed for precisely evaluating undrained cyclic shear strength. Since this procedure is rather expensive, it cannot be applied to the whole of a wide area of one big construction project. In this case, standard penetration tests should be performed for the whole area as a supplemental one. Also this procedure cannot be applied to a relatively small construction project.

Considering these situations, while there are much criticism to the standard penetration test, it is necessary and useful to develop a simplified empirical formula to estimate in situ dynamic shear strength in terms of N-values by standard penetration tests and other soil index properties for practical design purposes.

In this respect, it should be noted, however, that N-values are considerably sensitive to grain size. In general, N-values for clayey deposits are much smaller than those for sandy deposits and N-values for sandy deposits are also much smaller than those for gravelly deposits. From these facts, it can be anticipated that N-value of one soil may differ considerably from that of another soil even if the two soils have an equal dynamic strength. Therefore, it seems that if N-values are utilized without considering the effects of grain size on N-values, results of analyses can be quite misleading.

Reported herein is a method of evaluating undrained cyclic strengths of sandy soils in triaxial stress condition from standard penetration resistances N-values with taking into account the effects of grain size on N-values. From the dynamic shear strengths estimated by the method which is proposed in this paper, in situ dynamic shear strength for liquefaction potential analyses can be evaluated.

TWO METHODS EXAMINED IN THIS STUDY

In most of previous studies, in situ dynamic shear strengths of sands for liquefaction potential analyses were evaluated using relative densities which were in turn estimated from measured N-values and effective overburden pressure σ_v' (Seed and Idriss, 1967). This method is denoted as the A-method in Fig. 1. In the A-method, two different equations are utilized, those are (i) the relationship among N, σ_v', relative density D_r and other soil and ground parameters, and (ii) the relationship among undrained cyclic strength, σ_v', D_r and other soil parameters. This method will be firstly examined in this paper. In the

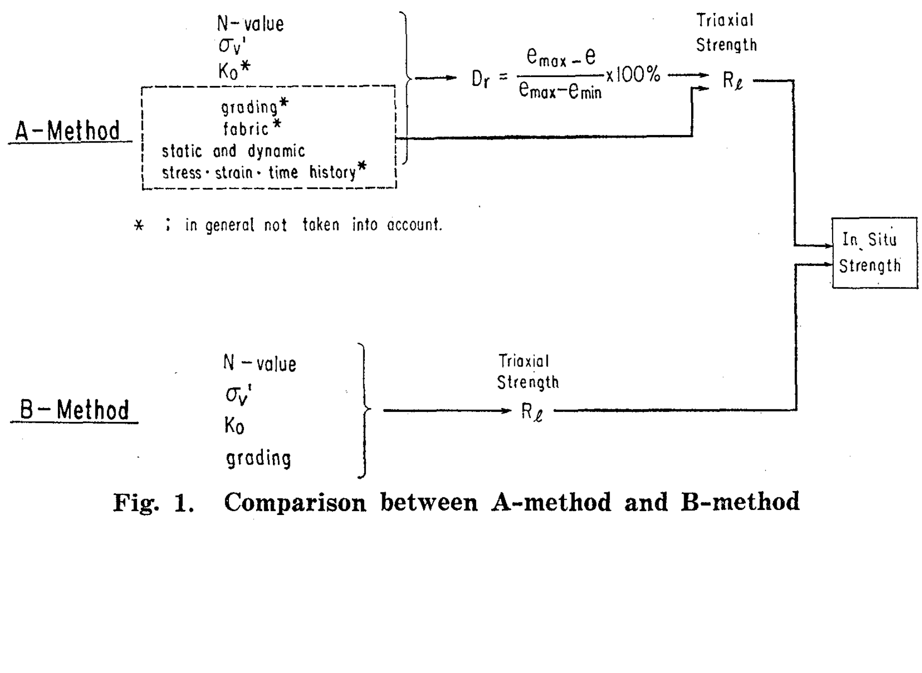

Fig. 1. Comparison between A-method and B-method

second method which is denoted as the B-method in Fig. 1, undrained cyclic strengths are directly evaluated from measured N-values, σ_v', K_0 and grading. In this method, a correlation equation among N-values, σ_v', K_0, grading and undrained cyclic strength is utilized. In this study, the second method was found to be more convenient and more precise than the A-method.

EXAMINATION OF A-METHOD

The most important factor which is firstly evaluated in the A-method is relative density D_r defined as

$$D_r=\frac{e_{\max}-e}{e_{\max}-e_{\min}}\times 100\ (\%) \tag{1}$$

in which e is the estimated in situ void ratio and $e_{\max}$ and $e_{\min}$ are the maximum and the minimum void ratios of the soil, respectively.

In order to estimate in situ dynamic shear strength from the values of relative density which have in turn been estimated from measured N-values, it is necessary that the following three points be confirmed.

(1) Relative density can be estimated from measured N-values within the limit of errors allowable for engineering purposes.

(2) The standard methods of measuring the minimum and the maximum densities of sands have been established.

(3) Dynamic shear strength can be estimated from estimated relative densities within the limit of errors allowable for engineering purposes.

To confirm item (1), a number of studies have been performed already. Among them, Meyerhof (1957) proposed a following empirical equation on the basis of laboratory tests using clean sands performed by Gibbs and Holtz (1957):

$$D_r^*=21\sqrt{N/(\sigma_v'+0.7)} \tag{2}$$

in which D_r^* is the estimated relative density (as distinguished from measured relative density D_r by Eq. (1)).

The almost identical one to Eq. (2) was utilized by Seed and Idriss (1967) in evaluating in situ relative densities of the sand deposits in Niigata city where soil liquefaction was observed widely during the Niigata Earthquake of 1964. Recently, Japanese Society of Soil Mechanics and Foundation Engineering conducted sand samplings at Kawagishi-cho (site H) using a modified Bishop-type sand sampler (JSSMFE, 1976; Bishop, 1948; Hanzawa and Matsuda, 1977). The liner had the inside diameter of 5.3 cm and the length of 65 cm. The liner was driven into the sand deposit with the position of the piston being fixed. For sampling, the liner was pulled up into the chamber filled with air, causing negative pore pressure and minimizing disturbance to the sample (see Table 1). In situ dry density γ_d was estimated by the equation

$$\gamma_d=\frac{W_s}{Al-\Delta V} \tag{3}$$

in which A, l and ΔV are inside cross-sectional area of the liner, the length of insertion of the liner and the volume of the lost sample and W_s is the dry weight of the secured sample. Void ratio determined by this procedure will be denoted as e_F hereafter. $e_{\min}$ was determined by tamping a mold using air-dry sand applying the pressure of 1 kg/cm^2 on the top. $e_{\max}$ was the average of two values by two methods (Kolbuszewski, 1948; Tanimoto, 1975). $e_{\min}$ and $e_{\max}$ of Toyoura Sand by these methods are 0.60 and 0.94, respectively. Figs. 2 and 3 show the comparisons between measured relative density D_r and estimated relative density D_r^* by Eq. (2). As Castro (1975) did, the standard penetration

