earthquake motions were represented and estimated as

$$(\tau/\sigma_v')_{av} = 0.7 \cdot \alpha \cdot \sigma_v/(g \cdot \sigma_v') \tag{1a}$$

by Castro (1975) and

$$(\tau/\sigma_v')_{av} = 0.65 \cdot \alpha \cdot \sigma_v \, r_d/(g \cdot \sigma_v') \tag{1b}$$

by Seed, Mori and Chan (1975) and Seed (1976).
In Equations (1a) and (b) $(\tau/\sigma_v')_{av}$ means average dynamic shear stress ratio, α is the maximum acceleration at the ground surface (estimated or recorded), g is the acceleration of gravity, σ_v is total overburden pressure, σ_v' is effective overburden pressure, and r_d is a reduction factor accounting for the elasticity of soils. The resistance of soil against liquefaction is estimated by a normalized standard penetration value, N' by Castro (1975) and N_1 by Seed, Mori and Chan (1975), using the relations.

$$N' = 3.5 \, N/(\sigma_v' + 0.7) \tag{2a}$$

$$N = \{1 - 1.25 \log_{10}(\sigma_v'/\sigma_1)\} \, N \tag{2b}$$

in which N is measured standard penetration resistance, σ_v' is effective overburden pressure in kg/cm^2 and σ_1= 1 ton/ft^2 (1.076 kg/cm^2). They found that while some overlapping can be seen between the two types of observations, in general, the data points of liquefaction occur above the data points of no-liquefaction. Presumably, the boundary between the two observations corresponds to the critical state where the shear strength is approximately equal to the shear stress. Therefore, using this boundary and Equation (2a) or (2b), a Ncr - $(\tau/\sigma_v')_{av}$ relation can be derived. In this method, the effects of groundwater level and magnitude of earthquake motions on liquefaction potential are taken into account. In this method, strength of soils is evaluated directly from normalized standard penetration resistences N' or N_1. It should be noted, however, that N-values are considerably sensitive to soil properties, especially 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 soil properties, especially grain size, on N-values, results of analysis can be quite misleading.

Reported herein is a method of evaluating undrained cyclic strengths of sandy soils in triaxial stress condition from standard penetration resistences, 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 Dr and other soil and ground parameters, and (ii) the relationship among undrained cyclic strength, σ_v', Dr and other soil parameters. This method will be firstly examined in this paper. In the 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_o and grading. In this method, a correlation equation among N-values, σ_v', K_o, 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 Dr defined as

$$Dr = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\ (\%) \tag{3}$$

in which e is the estimated in situ void ratio and e_{max} and e_{min} are the maximum and minimum void ratios of the soil, respectively. In order to estimate in situ dynamic shear strength from 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 the following empirical equation on the basis of laboratory tests using clean sands performed by Gibbs and Holtz (1957):

$$D_r^* = 21\sqrt{N/(\sigma_v' + 0.7)} \quad (4)$$

in which D_r^* is the estimated relative density (as distinguished from measured relative density D_r by Equation (3)). Needless to say, Equation (4) well fits the experimental data obtained by Gibbs and Holtz (1957) as shown in Fig. 2. The almost identical one to Equation (4) was utilized by Seed and Idriss (1967) in evaluating in situ relative densities of the sand deposits at Niigata city where soil liquefaction was observed widely during the Niigata Earthquake of 1964. And it was reported in their paper that the procedure for predicting liquefaction from estimated dynamic shear stresses and undrained cyclic strengths estimated from relative densities which were estimated using N-values and σ_v' gave a satisfactory agreement with actual liquefaction experiences at Niigata city. Recently Japanese Society of Soil Mechanics and Foundation Engineering conducted sand samplings at Kawagishi-cho using a reformed Bishop-type sand sampler (JSSMFE(1976), Bishop (1948), Hanzawa and Matsuda (1977)). The liner had an inside diameter of 5.3 cm and a length of 65 cm. The liner was driven into the sand deposit with the position of the piston fixed. For sampling, the liner was pulled up into the chamber filled with air, causing negative pore pressure and minimizing disturbance to the sample. In situ dry density γ_d was estimated by the equation

$$\gamma d = \frac{Ws}{Al - \Delta V} \quad (5)$$

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 Ws is the dry weight of the secured sample. The methods for estimation of the maximum and minimum void ratios are shown in Table 1. Void ratio determined by this procedure will be denoted as e_F hereafter. Figs. 3 and 4 show the comparisons between measured relative density D_r and estimated relative density D_r^* by Eq. (4). As Castro (1975) did, the standard penetration resistances corresponding to undisturbed samples which are referred 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 blowcount (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 consist of relatively clean medium grain sized sand. While a scatter is observed in Fig. 3, the correlation between D_r and D_r^* is not bad. And it is evident that equation (4) 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. 4 that for sand including 5 percent or more gravel, D_r* is larger than D_r. This means that for gravelly deposits, equation (4) is misleading. These results seem to support the methodology adopted by Seed and Idriss (1967) in evaluating relative density from N-values at Niigata city. On the other hand, Fig. 5 shows another comparison between D_r and D_r* for fine sands whose D_{50} are smaller than 0.3 mm. The method for sand sampling at this site (site A) is identical to that adopted by JSSMFE (1976) (see Table 1). Site A consists of hydraulically filled deposits and alluvial sandy deposits which are located under hydraulic fill. The methods for estimation of e_{max} and e_{min} are described in Table 1. It can be seen in Fig. 5 that for these fine sands, Equation (4) 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 Equation (4) as shown in Fig. 5 has also been indicated by the experimental data by Gibbs and Holtz (1957). Fig. 6 shows the relationship between the difference D_r - D_r* and mean diameter D_{50} from their data. It can be also seen from Fig. 6 that for saturated fine sand Equation (4) underestimates relative density.

Fig. 7 shows another comparison between D_r and D_r* at site B where sand samplings were performed with use of a twist sampler (Ogura et al 1978). The surface deposit at site B is hydraulic fill and beneath this there is 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 reformed 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 measured for triaxial specimens which were confined by the vacuum pressure of-0.3 kg/cm^2. These specimens were made by cutting frozen samples pushed out from the liner. 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. The values of void ratio by this procedure will be denoted as e_o hereafter. The methods for measurements of e_{max} and e_{min} for site B are also described in Table 1. It can be seen in Fig. 7 that also for this site, Equation (4) underestimates relative density for finer sands. Fig. 8 shows a summarized relationship between D_r -

D_r* and D_{50} for several sites. The methods of sampling and measurements of e, e_{max} and e_{min} adopted for these sites are listed in Table 1. The method of sand sampling at site I is identical to those at sites A and H, but at sites C and D a large diameter sand sampler was utilized by Ishihara (1976, 1977), Ishihara and Silver (1977) and Ishizawa, Nakagawa and Kurohara (1977). And at 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 5 m in length and around 40 cm 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 as above are almost identical to in situ values of void ratio. It is seen from Fig. 8 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. 8 is too large for Equation (4) 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 Equation (4).

