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CHARACTERIZATION OF SANDY SOILS USING CPT AND DMT

SAMPLES by GROUND FREEZING

Fig. 13 !

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ABSTRACT

A site investigation campaign using cone penetration test (CPT) and dilatometer test (DMT) was carried out on three sandy soils of Japan. Two of these sites are natural sand deposits and the third one is a sand fill reclaimed about twenty years ago. On two sites, high quality samples were taken using the freezing sampling method so that the measured value of the relative density D_r is assumed to be accurate. Seismic cone tests were also performed in order to measure the shear modulus at a very small strain level in the order of 10^{-5}. Data from CPT and DMT were related to design parameters using correlations established from calibration chamber tests. The main conclusions obtained in the present study are: 1) existing soil classification methods using CPT and DMT data can be applied to the investigated sites; 2) D_r values predicted from relationships linking D_r to $q_t/(p'_{v0})^{0.5}$, where q_t is the point resistance from CPT and p'_{v0} is the vertical effective overburden pressure, seem to be in good agreement with actual values of D_r; 3) D_r can also be well predicted using DMT horizontal stress index K_D values, and 4) the ratios of the shear modulus G_{sc} from the seismic cone to q_t or E_D from DMT, G_{sc}/q_t or G_{sc}/E_D decrease with increasing values of the relative density D_r.

Key words: deformation, in-situ test, penetration test, sandy soil, site investigation, soil classification, sounding (**IGC:** C6)

INTRODUCTION

The recent popularity of computers and the accumulation of knowledge on geotechnical engineering have led to the development of numerous sophisticated numerical methods predicting the ground behavior and some of them have been applied successfully to real construction projects. Surprisingly, however, design parameters used in these apparently sophisticated calculation methods are still derived from empirical relationships developed decades ago from conventional soil investigation methods. For example, in sandy grounds, where high quality samples are difficult to retrieve due to lack of cohesion, design parameters such as internal frictional angle and deformation characteristics are still commonly derived from SPT N values. To minimize disturbance effect and enhance sample quality, freezing sampling methods were proposed and have been used for research purposes and even for the design of real projects (Yoshimi et al., 1994). Unfortunately, the cost of this method is high and the freezing sampling method does not seem to be destined to become a routine sampling method. Even with sophisticated sampling methods, sample disturbance, particularly the disturbance due to stress relief, can not be avoided during sampling procedures and laboratory test results are influenced by this disturbance. In situ tests are generally performed at in situ stress conditions, and the stress relief is negligible. However, for the large majority of in situ devices, a theoretical formulation is not easy to develop and the derivation of design parameters is difficult because boundary conditions such as stress, strain and drainage are generally complex and often impossible to define precisely. With the development of the self-boring pressuremeter, and in particular of the English probe known as the Camkometer (Wroth and Hughes, 1973), disturbance during the installation has been minimized and the test starts from actual in situ stress state. The problem with this tool is the cost of its operation. Consequently, it does not seem that even in future the pressuremeter including pre and self-bore types pressures will be standard in situ tests like the SPT in soil investigation.

The cone penetration test (CPT) and dilatometer (DMT) have been gradually used more for investigating sandy soil instead of the SPT, especially in Europe and North America. The most attractive point distinguishing these two tools from other in situ tests is that their operation is so simple and repeatable that the influence of personal errors on test results is minimized. It is well known that the N value from SPT is very much a function of the test conditions such as dropping energy, borehole bottom condition and so on. Since theoretical solutions of

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CPT and DMT design parameters cannot be easily derived, values measured with these apparatuses have been experimentally correlated with fundamental soil parameters, using calibration chamber tests where test conditions are controlled. However, these calibration chamber tests present some non-negligible disadvantages such as size effect, difference in soil specimen between laboratory manmade soils and natural deposits, in particular in terms of aging and structure. CPT and DMT have been used in sandy grounds to study whether correlations established from chamber tests can be applied to the real ground or not, and some studies have been reported in the literature. However, reliable density of the ground, in other words, relative density D_r, which is the most important state parameter characterizing sand behavior, could not be easily measured at the site due to sampling difficulties.

The authors have carried out a site investigation campaign, using CPT and DMT. In situ tests have been performed on three sand deposits, where high quality soil samples were taken by a freezing method and D_r was precisely measured at these sites. This paper presents test results from three sites, a ground reclaimed by sand fill and two natural sandy deposits, and examines the correlation obtained from calibration chamber tests.

TESTING METHOD

Cone Penetration Test

During the present investigation program, the CPT tool and procedures were in agreement with the specification of the international reference test procedure proposed by the ISSMFE technical committee on penetration testing (1988): cross sectional area of the cone is 10 cm^2 (the diameter is 35.7 mm); the apex angle of the cone is 60°; the location of the filter for measuring pore water pressure is the shoulder behind the cone and the speed of the penetration is 2 cm/s. In the present study, the point resistance q_t is corrected by the effective area and takes into account pore water pressure acting on the filter.