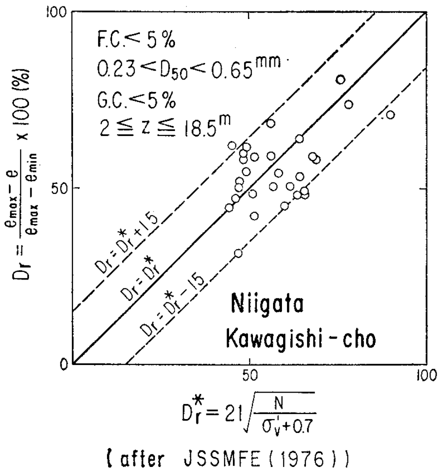

Fig. 2. D_r and D_r* relation by the data of JSSMFE (1976)

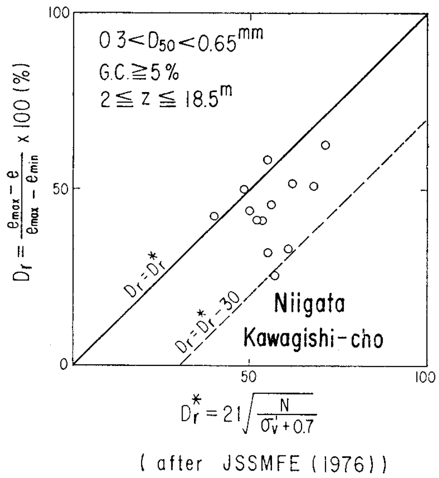

Fig. 3. D_r and D_r* relation by the data of JSSMFE (1976)

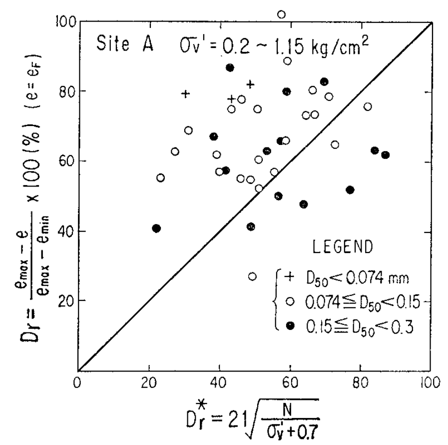

Fig. 4. D_r and D_r* relation of fine sands at site A

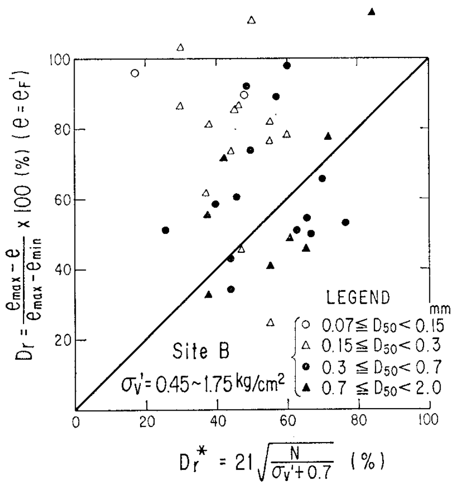

Fig. 5. D_r and D_r* relation of medium to coarse sands at site B

resistances corresponding to undisturbed samples which are referred to in this paper were obtained either in adjacent borings at the same elevation or directly above or below undisturbed samples in the same boring and the blow count (N-value) was not considered representative of the undisturbed sample if the soil description or grain size, or both, were different.

The sand deposit at Kawagishi-cho consists of relatively clean medium grain sized sand. While a scatter is observed in the data in Fig. 2 the correlation between D_r and D_r* is not bad. And it is evident that Eq. (2) may be used for such grounds if errors of 15 percent in relative density can be allowed. It is also worthy to note from Fig. 3 that for sand including 5 percent or more gravel, D_r* is larger than D_r. This means that for gravelly deposits, Eq. (2) is misleading. On the other hand, Fig. 4 shows another comparison between D_r and D_r* for fine sands whose D_{50} are smaller than 0.3mm. The method for sand sampling at this site (site A) is identical to that adopted by JSSMFE (1976). Site A along Tokyo Bay consists of hydraulically filled deposits and alluvial sandy deposits which are located under the hydraulic fills. $e_{\min}$ and $e_{\max}$ were determined by the method of Yoshimi and Tohno (1972). $e_{\min}$ and $e_{\max}$ of Toyoura Sand determined by the authors following the Yoshimi–Tohno method are 0.64 and 0.96, respectively (see Table 1). It can be seen in Fig. 4 that for these fine sands, Eq. (2) underestimates the relative densities and is quite misleading especially for sands with smaller grain size. It is to be noted that such underestimations in the relative densities of saturated fine sands by Eq. (2) as shown in Fig. 4 has also been indicated by the experimental data by Gibbs and Holtz (1957), while the date is not shown here.

Fig. 5 shows another comparison between D_r and $D_r{}^*$ at site B along Setonaikai Sea in Shikoku where sand samplings were performed with use of a twist sampler (Ogura et al., 1978). The surface deposit at site B is a hydraulic fill and beneath this there is an alluvial sandy deposit. The liner is 7.0 cm in inner diameter and 80 cm in length. This is a double tube thin wall sampler where sealing between the piston and the liner is designed to be perfect. Identically to the modified Bishop-type sand sampler, the position of the piston is kept fixed when the liner is driven into sand deposits. When driving is completed, the inner liner is pulled up 7 cm to make room between secured sample and sand deposit. Then only the inner liner is twisted with the outer liner being kept fixed. This is for the pre-equipped rubber tube to cover the bottom face of the secured sample by twisting. This rubber tube is equipped in advance at the bottom between the outer liner and the inner liner. With this procedure, secured sample is protected for dropping from the liner. Void ratios for estimation of in situ relative densities were averaged values in the liners before freezing. These void ratios will be denoted as e'_F hereafter. These liner and secured samples were frozen at the site with use of dry ice after pulling the apparatus to the ground surface as carefully as possible. e_{min} and e_{max} were determined by the Yoshimi–Tohno method. It can be seen in Fig. 5 that also for this site, Eq. (2) underestimates relative density for finer sands. Fig. 6 shows a summarized relationship between D_r-$D_r{}^*$ and D_{50} for several sites. Ishihara (1976, 1977) and Ishihara and Silver (1977) performed sand samplings at a hydraulic fill and an alluvium deposit beneath the hydraulic fill along Shinano River in Niigata city (site C) using a large diameter sand sampler (20 cm in inner diameter and 100 cm in inner height). Void ratios for estimation of in situ relative densities were measured for triaxial specimens which were confined by the vacuum pressure of $-0.2\,\mathrm{kg/cm^2}$. The values of void ratios by this procedure will be denoted as e_0 hereafter. e_{min} was determined by vibrating a mold using air-dry sand on a shaking table applying the pressure of $1\,\mathrm{kg/cm^2}$ on the top. e_{max} was determined by spooning air-dry sand into a mold. e_{min} and e_{max} of Toyoura Sand by these methods are 0.61 and 0.96, respectively. Ishizawa, Nakagawa and Kurokawa (1977) also performed large diameter sand samplings at an alluvium deposit in Tokyo (site D). The methods for determining e, e_{min} and e_{max} are identical to those for site C. At a site in a hydraulic fill named as Ohgishima in Yokohama city (site I), Saito (1977) performed sand samplings