One of the reasons which cause a large scatter in the data shown in Fig. 8 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 densify and dense sands loosen at the time of pushing liners into sand deposits (Marcuson, Cooper and Bieganousky (1977)). Nevertheless, 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 that possible variations in densities due to sampling and handling are much less that the scatter in D_r - D_r* in Fig. 8. To present authers, 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 earth-pressure coefficient at rest K_o (Saito (1977). However, all samplings referred in this study are performed at newly hydraulically reclaimed fills and alluvial deposits. Therefore, in this study, K_o can be estimated to be around 0.5 on past experiences. Therefore, the variation in K_o 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 not possible to determine uniquely the relative density of silty sands in Japan even when samples are given. It can be seen in 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. 8.

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. 9 shows typical test results of undrained cyclic triaxial tests on undisturbed specimens which were obtained from one liner. All of the tests referred in this study were preformed 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. 9, $\sigma_{dp}/2\sigma_c'$ 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

$$R_1 = (\sigma_{dp}/2\sigma_c') \tag{6}$$

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_1 used in this study were read from figures such as Fig. 9. In general, to obtain a value of R_1, three to six specimens obtained from one liner were tested. Of course, the definition 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 Equation (6) 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_1 between DA=5% and for 6% is quite small (3% at largest) and that R_1 for DA=5% and R_1 for DA=6% can be considered to be able to utilized for the same analyses. And in the all of the tests referred, 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 in this study were performed by the almost same method using the almost same apparatus. The other specifications employed are listed in Table 1. Fig. 10 shows the relationship between R_1 defined by Equation (6) and measured in situ relative density D_r for site A. Fig. 11 is a similar one for site B. Obviously, there is not a high correlation between two values for both sites. The relationship shown in Figs. 10 and 11

$$R_1 = 0.0042\ D_r \tag{7}$$

was derived from the data of reconstituted clean sands which are shown in Fig. 12. The references for the data are listed in Table 2. It is seen from Fig. 12 that there is rather unique relationship between strength R_1 and relative density D_r for several clean sands. Equation (7) was also proposed by Ishihara (1977) on the basis of Japanese data. Fig. 13 is the summary of the relationship between R_1 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. 13 or not, a parameter DR_1 was defined as

$$DR_1 = R_1 - 0.0042\ D_r \tag{8}$$

Fig. 14 shows the relationship between D_{50} and DR_1 for the data shown in Fig. 13. It is seen from Fig. 14 that there is not high correlation between DR_1 and D_{50}. From Fig. 13 and 14, 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_1 in Fig. 13 or DR_1 in Fig. 14. It was found that the differences in the methods of measuring e, e_{max}, and e_{min} cause smaller variations in DR_1 than the observed scatter in DR_1. 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' are different from σ_v'. This may cause some variations in R_1. However, it is obvious that this is not a main cause for the scatter in DR_1. It is likely that a main cause for the large scatter in DR_1 may be that undrained cyclic strength R_1 can not be related uniquely with relative density. Fig. 15 shows the relationship between fine contents and the ratio of R_1 of undisturbed specimens from site A to R_1 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 density with that of undisturbed specimens. It is seen from Fig. 15 that the differences of R_1 for equal density between undisturbed specimens and reconstituted specimens are considerable large for 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_1 and in DR_1. 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 insitu dynamic shear strengths, as shown in Fig. 15. Therefore, it is evident that relative density is not 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_1 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, $\sigma_h'=K_o\sigma_v'$, grading properties and strain or stress histories. As for parameter, K_o, the value of 0.5 may be assumed for all deposits examined in this study. To account for the effects of σ_v' on N-values, Equation (4) was adopted in correlating R_1 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. 16 shows the summary of the correlation where the relationship between R_1 and D_r^* is presented. Also shown is the line representing the equation

$$R_1 = 0.0042D_r^* \tag{9}$$

This can be derived by substituting $D_r = D_r^*$ into Equation (7). The data for $\sigma c' = 0.5$ to $2.5\ kg/cm^2$ from Castro (1975) are included in Fig. 16 in which R_1 for $N_c = 20$, $R_{1_{20}}$ were converted from R_1 for $N_c = 10$, $R_{1_{10}}$ by

$$R_{1_{20}} = \frac{1}{1.15} R_{1_{10}} \tag{10}$$

The value of 1.15 were determined on the basis of the experimental data of this study. Obviously, it is seen from Fig. 16 that there is no correlation between R_1 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. He suggested that this is due to a loosing 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 in this study. This is because all the specimens refered 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_1, D_r* and parameters which represent grading properties of sands. To find this correlation, a parameter was defined as

$$DR_1^* = R_1 - 0.0042\ D_r^* \tag{11}$$

in which R_1 is measured dynamic strength by Equation (6) and D_r* is measured value by Equation (4). Note that Equation (11) is analogous to Equation (8).
Fig. 17 shows the relationship between DR_1* and fine material content FC for fine sands D_{50} of which are smaller than 0.3 mm (Oh-hashi, Iwasaki and Tatsuoka (1978)). It can be seen from Fig. 17 that there is a good correlation between DR_1* and FC. The average line can be represented by

$$DR_1^* = 0.0035\ FC \tag{12}$$

in which FC is fine material content in percentage. From Equations (11) and (12),

$$R_1 = 0.0042 D_r^* + 0.0035\ FC \tag{13}$$

For fine sands D_{50} of which are smaller than 0.3 mm, approximate values of R_1 can be estimated from D_r* and FC using Equation (13). 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. Figs. 18 and 19 shows the relationships between DR_1* and D_{50} for fine sands from sites A and E, respectively. It is seen from these figures that there is also a high correlation between DR_1* and D_{50}. Fig. 20 shows that there is also a high correlation for medium fine to coarse sands from site B. Fig. 21 shows the summary of the data available at present. It can be seen from Fig. 21 that for a wide range of D_{50}, there is a high correlation between DR_1^* and D_{50}. The average line drawn in Fig. 21 appears to be a reasonable representation of the relationship between DR_1^* and D_{50}. This line was determined to be fit the data as well as possible, but not to be too complicated compared with their scatter. Especially for D_{50} larger than 0.6 mm, the constant value of DR_1^* was considered appropriate. This average line can be represented by

$$\left.\begin{array}{ll} DR_1^* = -0.225 \log_{10}(D_{50}/0.35) & \text{for } 0.04 \leqq D_{50} \leqq 0.6 \text{ mm} \\ \text{and} & \\ DR_1^* = -0.05 & \text{for } 0.6 \leqq D_{50} \leqq 1.5 \text{ mm} \end{array}\right\} \quad (14)$$

From Equations (11) and (14)

$$\left.\begin{array}{ll} R_1 = 0.0042\, D_r^* - 0.225 \log_{10}\left(\frac{D_{50}}{0.35}\right) & \text{for } 0.04 \leqq D_{50} \leqq 0.6\text{mm} \\ \text{and} & \\ R_1 = 0.0042\, D_r^* - 0.05 & \text{for } 0.6 \leqq D_{50} \leqq 1.5 \text{ mm} \end{array}\right\} \quad (15)$$

in which $D_r^* = 21\sqrt{N/(\sigma v' + 0.7)}$.