Dilatometer Test

A conventional Marchetti's dilatometer apparatus was used for this study. Such a tool has been described in numerous papers. As proposed by Marchetti (1980), three indices are derived from DMT;

Material Index

$$I_D=(p_1-p_0)/(p_0-u_0) \tag{1}$$

Horizontal stress index

$$K_D=(p_0-u_0)/p'_{v0} \tag{2}$$

Dilatometer coefficient

$$E_D=34.7(p_1-p_0) \tag{3}$$

where p_0 is the pressure when the membrane lift off, p_1 is the pressure when the membrane displacement is 1.1 mm, u_0 is the static water pressure and p'_{v0} is the effective vertical overburden pressure. Measurement of p_0 and p_1 was done within 15 s, as recommended by Marchetti.

Seismic Cone Penetration Test

Seismic cone penetration tests were performed using the down hole method technique in which a shear wave is generated on the ground surface by hitting a plank and is received by receivers located on the shaft of the cone. In order to get a higher accuracy in the measurement of the wave arrival time, receivers are installed at two different locations on the shaft, separated exactly by 1 m, and two accelerometers are horizontally installed at the same place. The shear wave velocity v_s is the time necessary for the shear wave to move from the first receiver to the second one divided by the distance separating the two receivers i.e., 1 m. This time interval is given the signal of the two receivers and was defined using an oscilloscope (Tanaka et al., 1994). The shear modulus may be calculated by Eq. (4).

$$G_{sc}=\rho_t v_s^2 \tag{4}$$

where G_{sc} is the shear modulus measured by the seismic cone, and ρ_t is the bulk modulus of soil.

Standard Penetration Test

SPT was performed according to the Japanese Industrial Standard JIS A 1219-1995. A hammer of 63.5 kg hooked at "tombi", which is a Japanese name for a hook, fell by a height of 75 cm. This dropping method is extensively used in Japan in practical investigations.

INVESTIGATED SITES

The location of the three investigated sites is shown in Fig. 1.