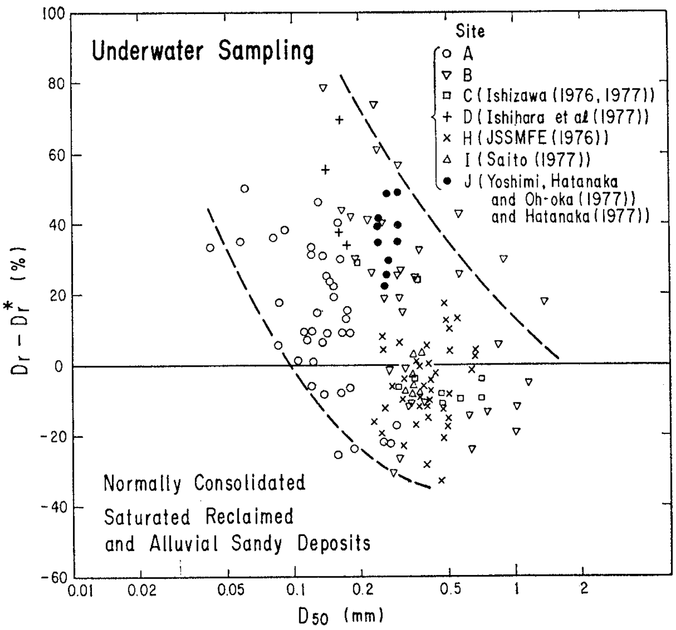

Fig. 6. D_r-$D_r{}^*$ and D_{50} relation (summarized)

Table 1. List of sand samplings and dynamic triaxial tests

(see NOTE)

Site	Sand Sampling	Void Ratio Measurement	e_{min} Measurement	e_{max} Measurement	Dynamic Triaxial Test Condition: σ_c' (kg/cm^2)	Frequency (Hz)	DA for R_l (%)	Specimen (ϕcm, hcm)	References
Site A (in Tokyo, along Tokyo Bay) Alluvial and Reclaimed	Modified Bishop-type sand sampler; the liner is 5.3 cmϕ and 65 cm$l$*)	e_F; estimated in situ void ratio by Eq. (3)	Tamping mold with air-dry sand**) (0.64)	Lightly pouring air-dry sand into mold**) (0.96)	$=\sigma_v'=$ 0.2~1.15	1.0	6	5ϕ, 10h	This Investigation *) Hanzawa and Matsuda (1977) **) Yoshimi and Tohno (1972)
Site B (in Shikoku, along Setonaikai Sea) Alluvial and Reclaimed	Twist sand sampler; the liner is 7 cmϕ and 80 cm$l$***)	e_F'; average void ratio in the liner before freezing	Identical to Site A		$=\sigma_v'=$ 0.45~1.75	0.5	5	7ϕ, 14h	This Investigation ***) Ogura et al. (1978)
Site C (in Niigata City, along Shinano River) Alluvial and Reclaimed	Large diameter sand sampling; the liner is 20 cmϕ and 100 cm$l$;	e_0; void ratio of thawed triaxial specimen at $\sigma_c'=$ 0.2 kg/cm^2	Vibrating mold using air-dry sand on shaking table with 1 kg/cm^2 being applied on the top (0.61)	Spooning air-dry sand into mold (0.96)	1.5 ($\sigma_v'=$ 0.3~1.15)	1.0	5	5ϕ, 10h	Ishihara (1976, 1977) Ishihara and Silver (1977)
Site D (in Tokyo) Alluvial	Identical to Site C				0.5 ($\sigma_v'=$ 0.265~0.610)	1.0	5	5ϕ, 10h	Ishizawa, Nakagawa and Kurohara (1977)
Site E (in Yokohama, along Tokyo Bay) Reclaimed	Identical to Site A	—	—	—	$=\sigma_v'=$ 0.37~0.77	1.0	6	5ϕ, 10h	This Investigation
Site F (in Tokyo, along Tokyo Bay) Alluvial	Thin-wall sampling with enough cares	—	—	—	0.5, 1.0 ($\sigma_v'=$0.24~0.65)	1.0	6	5ϕ, 10h	This Investigation
Site G (in Tokyo along Tokyo Bay) Alluvial and Reclaimed	Identical to Site A				$=\sigma_v'=$ 0.8~1.14	0.5	6	5ϕ, 10h	This Investigation
Site H (Kawagishicho, Niigata City) Alluvial and Reclaimed	Identical to Site A	e_F	Tamping mold using air-dry sand and applying pressure of 1 kg/cm^2 on the top (0.60)	Average by two method; Kolbuszewski (1948) and Tanimoto (1975) (0.94)	—	—	—	—	JSSMFE (1976)
Site I (Oh-gi Shima) Reclaimed	Identical to Site A	e_F	Identical to Site H		—	—	—	—	Saito (1977)
Site J (in Yokohama, along Tokyo Bay) Reclaimed	Frozen Column Method; making frozen 5 ml, 40 cmϕ large column in ground	Void ratio of frozen specimen	The Yoshimi-and-Tohno method (0.62)	(0.98)	—	—	—	—	Yoshimi, Hatanaka and Oh-oka (1977), Hatanaka (1977)

(NOTE) The numbers in ( ) represent the maximum and minimum void ratios of Toyoura Sand determined by each method.

using the modified Bishop-type sand sampler. The methods for determining e, e_{min} and e_{max} were identical to those for site H (Kawagishi-cho). And at a hydraulic fill along Tokyo Bay in Yokohama city (site J), a frozen column method was utilized by Yoshimi, Hatanaka and Oh-oka (1977) and Hatanaka (1977). In this method, a large scaled frozen column of sand, around 5m in length and around 40cm in diameter, was made in the ground and this was pulled out with a force of 5 tons or more to the ground. Small pieces of frozen specimen were cut from the large frozen column and their void ratios were measured with being kept frozen. It was confirmed by them from other basic experiments that void ratios determined by the method described above are almost identical to in situ values of void ratio. e_{min} and e_{max} were determined by the Yoshimi-Tohno method. e_{min} and e_{max} of Toyoura Sand by them are 0.62 and 0.98, respectively. It is seen from Fig. 6 that there is a general trend showing that D_r-D_r* decreases with the increase in D_{50}. Neverthless, a scatter shown in Fig. 6 is too large for Eq. (2) to be used in precisely evaluating relative densities of various sands with a large range of D_{50}. Especially for silty sands which include a large amount of fine soils, it is obvious that relative densities are usually underestimated by Eq. (2).