For D_{50} ranging from 0.04 to 1.5 mm, Equation (15) can be available to estimate approximate dynamic shear strength R_1 using D_r^* and D_{50}. It can be noted that for the same value of D_r^*, R_1 increases with the decrease in D_{50} in Equation (15). This means that if equation (9) is used to estimate R_1 from D_r^*, R_1 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 sands. Equation (15), 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 equation (15), 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 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 Equation (15) with the data from which Equation (15) was derived, a parameter was difined as

$$\Delta R_1 = R_{1\,measured} - R_{1\,estimated} \quad (16)$$

in which $R_{1\,measured}$ is measured dynamic shear strength defined by Equation (6) and $R_{1\,estimated}$ is estimated dynamic shear strength by Equation (15). The average value μ of ΔR_1 for all the data used in this analyses, the number of which is 123, is 0.003 and the standard deviation σ of ΔR_1 for all the data is 0.058. The

small value of μ of 0.003 means that Equation (15) is adequate for all the data used in the study. And it can also be pointed out that when Equation (15) is used, the errors in estimated R_1 can be 0.058 at least. Further investigations are necessary to account for this uncertainty in evaluating liquefaction potential.
Fig. 22 shows the relationship between ΔR_1 and uniformity coefficient $U_c = D_{60}/D_{10}$ which was not taken into account in deriving Equation (15). On the basis of the data shown in Fig. 22, it may be concluded that there is not a high correlation between R_1 and U_c and that the effects of U_c on the correlation among R_1, D_r* and grading properties are relatively small compared with D_{50}. Figs. 23, 24 and 25 show the relationship between ΔR_1 and $\sigma v'$, $R1_{measured}$ and D_r*, respectively. In these figures, high correlations can not be observed. This means that equation (15) is rather homogeneous for σ_v' from 0.2 to 1.7 kg/cm^2, R_1 from 0.15 to 0.4 and D_r* from 15 to 80.

To obtain in situ dynamic shear strength from R_1 defined Equation (6), some corrections are necessary. This problems is beyond the scope of this paper. Needless to say, for a specific liquefaction potential analyses it is better to perform sand samplings and dynamic shear tests on undisturbed specimens. However, in evaluating liquefaction potential of wide and/or inhomogeneous area, the information from standard penetration tests has to be utilized besides the sophisticated geological surveys. Equation (15) may be a good guide to estimate approximate dynamic shear strengths from N-values and gradings of disturbed specimens.
It is likely that some disurbances during sampling operations and handling during soil testings may affect the values of coefficients in equations (12), (13), (14) and (15). And it is also noted that the amount of the available data at present is limited. Therefore, it can be anticipated that these equations will be modified with refining sampling and testing methods and with increasing data available.

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 Equation (15). This equation can be effective for normally consolidated reclaimed and alluvial deposits for σ_v' ranging from 0.2 to 1.7 kg/cm^2 and for D_{50} ranging from 0.04 to 1.5 mm. 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 $D_r = (e_{max} - e) / (e_{max} - e_{min})$ x

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, Equation (15) will be modified. But the principal form of Equation (15) 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_1 = ; $R_1 - 0.0042D_r$

DR_1^* = $R_1 - 0.0042D_r^*$

ΔR = $R1_{measured} - R1_{estimated}$

FC ; fine content (%)

N-value ; blow counts by the standard penetration test

Nc ; number of cyclic loading in dynamic triaxial test

R1 ; dynamic shear strength in dynamic triaxial test = $(\sigma_{dp} / 2\sigma_c')$ at Nc=20 and for DA=5 or 6%

Uc ; unifomity coefficient = D_{60}/D_{10}

e ; void ratio

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 single amplitude in dynamic triaxial test

$(\frac{\tau}{\sigma_v'})_{av}$; average dynamic shear stress ratio by earthquake motion

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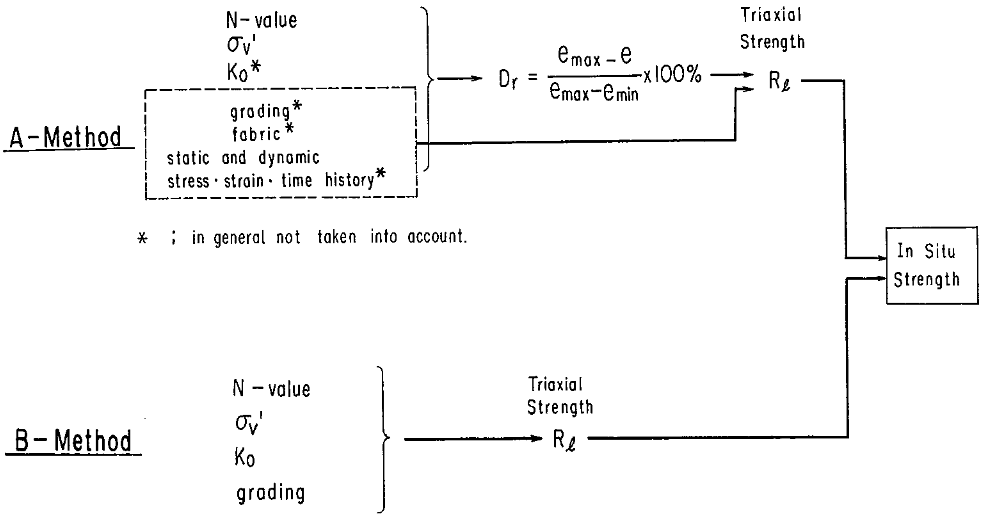
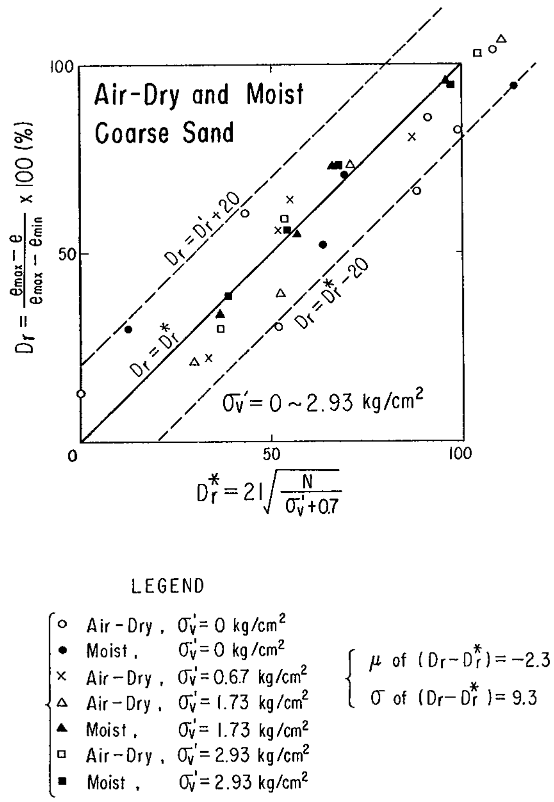
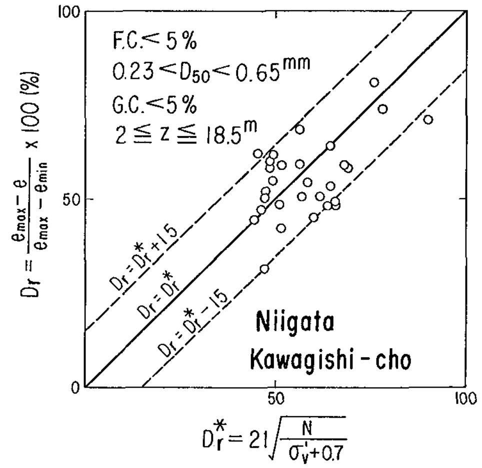