Ohgishima

This site is located in the vicinity of Kawasaki city, fac-

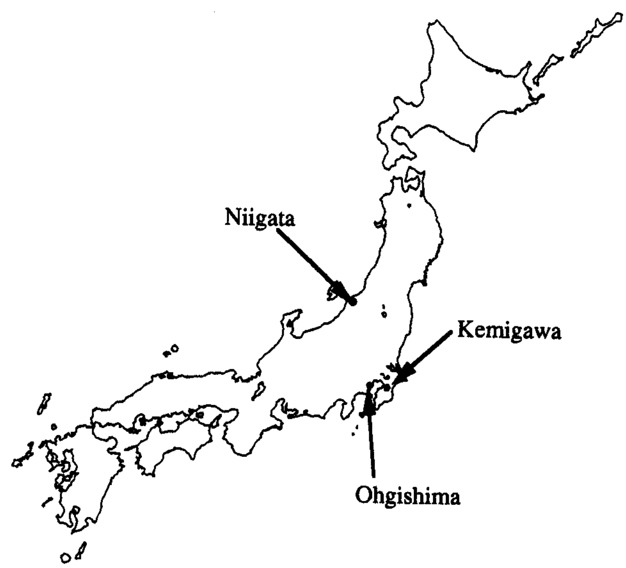


Fig. 1. Location of test sites

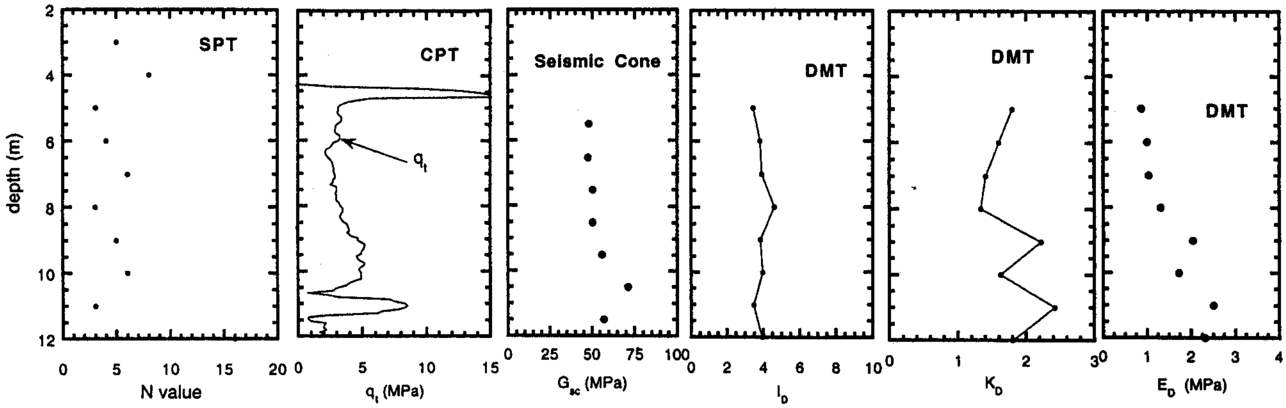


Fig. 2. Test results at Ohgishima

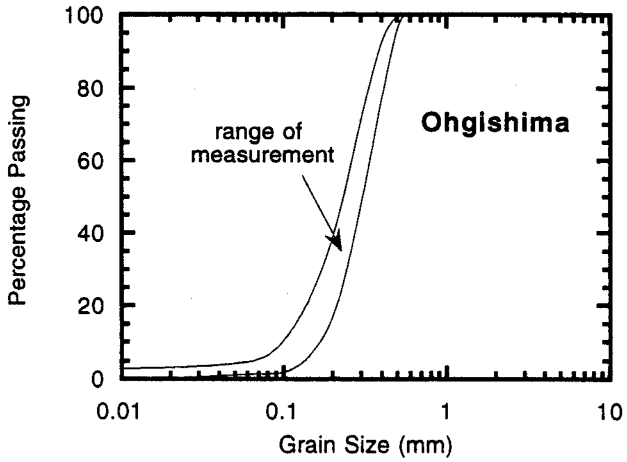


Fig. 3. Gradation curve of the objective layer at Ohgishima

ing the Tokyo Bay. It was reclaimed with sand in the early 70's (from 1972 through 1973). Test results from SPT, CPT, seismic cone and DMT are summarized in Fig. 2. This site was paved by asphalt concrete so that a borehole was made from the ground surface to a depth of 3 to 4 m. It can be judged by the q_t (CPT) profile that the original sea bottom was located at 11 m depth. Below this depth, alternative sand and clay layers