One of the reasons which cause a large scatter in the data shown in Fig. 6 may be variations in densities of sands during sampling and handling operations. Except the frozen column method by Yoshimi, Hatanaka and Oh-oka (1977), there is a possibility that loose sands density and dense sands loosen at the time of pushing liners into sand deposits (Marcuson, Cooper and Bieganousky, 1977). Neverthess, it is likely that these variations in densities are not a main reason for a large scatter in D_r-D_r* for the same value of D_{50}. This is because it was found by the authors that possible variations in densities due to sampling and handling were much less that the scatter in D_r-D_r* in Fig. 6. To present authors, It is likely that a main reason for a large scatter in D_r-D_r* is that N-values can be largely affected by other factors than D_r, σ_v' and grading. One of these factors may be the in situ earthpressure coefficient at rest K_0 (Saito, 1977). However, all samplings referred to in this study are performed at newly hydraulically reclaimed fills and alluvial deposits. Therefore, in this study, K_0 can be estimated to be around 0.5 on past experiences. Therefore, the variation in K_0 may not be the main reason for a large scatter in D_r-D_r*. Other factors affecting D_r-D_r* may include fabrics of soils, static and dynamic stress-strain-time histories, inhomogeneity of soil in a sampler or so. As further investigations are necessary to clarify the effects of these factors on N-values, it can be concluded that it is very difficult at present to estimate in situ relative density from standard penetration resistances.

As to item (2), the standard method of measuring the values of e_{max} and e_{min} have not been established in Japan so far. Several methods have been proposed by different researchers, and it is well-known that the values for silty sands depend on the method employed significantly. Therefore, it is difficult in Japan to determine uniquely the relative densities of silty sands even when samples are given. It can be seen from Table 1 that the values of e_{max} and e_{min} for an identical sand (Toyoura Sand) are different among different methods. This may be one of the reasons which cause a scatter in the data shown in Fig. 6.

Finally, as to item (3), it is necessary that the relationship between dynamic shear strength and relative density D_r be established for undisturbed specimens. For reconstituted specimens, Lee and Fitton (1969) and Seed and Idriss (1971) show that there is a variation in dynamic shear strength for an identical relative density with the variation in D_{50}. In this study, available data of undisturbed specimens were utilized to examine whether there is a correlation between dynamic shear strength and D_r as follows. Fig. 7 shows typical test results of undrained cyclic triaxial tests on undisturbed specimens which were obtained from one liner. These samples were obtained from site G which is close to site

A. The methods for sampling and determining e_{min} and e_{max} are identical to those for site A, and e is e_0. All of the tests referred to in this study were performed on specimens made from samples which were made frozen at the sites if the sample did not include a large amount of fine soils. This avoided disturbances caused during transportation from the site of sampling to the laboratory. Silty samples, however, were not frozen in order to avoid the disturbance due to volume expansion caused by freezing. In Fig. 7, $\sigma_{dp}/2\sigma_c'$ is stress ratio where σ_{dp} is dynamic axial stress in single amplitude, σ_c' is effective confining pressure at dynamic triaxial testing and N_c means the number of loading cycles at which the state of initial liquefaction or a certain value of dynamic axial strain amplitude is observed. In this study, the dynamic shear strength in dynamic triaxial tests is defined as

$$R_l = (\sigma_{dp}/2\sigma_c') \tag{4}$$

which is the stress ratio $(\sigma_{dp}/2\sigma_c')$ at the number of loading cycles $N_c=20$ where the amplitude of axial strain in double amplitude (DA) becomes 5 or 6 percent. The values of R_l used in this study were read from such figures as Fig. 7. In general, to obtain a value of R_l, three to six specimens obtained from one liner were tested. Of course, the definition

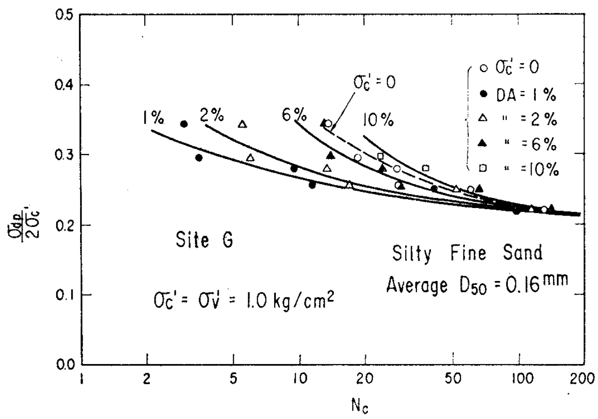

Fig. 7. Typical test result of dynamic triaxial test on undisturbed sandy specimens from site G