Fig.1. Comparison between A-Method and B-Method



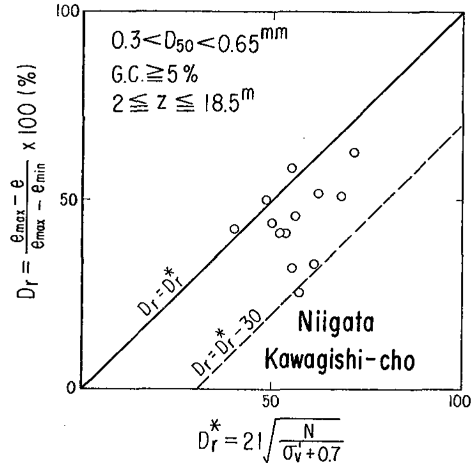
(after Fig.4 of Gibbs and Holtz (1957))

Fig.2. Dr and Dr* Relation by the Data of Gibbs and Holtz (1957)



(after JSSMFE (1976))

Fig.3. Dr and Dr* Relation by the Data of JSSMFE (1976)



(after JSSMFE (1976))

Fig.4. Dr and Dr* Relation by the Data of JSSMFE (1976)

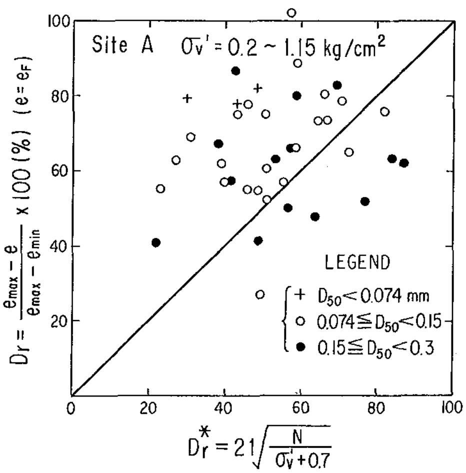


Fig.5. Dr and Dr* Relation of Fine Sands at Site A

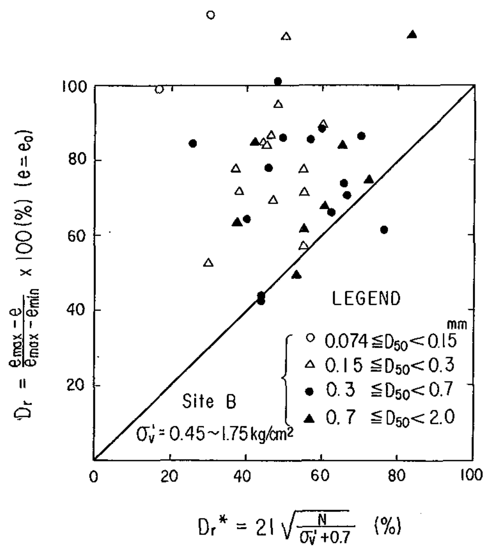


Fig.7. Dr and Dr* Relation of Medium to Coarse Sands at Site B

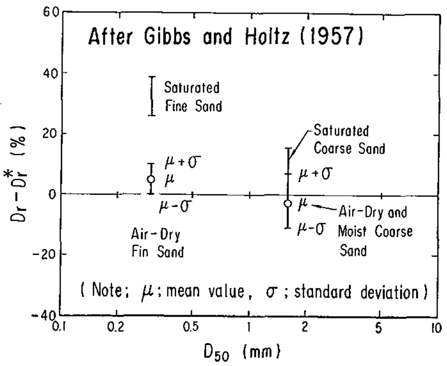


Fig.6. Dr-Dr* and D50 Relation by the Data of Gibbs and Holtz (1957)

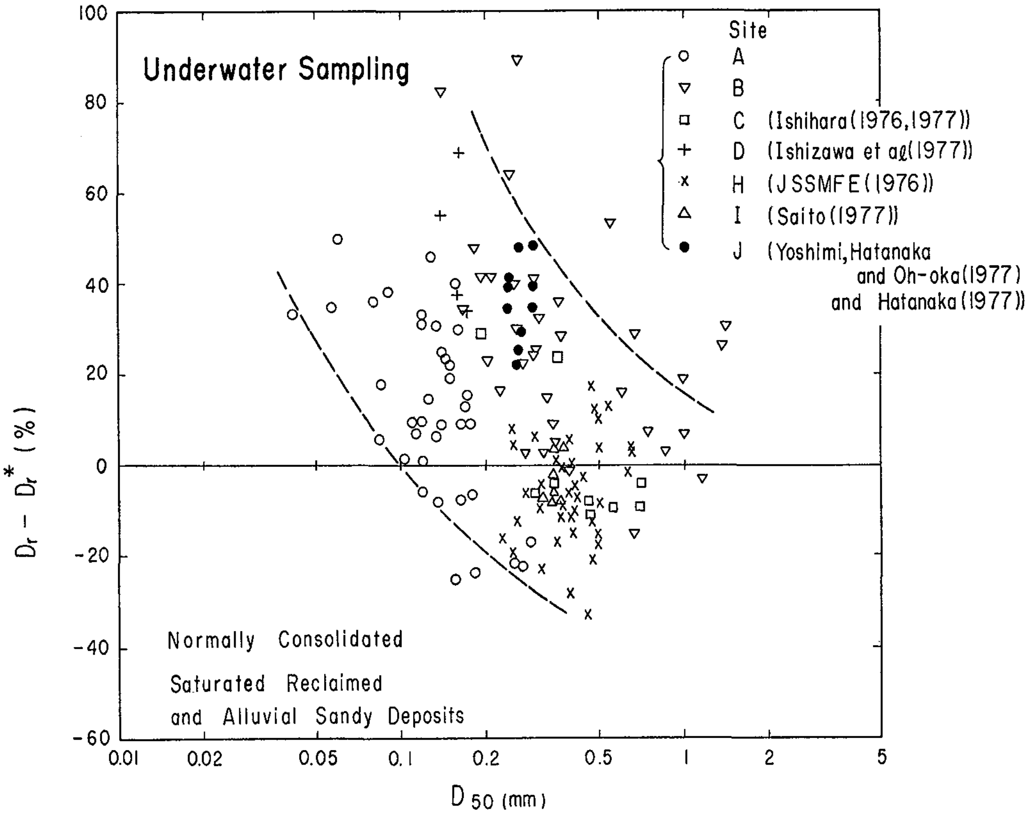


Fig.8. Dr-Dr* and D50 Relation (summarized)