are found. The objective layer in the present investigation is the 7 m thick sand layer located from 4 m to 11 m depth below the ground surface.

Grain size analysis was carried out for sands specimens from several depths and grain size distribution curves fall in a narrow range, as shown in Fig. 3. The gradient of the grading curve is steep and the uniformity coefficient U_c, which is defined by the ratio of soil particle diameters at 10% and 60% passing in weight, is about 2. The mean particle size D_{50} varies from 0.2 to 0.3 mm.

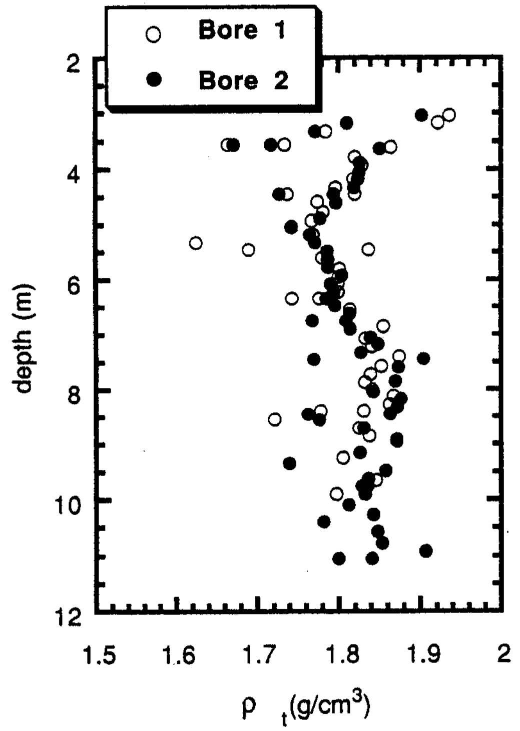


Fig. 4. Unit soil mass directly measured by core samples (Ohgishima)

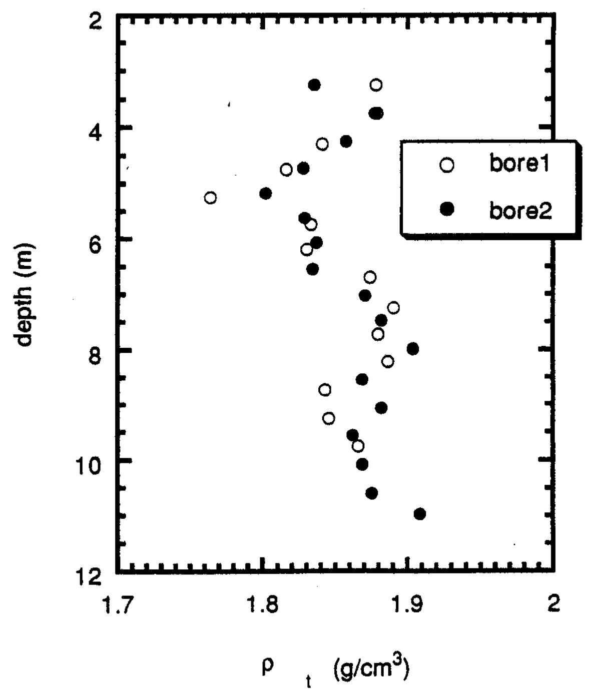


Fig. 5. Average of unit soil mass for 0.5 m depth (Ohgishima)