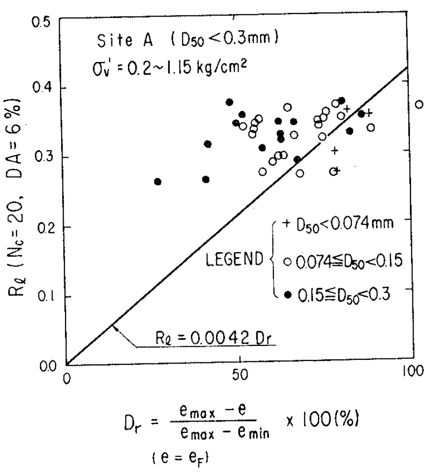

Fig. 8. R_l and D_r relation of undisturbed specimens from site A

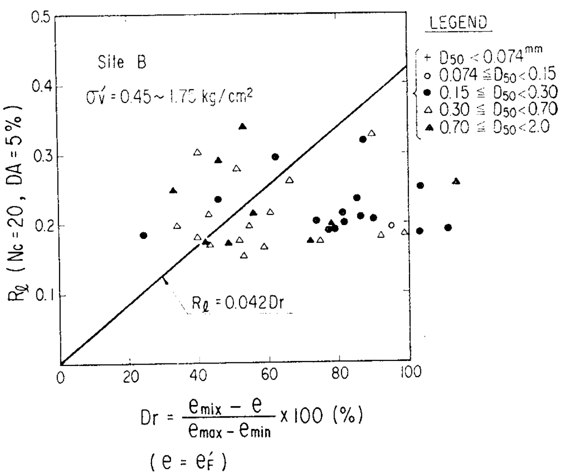

Fig. 9. R_l and D_r relation of undisturbed specimens from site B

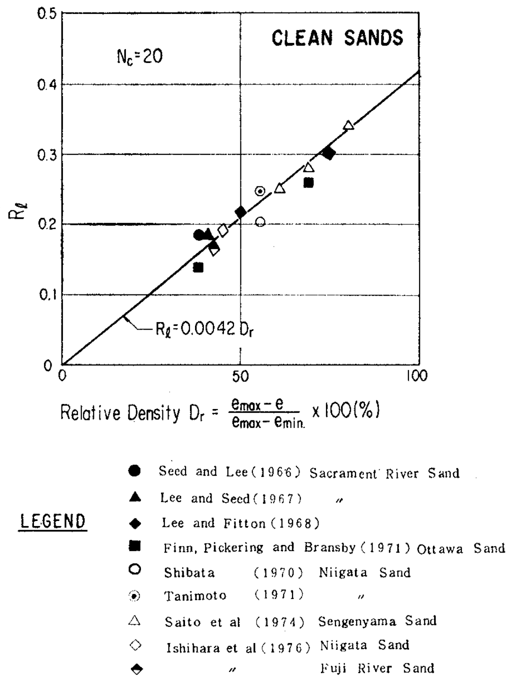

Fig. 10. Comparison of R_l and D_r for clean sands

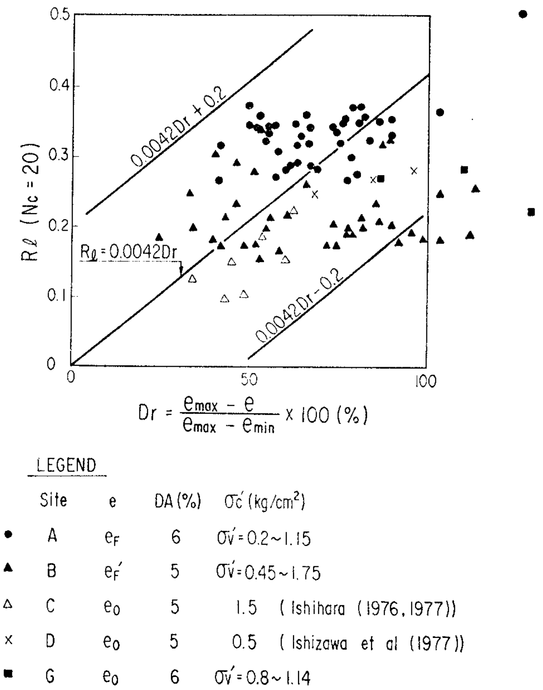

Fig. 11. R_l and D_r relation (summarized)

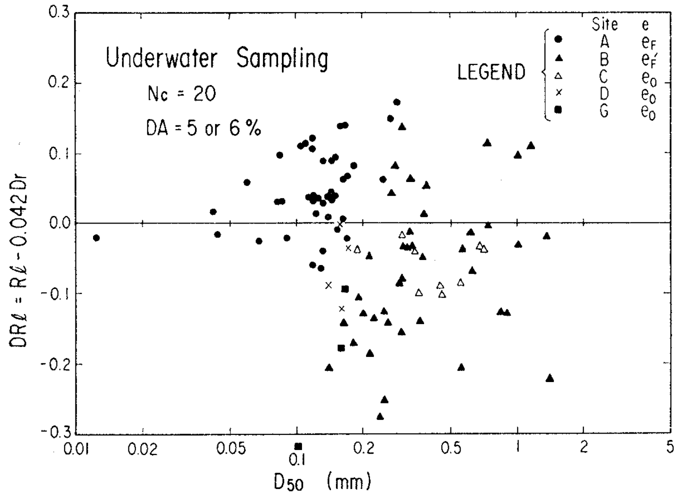

Fig. 12. $DR_l = R_l - 0.0042\ D_r$ and D_{50} relation

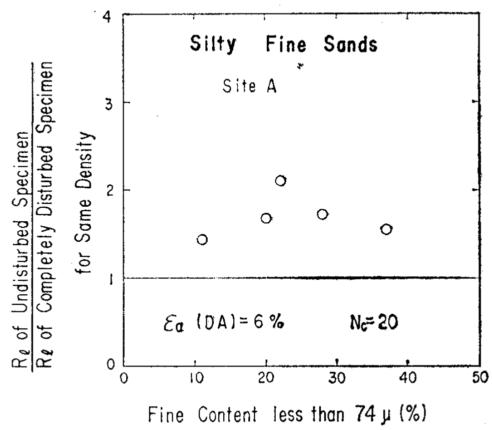

Fig. 13. Comparison of R_l of undisturbed specimens with reconstituted specimens from site A

of strength as a function of the number of loading cycles and amplitude of axial strain should depend on the purpose of the study; for this research, the definition of Eq. (4) was considered adequate. Effects of changes in N_c and in amplitude of axial strain will be considered in future studies. It was found that the difference of R_l between for $DA=5\%$ and for 6% is quite small (3% at largest). Therefore, R_l for $DA=5\%$ and R_l for $DA=6\%$ can be considered to be able to utilize for the same analysis. And in all of the tests referred to, the Skempton's B-values were larger than 0.96. Note that except the values of isotropical confining pressure σ_c' and frequencies of cyclic loading the dynamic triaxial tests referred to in this study were performed by the almost same method using the almost same apparatus. The frequency of cyclic loading for the specimens from the sites

A, C, D, E and F is 1.0Hz and that for the specimens from the sites B and G is 0.5Hz. (The data for the sites E and F will be shown later.) The triaxial specimens from site B are 7cm in diameter and 14cm in height and the others are 5cm in diameter and 10cm in height (see Table 1). Fig. 8 shows the relationship between R_l defined by Eq. (4) and measured in situ relative density D_r for site A. Fig. 9 is a similar one for site B. Obviously, there is not a high correlation between two values for both sites. The relationship shown by solid lines in Figs. 8 and 9

$$R_l = 0.0042 D_r \tag{5}$$

was derived from the data of reconstituted clean sands made by an almost identical method. This is shown in Fig. 10. It is seen from Fig. 10 that there is a rather unique relationship between strength R_l and relative density D_r for several clean sands. Eq. (5) was also proposed by Ishihara (1977) on the basis of Japanese data. Fig. 11 is the summary of the relationship between R_l and D_r for undisturbed specimens. It is obvious that there is no correlation among the data. To examine whether the variation in D_{50} is a main cause for a large scatter in the data in Fig. 11 or not, a parameter DR_l was defined as