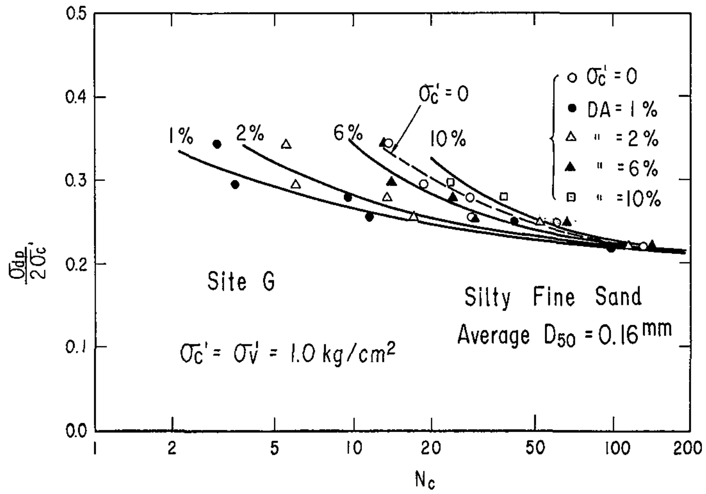


Fig.9. Typical Test Result of Dynamic Triaxial Test on Undisturbed Sandy Specimens from Site G

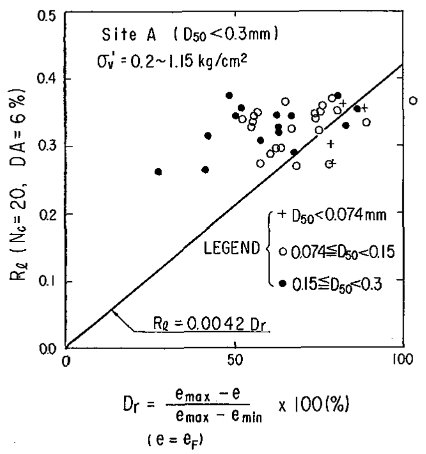


Fig.10. R_l and Dr Relation of Undisturbed Specimens from Site A

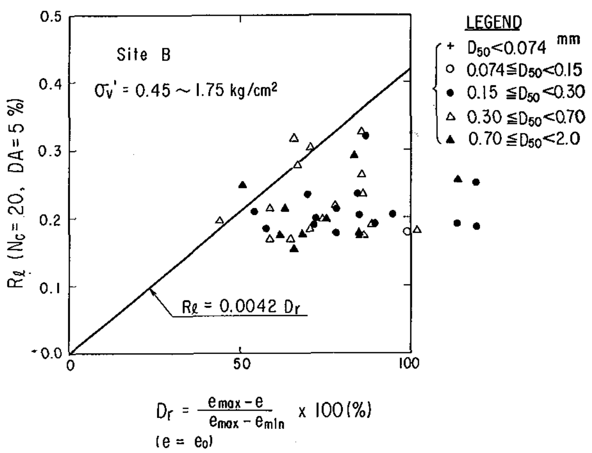


Fig.11. R_l and Dr Relation of Undisturbed Specimens from Site B

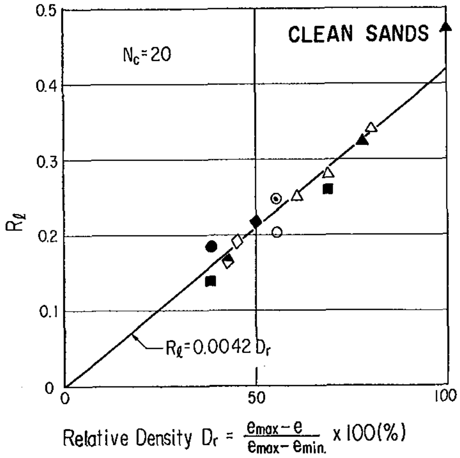


Fig.12. R_l and Dr Relation of Reconstituted Clean Sands (see Table 2 for Legend)

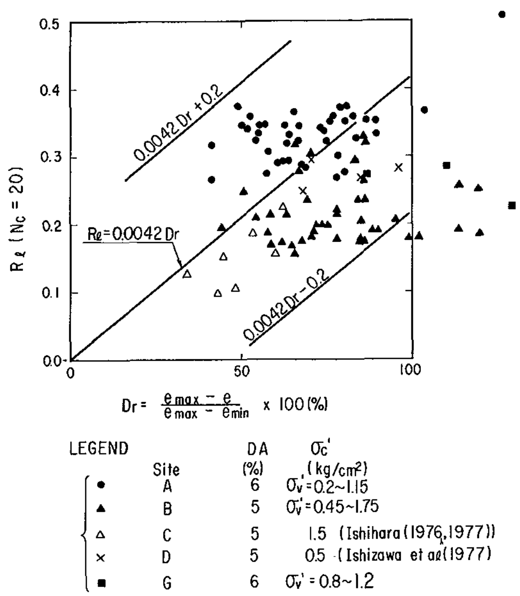


Fig.13. R_l and Dr Relation (summarized)

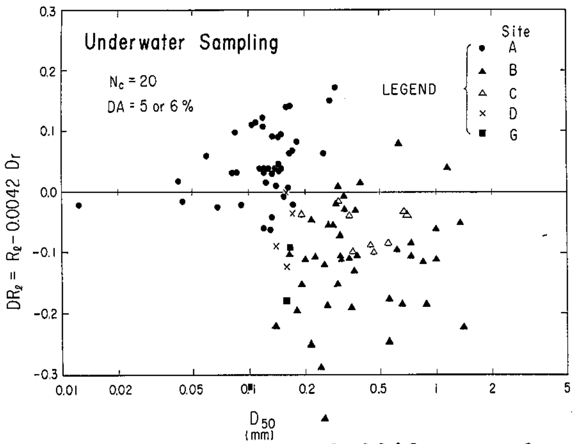


Fig.14. $DR_1 = R_1 - 0.0042$ Dr and D_{50} Relation

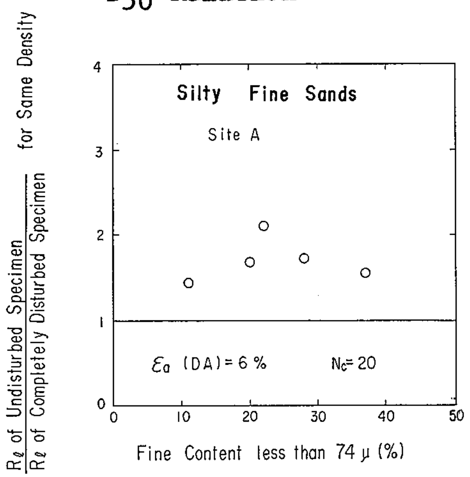
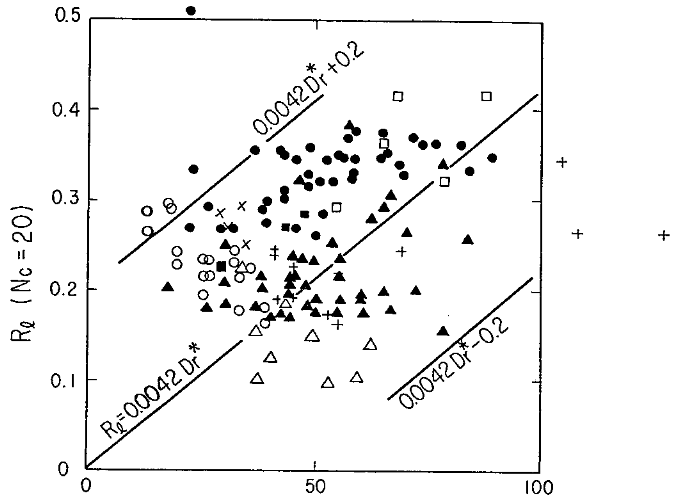