Soil samples were taken by the freezing method. In this method, liquid nitrogen injected into the ground through a cooling pipe. The surrounding soil is then progressively frozen in a cylinder shaped sample. When the diameter of the in situ frozen zone reached a diameter of about 0.7 m, which is shown by thermometers installed in the ground, two 15 cm diameter tubes of frozen ground were cored 25 cm from the center of the cooling pipe. These tubes were cored continuously over the full length of the frozen zone from 4 to 11 m, and were stored in a freezing container. The frozen cores were divided in the laboratory into 10 to 15 cm long samples. Diameter, length and weight of these samplers were measured. The unit mass of ρ_t was determined from these figures, assuming that the unit mass of frozen water in the specimen is 0.917 g/cm^3.

In Fig. 4 are presented the profiles of the unit mass as measured from the two frozen samples. As shown in this figure the scatter in the measured ρ_t is large. Considering the relatively smooth curves of the cone tip resistance q_t measured by CPT or any indices derived from DMT, such a scatter is beyond our expectations. However, when the average value of ρ_t for every 0.5 m depth was recalculated as in Fig. 5, the large scatter in ρ_t is reduced

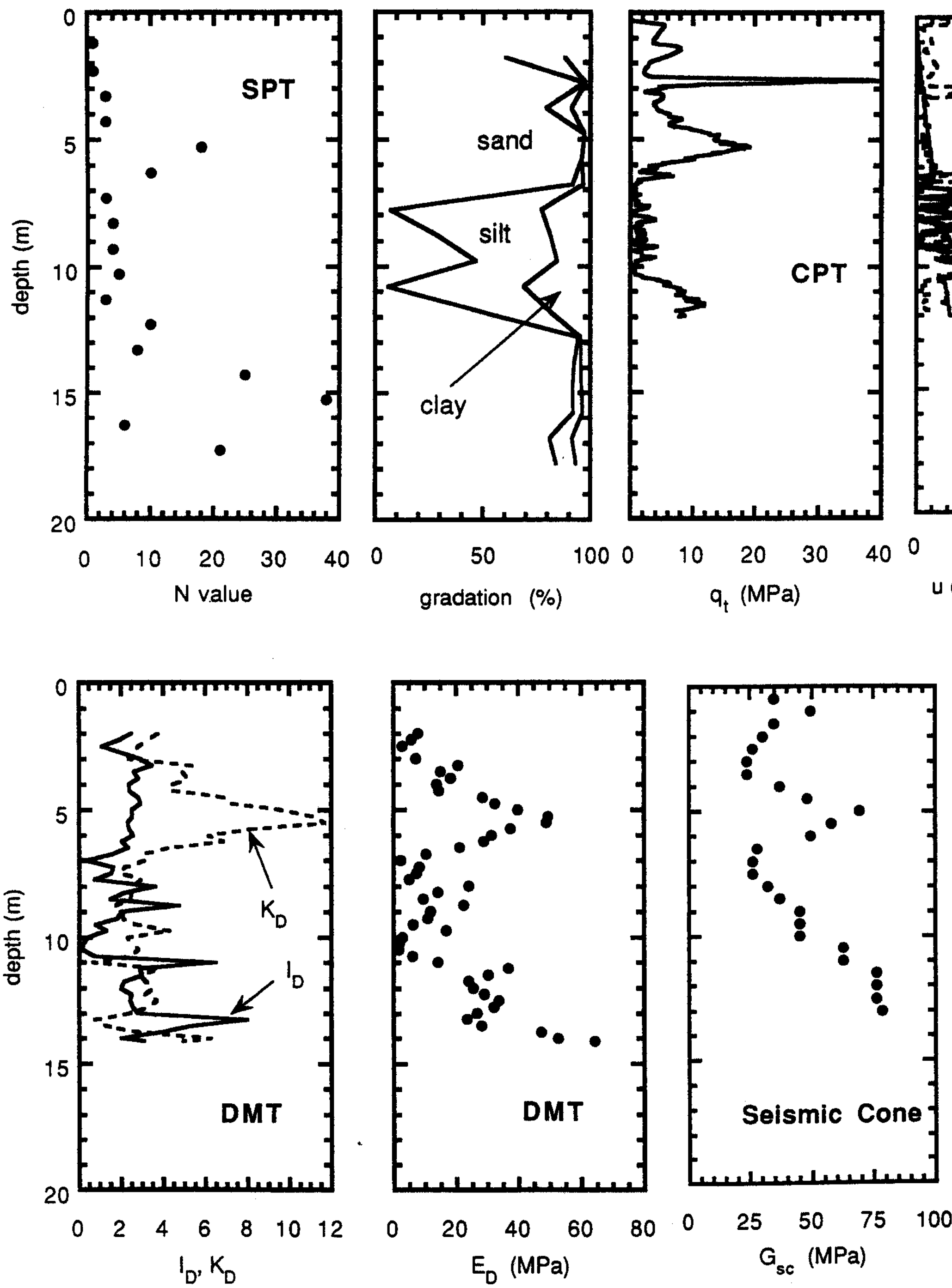


Fig. 6. Test results in Niigata

and the values of ρ_t for two bore holes are almost identical. The density value decreases to reach a minimum around 5 m and then increases slightly with depth. This trend indicates that the fill is not microscopically homogeneous and that a larger size of soil mass is required to characterize the fill material accurately. It should be kept in mind that laboratory test results do not always represent the behavior of the whole ground because the specimens size are usually less than 10 cm.

Niigata

This site is in Niigata city, Niigata prefecture, which was heavily damaged in 1964 by the Niigata earthquake. The study site is located at the left side of the mouth of the Shinano river just before entering the Japan Sea. Soil samples were taken using the thin wall piston sampler commonly used in Japan for clay sampling. Grain size analysis and in situ tests were carried out. Results of the grain size analysis as well as results from SPT, CPT, DMT and the seismic cone are shown on Fig. 6. The layer between depths of 6 and 10 m contains a large amount of silt and clay, leading to small N (SPT) and q_t (CPT) values. Such a material is not considered as sand and the data obtained at these depths will not be considered in the present paper. The particle sizes of the upper and lower sand layers are almost identical, as shown in the gradation curves presented in Fig. 7, with U_c value of 3 and D_{50} value varying from 0.2 to 0.4 mm.

Kemigawa

This site is located in the Tokyo suburb, at Chiba city. Test results from the investigation campaign on this site are shown in Fig. 8. An 8 m thick peaty sand layer covers a sandy layer which is the layer of interest for the present study. The soil freezing method was used for sampling and the sampling procedure was the same as the procedure previously described for Ohgishima. N values from SPT as well as q_t are much higher than those measured at Ohgishima and Niigata. A typical gradation curve for this site is shown in Fig. 9. U_c value is about 5 and D_{50} value is 0.15 mm.

Summarizing the main features of the investigated sites, we can conclude that the Ohgishima site was reclaimed by a relatively loose sand; Kemigawa and Niigata sites are natural sand deposits but the density is much higher for the Kemigawa deposit. All tested layers were