$$DR_l = R_l - 0.0042 D_r \tag{6}$$

Fig. 12 shows the relationship between D_{50} and DR_l for the data shown in Fig. 11. It is seen from Fig. 12 that there is not a high correlation between DR_l and D_{50}. From Figs. 11 and 12, it is obvious that even if relative density could be estimated precisely, it is still difficult to estimate undrained cyclic triaxial strength from estimated relative density on the basis of the data shown in these figures. There may be several reasons for the large scatter in R_l in Fig. 11 or DR_l in Fig. 12. It was found that the differences in the methods of measuring e, $e_{\max}$, and $e_{\min}$ cause smaller variations in DR_l than the observed scatter in DR_l. For specimens from sites A and B, isotropical effective confining pressures σ_c' at cyclic tests were identical to in situ effective overburden pressure σ_v'. But for specimens from sites C and D, σ_c' was different from σ_v'. This may cause some variations in R_l. However, it is obvious that this is not a main cause for the scatter in DR_l. It is likely that a main cause for the large scatter in DR_l may be that undrained cyclic strength R_l can not be related uniquely with relative density. Fig. 13 shows the relationship between fine contents and the ratio of R_l of undisturbed specimens from site A to R_l of reconstituted specimens which were made from completely disturbed soils obtained from the undisturbed specimens. The reconstituted specimens were made by raining de-aired soil into a mold fulfilled with de-aired water and had the equal relative densities to those of undisturbed specimens. It is seen from Fig. 13 that the differences in R_l for equal densities between undisturbed specimens and reconstituted specimens are considerable large in this case. This has been also reported by Ishihara and Tanaka (1974), Seed, Mori and Chan (1975) and Mulilis, Mori, Seed and Chan (1977). This means that there are some other unknown factors causing a scatter in R_l and in DR_l. Ladd (1974, 1976) and Mulilis, Chan and Seed (1975) has reported that one of these reasons is the fabric of sand. They showed that dynamic shear strength of reconstituted sands are greatly affected by the method of sample preparation. The variation in dynamic shear strength due to the variation in fabric for the same density may also be possible in in situ dynamic shear strengths, as shown in Fig. 13. Therefore, it is evident that relative density is not a unique parameter which determines the dynamic shear strength of sand.

In summary, it is evident that with the present knowledges it is extremely difficult to estimate undrained cyclic strength R_l of various sands by the A-method using standard penetration resistances, σ_v' and other factors within the limit of errors allowable for engineering purposes.

A NEW SIMPLIFIED METHOD FOR EVALUATION OF UNDRAINED CYCLIC STRENGTH FROM STANDARD PENETRATION RESISTANCES (B-METHOD)

The B-method shown in Fig. 1 is a more direct method than the A-method as described below. First, it is logical that in situ dynamic shear strength is in general related to several factors such as N-values, overburden effective pressure, σ_v', lateral earth pressure $\sigma_h'=K_0\sigma_v'$, grading properties and strain or stress histories. As for parameter K_0, the value of 0.5 can be assumed for all deposits examined in this study. To account for the effects of σ_v' on N-values, Eq. (2) was adopted in correlating R_l with σ_v', N and gradings of sands. However, it should be noted that in these procedures, the value of D_r* does not necessarily mean relative density but represents some in situ condition of the soil. Fig. 14 shows the summary of the relationship between R_l and D_r*. Site E is close to site J in the same reclaimed land. Sand samplings were performed using the modified Bishop-type sand sampler. At site F in Tokyo, samples were taken carefully from an alluvium deposit using a thin wall sampler. Also shown is the line representing the equation

$$R_l=0.0042D_r^* \qquad (7)$$

This can be derived by substituting $D_r=D_r^*$ into Eq. (5). The data for $\sigma_c'=0.5$ to $2.5\,\mathrm{kg/cm^2}$ from Castro (1975) are included in Fig. 14 in which R_l for $N_c=20$, R_{l20} were converted from R_l for $N_c=10$, R_{l10} as $R_{l20}=R_{l10}/1.15$. Obviously, it is seen from Fig. 14 that there is no correlation between R_l and D_r*. Castro (1975) has reported that the liquefaction of laboratory samples extracted from zones of sand having a high penetration resistance is little better than that of samples extracted from zones of low penetration resistance.

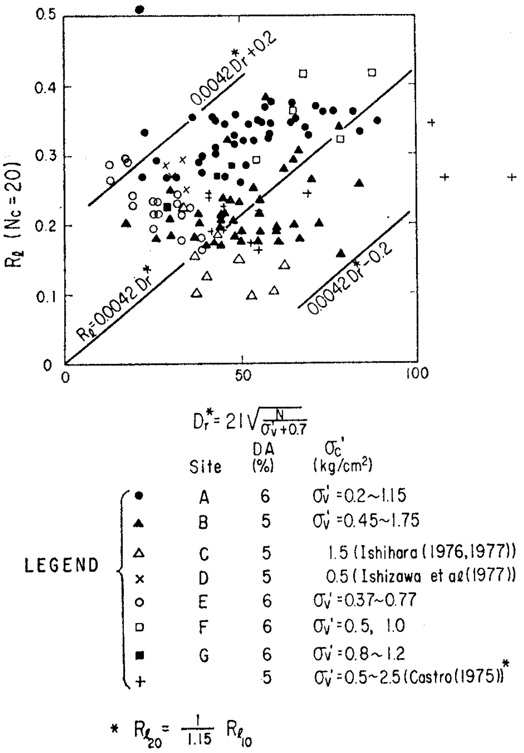

Fig. 14. R_l and D_r* relation (summarized)