Fig.15. Comparison of R_1 of Undisturbed Specimens with Reconstituted Specimens From Site A



$$D_r^* = 21\sqrt{\frac{N}{\sigma_v' + 0.7}}$$

LEGEND

	Site	DA (%)	σ_c' (kg/cm²)
●	A	6	$\sigma_v' = 0.2 \sim 1.15$
▲	B	5	$\sigma_v' = 0.45 \sim 1.75$
△	C	5	1.5 (Ishihara (1976, 1977))
×	D	5	0.5 (Ishizawa et al (1977))
○	E	6	$\sigma_v' = 0.37 \sim 0.77$
□	F	6	$\sigma_v' = 0.5,\ 1.0$
■	G	6	$\sigma_v' = 0.8 \sim 1.2$
+		5	$\sigma_v' = 0.5 \sim 2.5$ (Castro (1975))*

$$* \quad R_{\ell_{20}} = \frac{1}{1.15} R_{\ell_{10}}$$

Fig.16. R_1 and Dr^* Relation (summarized)

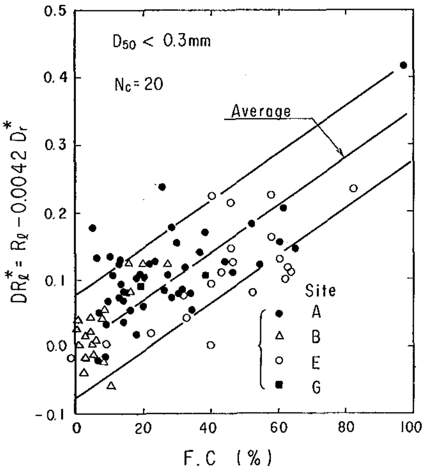


Fig.17. $DR_1* = R_1 - 0.0042\ Dr*$ and Fine Content Relation

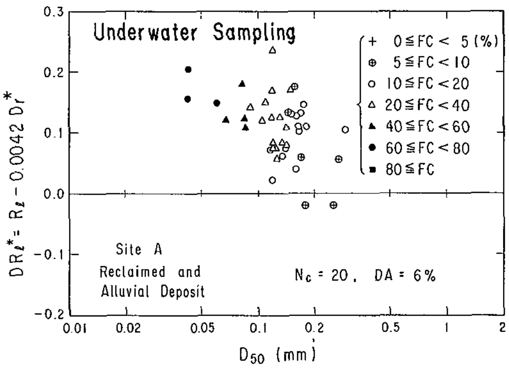


Fig.18. $DR_1* = R_1 - 0.0042\ Dr*$ and D_{50} Relation for Site A

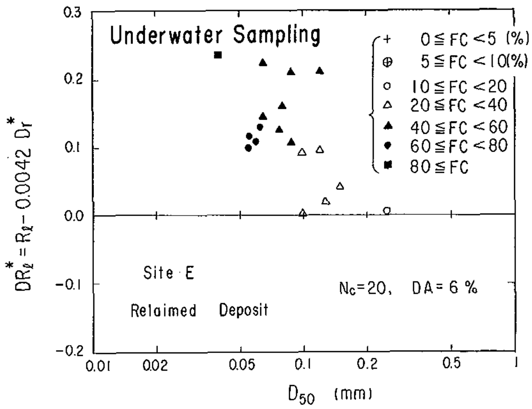


Fig.19. $DR_1* = R_1 - 0.0042\ Dr*$ and D_{50} Relation for Site E

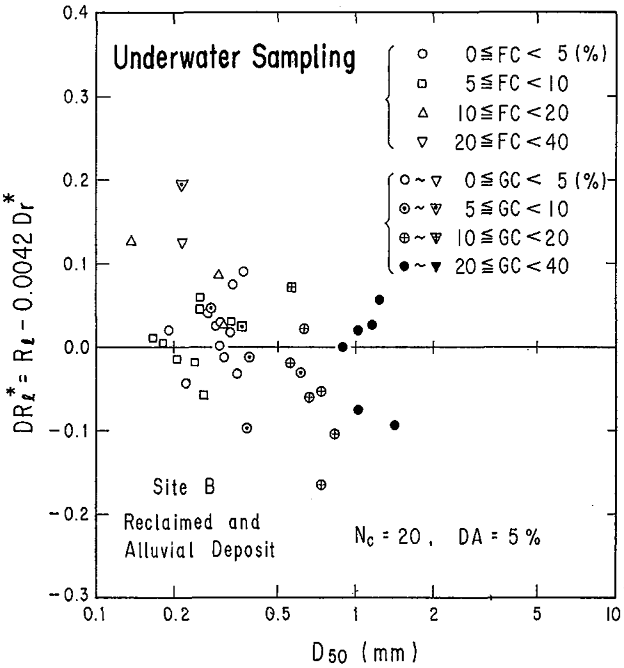


Fig.20. $DR_1^* = R_1 - 0.0042\ Dr^*$ and D_{50} Relation for Site B

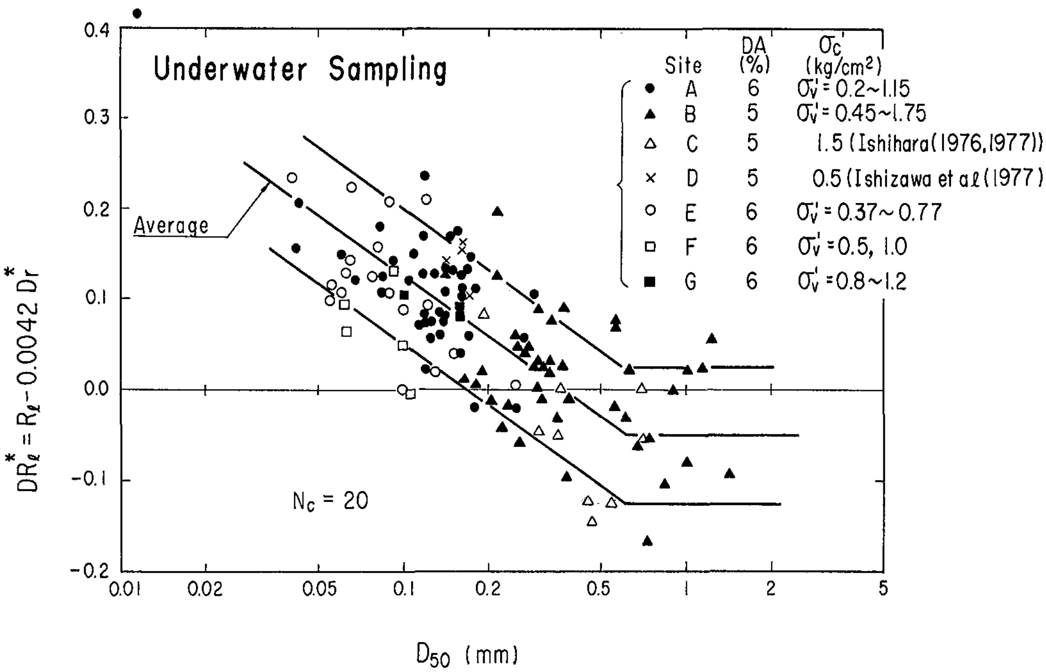


Fig.21. $DR_1^* = R_1 - 0.0042\ Dr^*$ and D_{50} Relation (summarized)