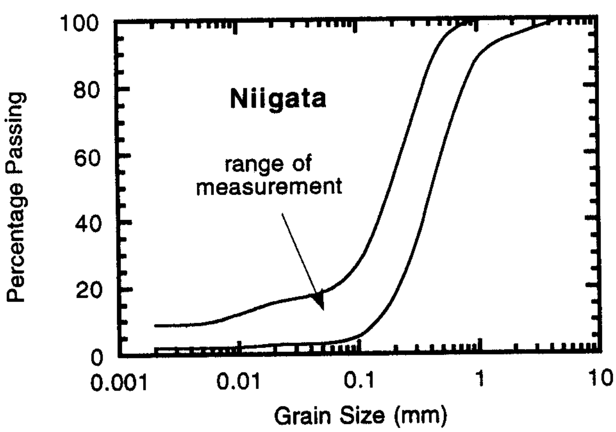


Fig. 7. Gradation curve of the objective layer at Niigata

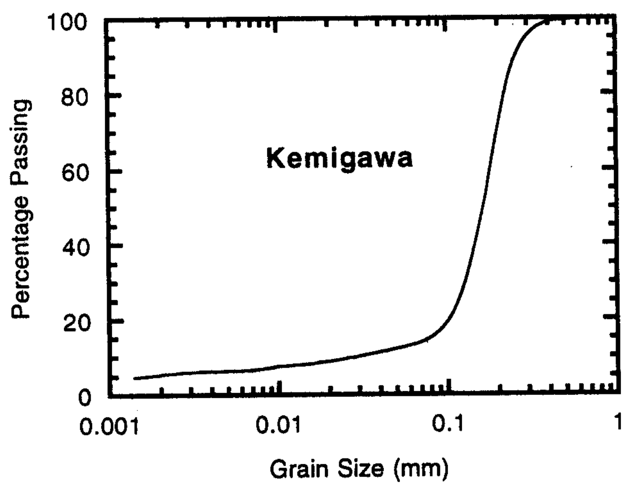


Fig. 9. Gradation curve of the objective layer at Kemigawa

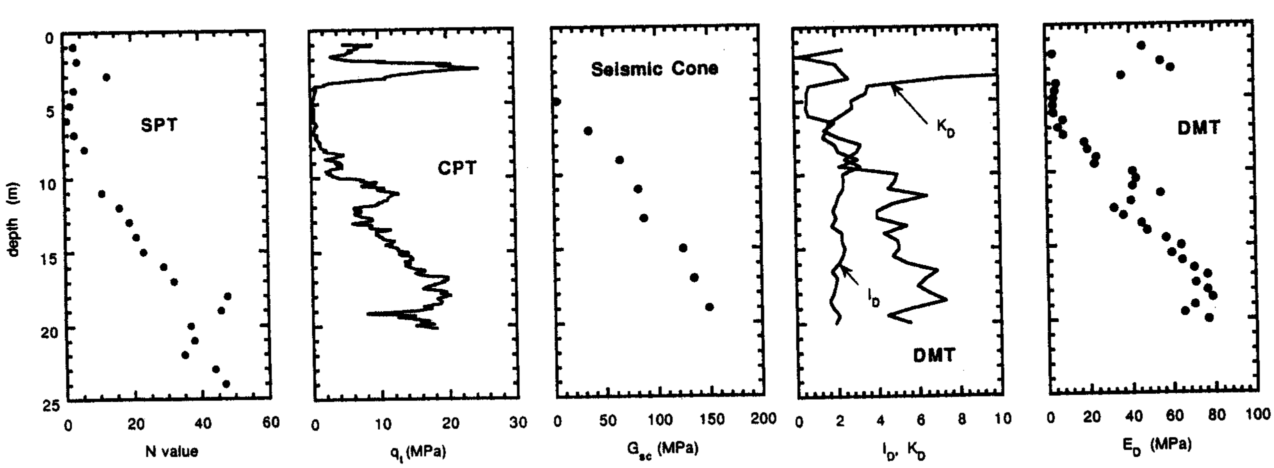


Fig. 8. Test results at Kemigawa

NC

deposited after the ice age, i.e., Holocene deposit, and consist of almost the same diameter of soil particles.

SOIL CLASSIFICATION

Several classification methods using CPT or DMT have been proposed in the literature. CPT classification methods are based on the following fundamental behavior:

1) for clay, low value of q_t, high pore water pressure u and high friction ratio f_s/q_t. where f_s is friction measured in the sleeve:

2) for sand, large value of q_t, pore pressure u equal to or lower than static hydraulic pressure due to dilantancy, and a rather low f_s/q_t ratio.

Figure 6 presents various in situ test results for the Niigata site. The value of the f_s/q_t ratio seems to respect the tendencies enumerated above since the value of the ratio is small in the upper zone where the soil is mostly sand and the ratio increases in the silty layer. However, according to the comparative study using eight different types of cones which was done at the test field of the Port and Harbour Research Institute (PHRI) in 1994 (Tanaka, 1995), the friction value f_s seems to be sensitive to the cone type used. A considerably large scatter in the f_s value at the same depth, with a ratio f_{smax}/f_{smin} sometimes as high as 6, was obtained by Tanaka (1995). Thus, when the soil classification is carried out using f_s, it is of importance to consider the reliability of the measured friction.

For DMT, the Material index I_D is usually used in the soil classification. According to Marchetti's proposal, when I_D is less than 0.6, between 0.6 and 2.0, and larger than 2.0, then the soil is classified respectively as clay, silt and sand. It can be seen in the DMT results profiles presented in Figs. 2, 6 and 8 as well as the grain size distribution curves of Figs. 3 and 9 that I_D value is consistent with the soil classification method based on the gradation. However, the value I_D of 2, which classifies the soil as sand, is rather large since I_D values in sand layers at Kemigawa and Niigata are sometimes smaller than 2.0, although these layers definitely consist of sand as shown in the gradation curves of Figs. 3 and 9.

CORRELATION TO *N* VALUE

Putting aside discussions about whether the SPT provides reliable data to evaluate soil parameters or not, it is nearly impossible to design foundations without knowing N value because the present design methods established by authorities or associations are deeply based on N values. Soil parameters determined from CPT or DMT should be correlated with N values for a while until design methods based on CPT or DMT without the aid of N value are developed and accepted in the geotechnical profession. Tip resistance values q_t have been correlated to N values by several researchers and it is known that the q_t/N ratio is influenced by the diameter of soil particles. According to a review conducted by Robertson (1986), the q_t/N ratio (with q_t expressed in bar; 100 kPa) is about 4 if the diameter of sand particle is 0.2 mm. Comparison of q_t and N values is shown in Fig. 10. It can be seen that the majority of the data points from the present study is located below the q_t/N=4 line. This means that the q_t/N ratio reviewed by Robertson (1986) considerably overestimates N value. A part of this overestimation may be attributed to differences in conducting SPT and in particular in dropping the hammer. To avoid scatter in N values caused by different dropping methods, N values in the diagram reviewed by Robertson were obtained using the same energy ratio of about 55-60% of ideally free hammer dropping. According to test results by Seed et