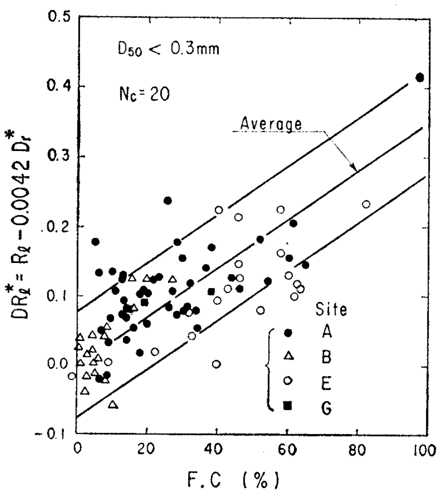

Fig. 15. $DR_l^*=R_l-0.0042D_r^*$ and fines content relation

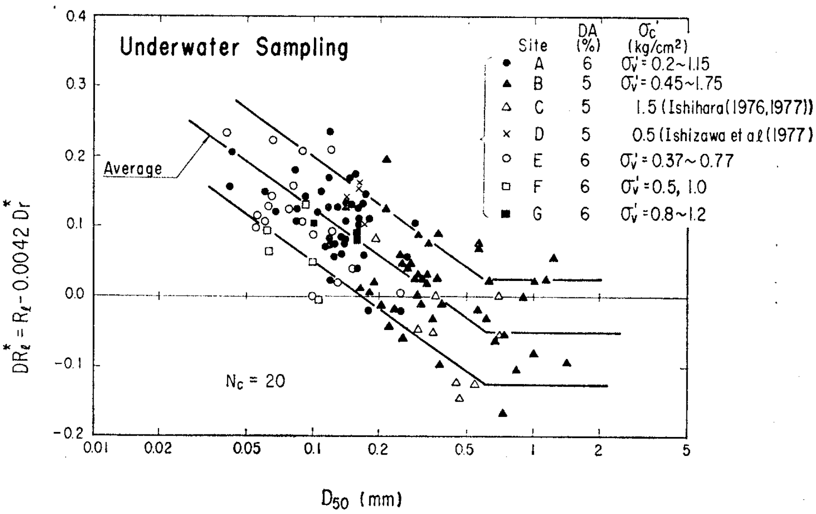

Fig. 16. $DR_l^*=R_l-0.0042D_r^*$ and D_{50} relation (summarized)

He suggested that this is due to a loosening of the dense sand during the sampling process. The data of such dense sands by Castro (1975) are shown by three points D_r^* of which are larger than 100. However, it can be assumed that the effects of such loosening may be relatively small for the sand deposits referred to in this study. This is because all the specimens referred to in this study were extracted from loose or medium sandy deposits which have D_r^* less than 100. Furthermore, it should be noted that D_r^* or N-values can be largely affected by grain size. This means that a large D_r^* or a large N-value may be caused by that the zones are gravelly and may not be caused by high density. Therefore, it can be anticipated that there can be a relatively high correlation among R_l, D_r^* and parameters which represent grading properties of sands. To find this correlation, a parameter was defined as

$$DR_l^* = R_l - 0.0042 D_r^* \tag{8}$$

in which R_l is measured dynamic strength by Eq. (4) and D_r^* is measured value by Eq. (2). Note that Eq. (8) is analogous to Eq. (6). Fig. 15 shows the relationship between DR_l^* and fines content FC for fine sands D_{50} of which are smaller than 0.3mm (Oh-hashi, Iwasaki and Tatsuoka, 1978). It can be seen from Fig. 15 that there is a good correlation between DR_l and FC. The average line can be represented by

$$DR_l^* = 0.0035 FC \tag{9}$$

in which FC is fines content in percentage. From Eqs. (8) and (9),

$$R_l = 0.0042 D_r^* + 0.0035 FC \tag{10}$$

For fine sands D_{50} of which are smaller than 0.3mm, approximate values of R_l can be estimated from D_r^* and FC using Eq. (10). For a wider range of D_{50}, FC is not a good parameter enoughly representing grading properties of sands. The mean diameter D_{50} can be a more general parameter than FC. Fig. 16 shows the summary of the data available at present. It can be seen from Fig. 16 that for a wide range of D_{50}, there is a high correlation between DR_l^* and D_{50}. The average line drawn in Fig. 16 appears to be a reasonable representation of the relationship between DR_l^* and D_{50}. This line was determined to fit the data as well as possible, but not to be too complicated compared with their scatter. Especially for D_{50} larger than 0.6mm, a constant value of DR_l^* was considered appropriate. This average line can be represented by

$$\left.\begin{array}{lll} & DR_l^* = -0.225\log_{10}(D_{50}/0.35) & \text{for } 0.04 \leqq D_{50} \leqq 0.6\,\text{mm} \\ \text{and} & DR_l^* = -0.05 & \text{for } 0.6 \leqq D_{50} \leqq 1.5\,\text{mm} \end{array}\right\} \tag{11}$$

From Eqs. (8) and (11),

$$\left.\begin{array}{lll} & R_l = 0.0042 D_r^* - 0.225\log_{10}\left(\dfrac{D_{50}}{0.35}\right) & \text{for } 0.04 \leqq D_{50} \leqq 0.6\,\text{mm} \\ \text{and} & R_l = 0.0042 D_r^* - 0.05 & \text{for } 0.6 \leqq D_{50} \leqq 1.5\,\text{mm} \end{array}\right\} \tag{12}$$

in which $D_r^* = 21\sqrt{N/(\sigma_v' + 0.7)}$. For D_{50} ranging from 0.04 to 1.5mm, Eq. (12) can be available to estimate approximate dynamic shear strength R_l using D_r^* and D_{50}. It can be noted that for the same value of D_r^*, R_l increases with the decrease in D_{50} in Eq. (12). This means that if Eq. (7) is used to estimate R_l from D_r^*, R_l can be underestimated for finer sands. Therefore, it can be pointed out from the facts shown in the above that if liquefaction potentials are estimated directly from N-values without taking into account grading properties of a sand, liquefaction potential can be overestimated for finer sand. Eq. (12), which can be considered to be one of the best ones which fit the data available at present, has an advantage over the B-method as follows. In Eq. (12), factors affecting undrained cyclic strength such as fabrics of sands, static and dynamic stress-strain-time histories or so other than relative density can be considered to have been taken into account for in a