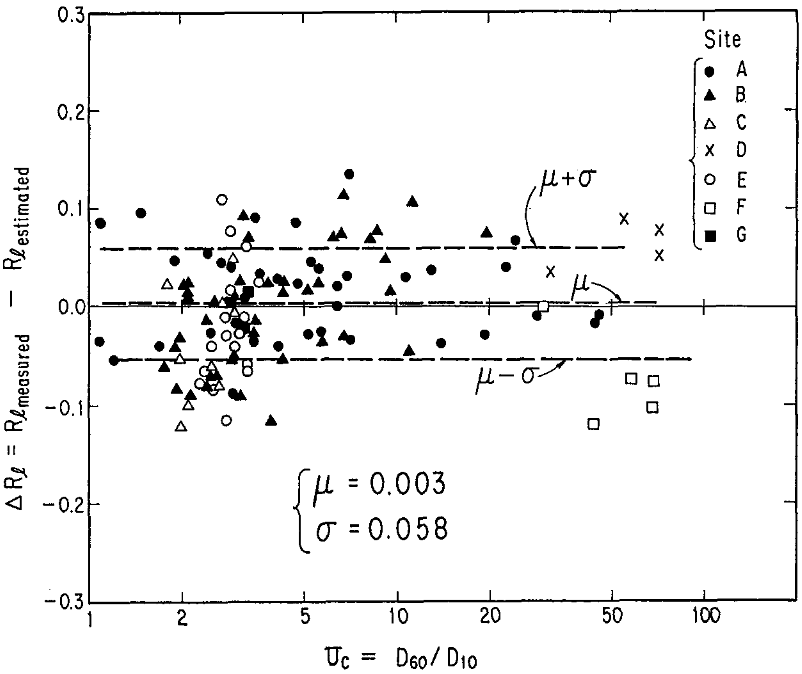


Fig.22. ΔR_l and U_c Relation

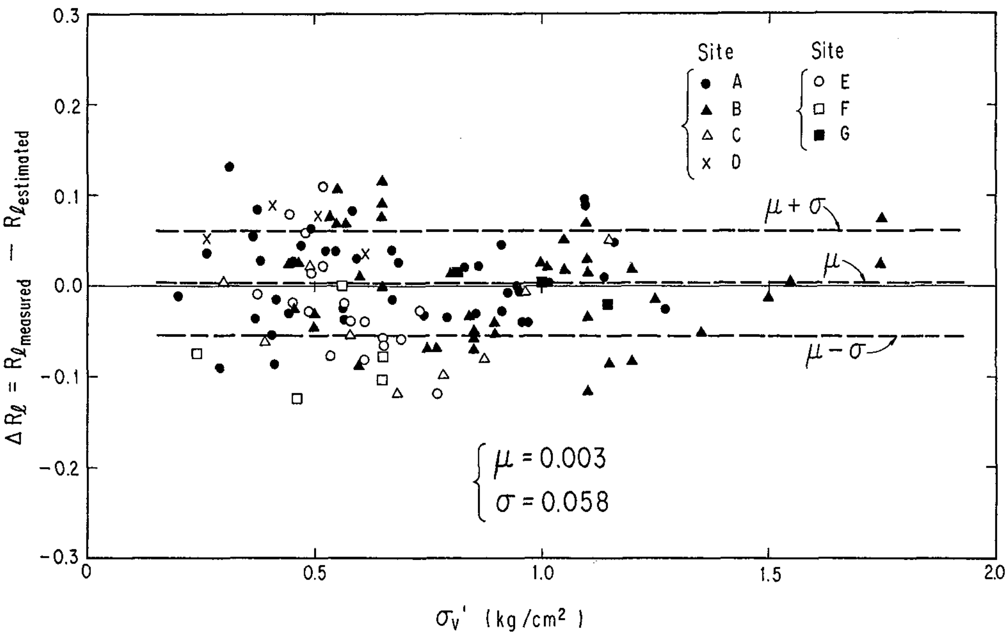


Fig.23. ΔR_l and σ_v' Relation

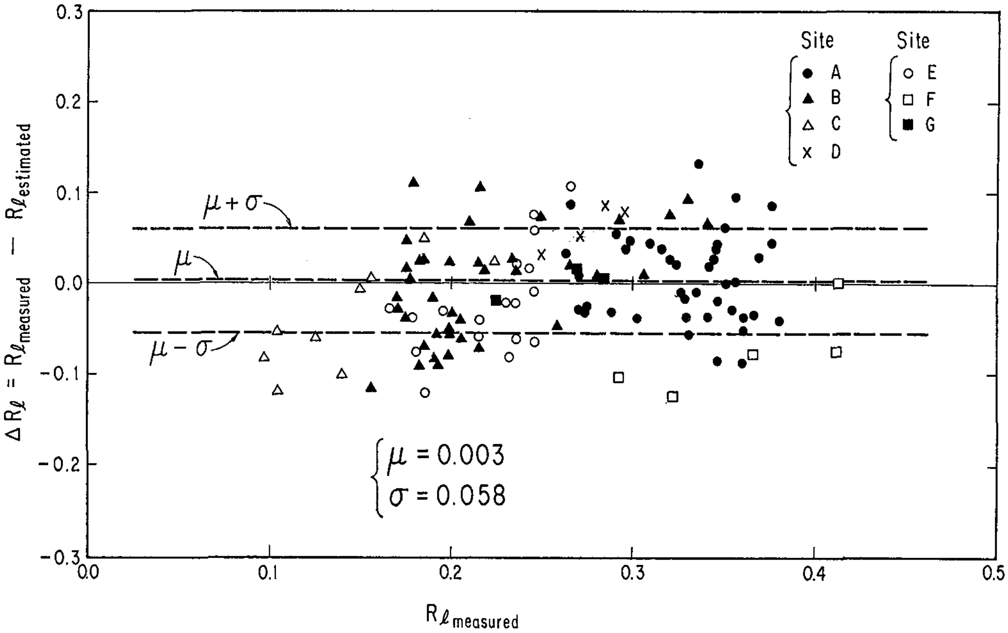


Fig.24. ΔR_1 and $R1_{measured}$ Relation

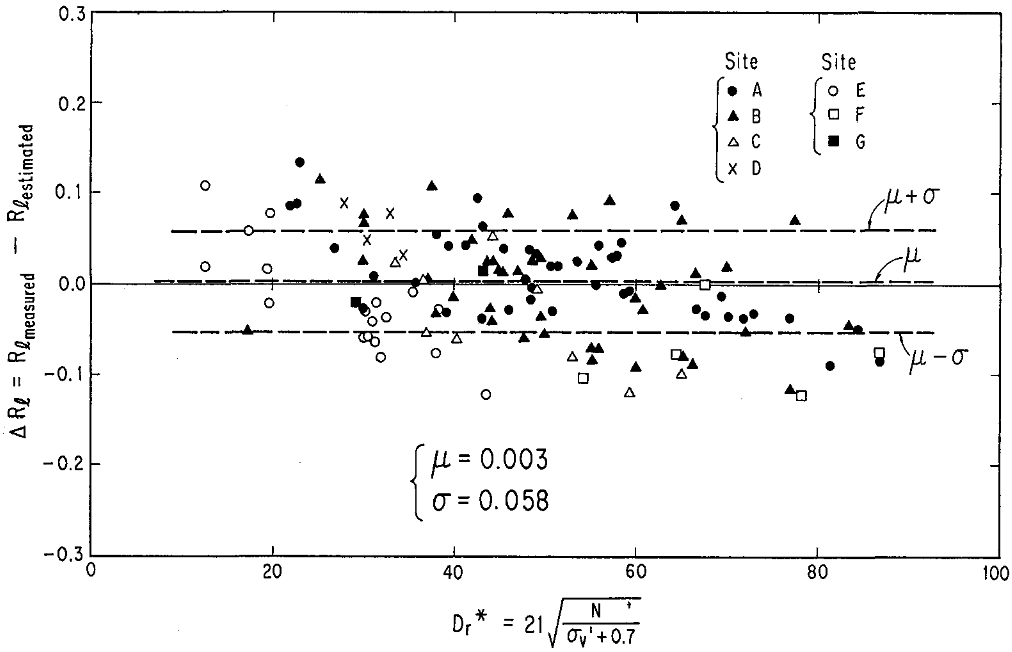


Fig.25. ΔR_1 and Dr* Relation

Table 1. List of Sand Samplings and Dynamic Triaxial Tests

(see NOTE)

No	Site	Moist Condition	Sand Sampling	Void Ratio Measurement	e_{min} Measurement	e_{max} Measurement	Dynamic Triaxial Test Condition				References
							σ_c' (kg/cm²)	Frequency (Hz)	DA for Rℓ(%)	Specimen (φcm, hcm)	
1	Artificial in tank, compacted by vibration	Air Dry. Moist. Saturated		Direct density determination	Vibrating or hemmering container using saturated sand	Lightly pouring dry sand into container	–	–	–	–	Gibbs and Holtz (1957)
2	**Site A** (in Tokyo, along Tokyo Bay) Alluvial and Reclaimed	Underwater	Reformed Bishop-type sand sampler; the liner is 5.3cmφ and 65 cmℓ *)	e_F; averaga with-air-dry in the liner	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, 10 h	This Investigation *)Hanzawa and Matsuda(1977) **)Yoshimi and Tohno (1972)
3	**Site B** (in Shiko-ku, along Setonai-kai Sea) Alluvial and Reclaimed	Underwater	Twist sand sampler; the liner is 7 cmφ and 80 cmℓ	e_o; void ratio of unconsolida-ted thawed specimen	Identical to No 2		$=\sigma_v'=$ 0.45~1.75	0.5	5	7φ, 14 h	This Investigition
4	**Site C** (in Niigata City, along Shinano River) Alluvial and Reclaimed	Underwater	Larg diameter sand sampling; the liner is 20 cmφ and 100 cmℓ;	e_o	Vibrating mold using air-dry sand on shaking table with 1kg/cm² being applied on the top (0.61)	Spooning air-dry sand into mold (0.96)	1.5 (σ_v'=0.3~1.15)	1.0	5	5φ, 10h	Ishihara (1976, 1977) Ishihara and Silver (1977)
5	**Site D** (in Tokyo) Alluvial	Underwater	Identical to No 4				0.5 (σ_v'= 0.265~0.610)	1.0	5	5φ, 10h	Ishizawa, Nakagawa and Kurohara (1977)