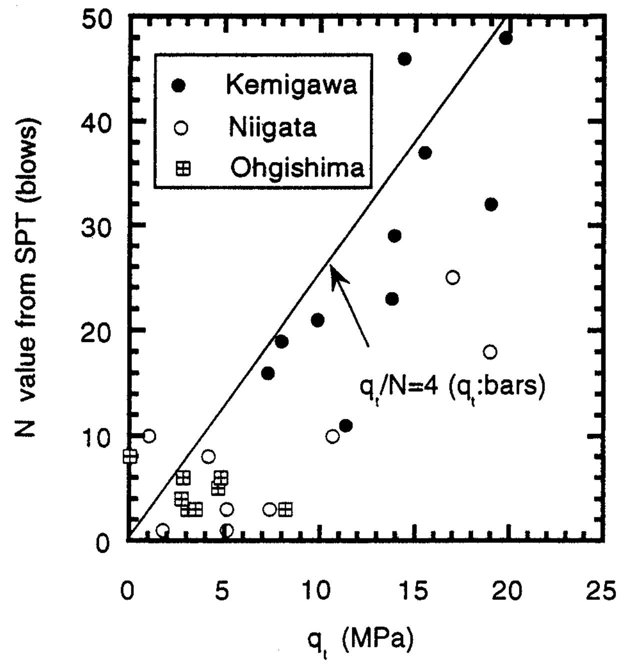


Fig. 10. Correlation of N value and point resistance q_t of CPT

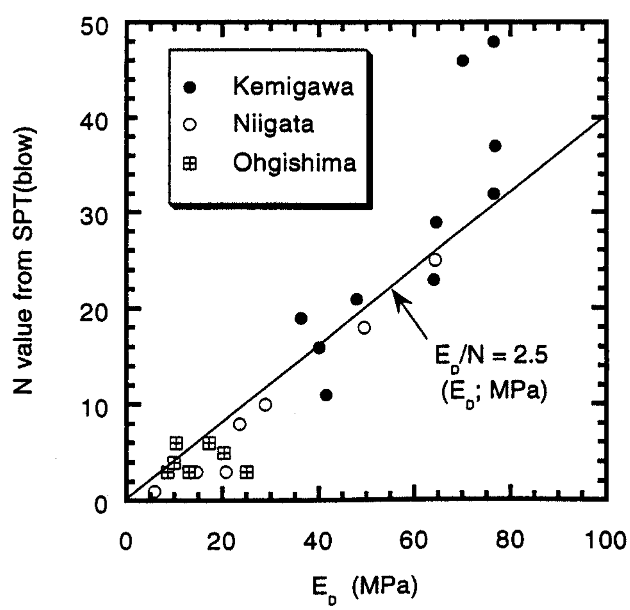


Fig. 11. Correlation of N value and E_D from DMT

$N = E_D/2.5$
MPa

al. (1985), an energy ratio for the SPT using the tombi is 78%. The energy ratio using the tombi is then larger than the energy ratio of the data points of the diagram when deriving the relation $q_t/N=4$. If we consider the difference in the dropping energy between Robertson's diagram and the present investigation, which was conducted used the tombi, data obtained in the present study come closer to the relation proposed by Robertson (1986) $q_t/N=4$.

As seen in Fig. 11, there is a good correlation between N and E_D from DMT as $E_D=2.5\ N$ (a dimension of E_D: MPa). Although further research is necessary to confirm whether the E_D/N ratio is affected by diameter of soil particles like q_t/N or not, this correlation is quite useful in practical design using DMT.

RELATIVE DENSITY

Relative density D_r is one of the most important state parameters in predicting the behavior of sand. Until now, many methods predicting D_r from CPT have been proposed and D_r in most of these methods is correlated with values of $q_t/(p'_{v0})^{0.5}$ where p'_{v0} is the effective vertical overburden pressure. Lancelotta, as reported by Jamiolkowski (1985), proposed the following equation based on a number of tests in a calibration chamber.

$$D_r = -98 + 66 \log_{10}(q_t/(p'_{v0})^{0.5}) \tag{4}$$

where D_r is expressed in % and q_t and p'_{v0} in tf/m^2 (=10 kPa).

Tanizawa et al. (1990) proposed a similar equation, Eq. 5, to predict D_r from CPT.

$$D_r = -85.1 + 76 \log_{10}(q_t/(p'_{v0})^{0.5}) \tag{5}$$

where the dimension of D_r is % and that of q_t and p'_{v0} is kgf/cm^2 (=100 kPa).