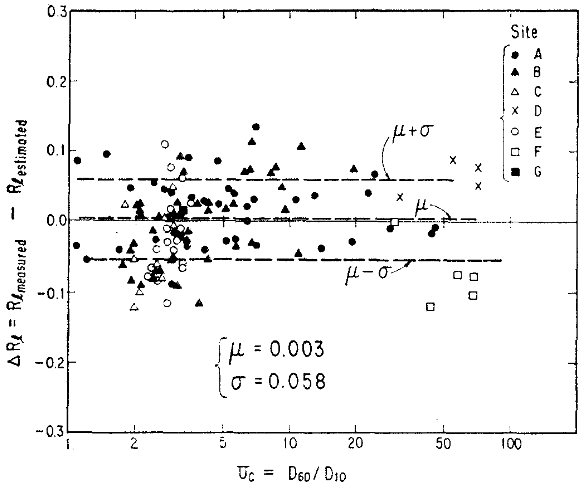

Fig. 17. ΔR_l and U_c relation

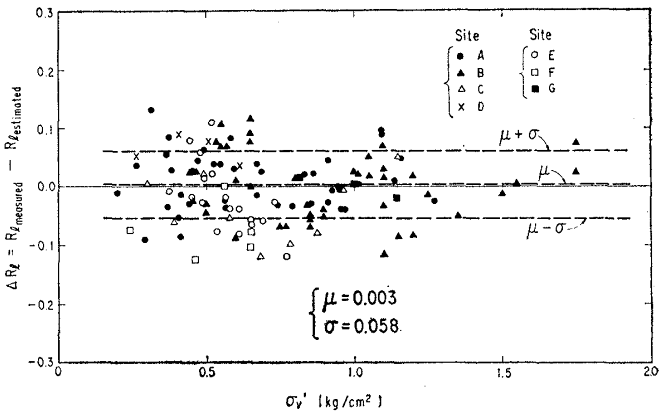

Fig. 18. ΔR_l and σ_v' relation

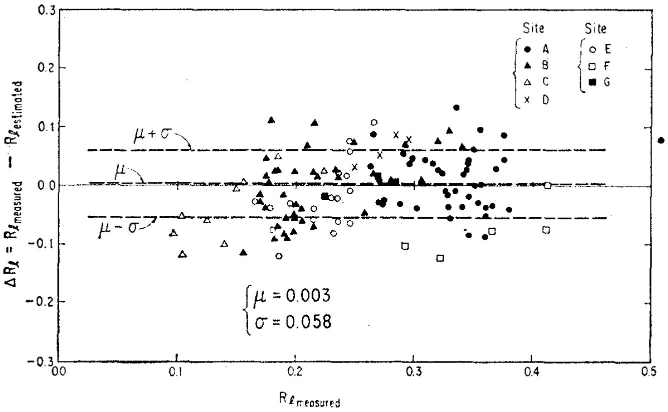

Fig. 19. ΔR_l and $R_{l\,\text{measured}}$ relation

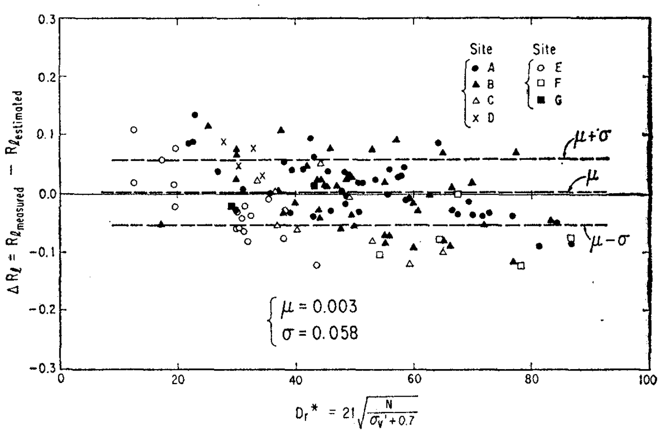

Fig. 20. ΔR_l and D_r^* relation

simple manner. This is because these factors also affect standard penetration resistances in the similar manner to undrained cyclic strength (Seed, 1976). This makes the B-method considerably simpler than the A-method.

To examine the validity of Eq. (12) with the data from which Eq. (12) was derived, a parameter was defined as

$$\Delta R_l = R_{l\,\text{measured}} - R_{l\,\text{estimated}} \tag{13}$$

in which $R_{l\,\text{measured}}$ is measured dynamic shear strength defined by Eq. (4) and $R_{l\,\text{estimated}}$ is estimated dynamic shear strength by Eq. (12). The average value μ of ΔR_l for all the data used in this analyses, the number of which is 123, is 0.003 and the standard deviation σ of ΔR_l for all the data is 0.058. The small value of μ of 0.003 means that Eq. (12) is adequate for all the data used in the study. And it can also be pointed out that when Eq. (12) is used, some errors in estimated R_l can be involved. Further investigations are necessary to account for this uncertainty in evaluating liquefaction potential. Fig. 17 shows the relationship between ΔR_l the uniformity coefficient $U_c = D_{60}/D_{10}$ which was not taken into account in deriving Eq. (12). On the basis of the data shown in Fig. 17, it may be concluded that there is not a high correlation between ΔR_l and U_c and that the effects of U_c on the correlation among R_l, D_r^* and grading properties are relatively small compared with D_{50}. Figs. 18, 19 and 20 show the relationships between ΔR_l and σ_v', $R_{l\,\text{measured}}$ and D_r^*, respectively. In these figures, high correlations can not be observed.

This means that Eq. (12) is rather homogeneous for σ_v' from 0.2 to 1.7kg/cm^2, for R_l from 0.15 to 0.4 and for D_r^* from 15 to 80. To obtain in situ dynamic strength from triaxial strength R_l defined by Eq. (4), some corrections are necessary. This problem is beyond the scope of this paper.

CONCLUSIONS

On the basis of the data from sand sampling procedures and dynamic triaxial tests on undisturbed specimens, a new simple method for evaluation of dynamic shear strength of sands from N-values by standard penetration tests and D_{50}-values was proposed. This is represented by Eq. (12). This equation can be effective for normally consolidated reclaimed and alluvial deposits for σ_v' ranging from 0.2 to 1.7kg/cm^2 and for D_{50} ranging from 0.04 to 1.5mm. With this equation dynamic shear strengths are evaluated higher for finer sands for the same value of $D_r^*=21\sqrt{N/(\sigma_v'+0.7)}$. This accords with past experiences with standard penetration tests. Another point to be noted in this method is that relative density $D_r=(e_{max}-e)/(e_{max}-e_{min})\times 100(\%)$ is not used. This extremely reduces uncertainties in evaluating strength from N-values, σ_v' and other soil parameters. With refining sampling and soil testing methods and with increasing the amount of data available, Eq. (12) will be modified. But the principal form of Eq. (12) may not be changed.

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NOTATION

DA = double axial strain amplitude in dynamic triaxial tests
D_r = relative density $=(e_{max}-e)/(e_{max}-e_{min})\times 100(\%)$
$D_r^*=21\sqrt{N/(\sigma_v'+0.7)}$
D_{50} = mean diameter (mm)
$DR_l=R_l-0.0042D_r$
$DR_l^*=R_l-0.0042D_r^*$
$\Delta R=R_{l\,measured}-R_{l\,estimated}$
FC = fines content (%)
N-value = blow counts by the standard penetration test
N_c = number of cyclic loading in dynamic triaxial test
R_l = dynamic shear stength in dynamic triaxial test $=(\sigma_{dp}/2\sigma_c')$ at $N_c=20$ and for $DA=5$ or 6%
U_c = uniformity coefficient $=D_{60}/D_{10}$
e = void ratio
e_F = in situ void ratio from γ_d by Eq. (3)
e_F' = mean void ratio of unfrozen sand in the liner of the sand sampler
e_0 = void ratio of thawed triaxial specimens confined by the vacuum pressure of -0.2kg/cm^2
e_{max}, e_{min} = maximum and minimum void ratios

σ=standard deviation
σ_v'=in situ effective overburden stress (kg/cm^2)
σ_c'=isotropic effective confining stress in dynamic triaxial test (kg/cm^2)
μ=mean value
σ_{dp}=dynamic axial stress in sigle amplitude in dynamic triaxial test

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