6	**Site E** (in Yokoha-ma, along Tokyo Bay) Reclaimed	Underwater	Identical to No 2	–	–	–	$=\sigma_v'=$ 0.37~0.77	1.0	6	5φ, 10h	This Investigation
7	**Site F** (in Tokyo, along Tokyo Boy) Alluvial	Underwater	Thin-wall sampling with enough cares	–	–	–	0.5, 1.0 (σ_v'= 0.24~0.65)	1.0	6	5φ, 10h	This Investigation
8	**Site G** (in Tokyo along Tokyo Bay) Alluvial and Reclaimed	Underwater	Identical to No 2				$=\sigma_v'=$ 0.8~1.14	0.5	6	5φ, 10h	This Investigation
9	**Site H** (Kawagish-cho, Niigata City) Alluvial and Reclaimed	Underwater	Identical to No 2	e_F	Tamping mold using air-dry sand and applying pressure of 1kg/cm² on the top (0.60)	Average by two method; Kolbuszewski (1948) and Tanimoto (1975) (0.94)	–	–	–	–	JSSMFE (1976)
10	**Site I** (Oh-gi Shima) Reclaimed	Underwater	Identical to No 2	e_F	Identical to No 9		–	–	–	–	Saito (1977)
11	**Site J** (in Yokohma, along Tokyo Bay) Reclaimed	Underwater	Frozen Column Method; making frozen 5mℓ, 40cmφ 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 Figures in ( ) represent the maximum and minimum void ratios of Toyoura Sand determined by each mothod.

Table 2. Summary of Dynamic Triaxial Test Results on Clean Sands

Reference	D_{50}	<74μ(%)	e_{max}	e_{min}	R_{l20}	D_r	Symbol in Fig.12	Note
Seed and Lee (1966)		0	1.03	0.61	0.185	38%	●	0.297 ~ 0.149mm Sacrament River Sand
Lee and Seed (1967)		0	"	"	0.325	78	▲	"
		0	"	"	0.475	100		"
Lee and Fitton (1968)	6.5	0			0.427	50	◆	ε = ±2.5%
Finn, Pickering and Bransby (1971)	0.40	0	0.82	0.50	0.26	69	⧇	Ottawa Sand
					0.14	38		ASTM C109
Shibata (1970)			1.007	0.590	0.205	55	○	Niigata Sand
Tanimoto (1971)			"	"	0.25	"		
			"	"	0.325	"	⊙	
Saito et al (1974)	0.712	0	0.921	0.623	0.34	80	△	Sengenyama Sand
			"	"	0.28	69		
			"	"	0.25	61		
Ishihara et al (1976)	0.260	2	0.99	0.55	0.19	45	◇	Niigata Sand
"	0.40	0	1.03	0.48	0.165	42.5	⬘	Fuji River Sand