Using test results obtained in Ohgishima and Kemigawa where D_r is assumed to be accurate since it was defined from frozen samples, applicability of Eqs. (4) and (5) is examined. In Fig. 12, the relationships proposed by Lancellota and Tanizawa et al. are presented as well as the data obtained in the present study. Equations (4) and (5) are almost identical, they both overestimate D_r in Ohgishima and underestimate it in Kemigawa. A further investigation is required to conclude whether this difference is caused by the different processes of sedimentation in these sand layers (natural deposit in Kemigawa and artificially reclaimed one in Ohgishima).

Using indices from DMT, a method for obtaining D_r has been proposed by Robertson and Campanella (1986) as part of a procedure to estimate liquefaction potential

in Reyna even better!

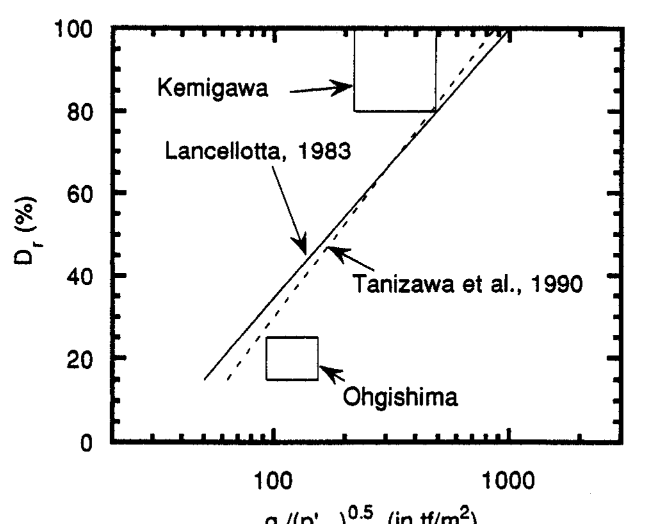


Fig. 12. Comparison of measured relative density and predicted by CPT

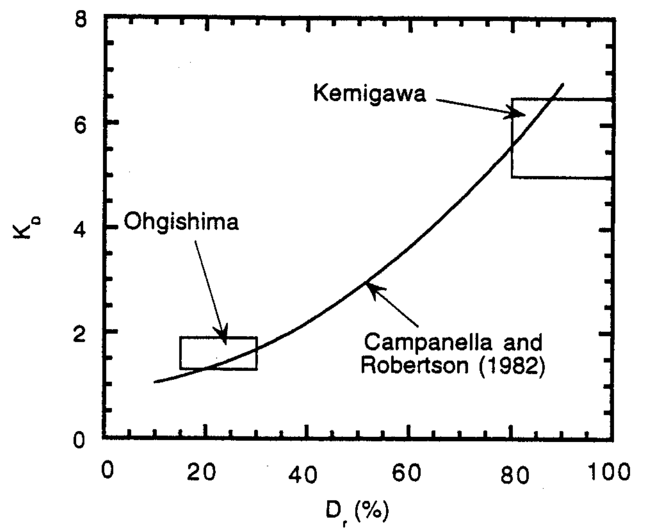


Fig. 13. Comparison of measured relative density and predicted by DMT

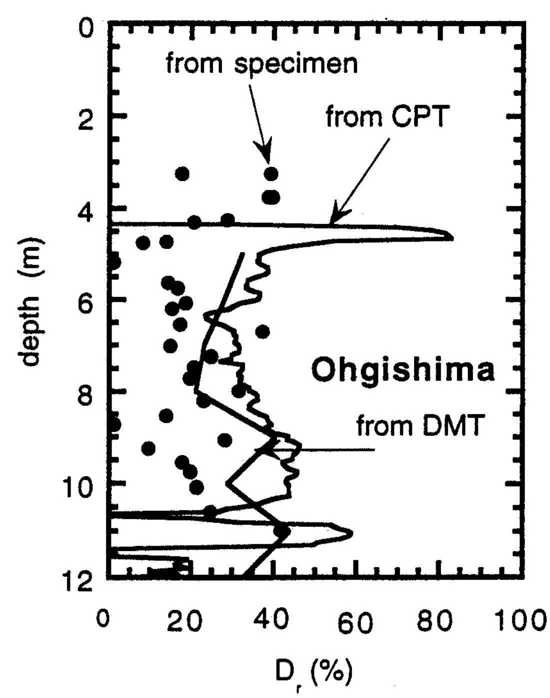


Fig. 14. Relative density against depth at Ohgishima

from DMT. The proposed relation between D_r and K_D is shown in Fig. 13. In this figure test results from Ohgishima and Kemigawa are also presented. Results from the present study seem to be in good agreement with the relation proposed by Robertson and Campanella (1986).

Figures 14 and 15 present profiles of the relative density D_r measured from frozen samples and predicted using Eq. (4) and the relation proposed by Robertson and Campanella in Fig. 13 for Ohgishima and Kemigawa respectively. The D_r value predicted from CPT using Eq. (4) is always larger than that predicted from DMT data. In Niigata, the accuracy in D_r from CPT and DMT cannot be assessed because the reference value of D_r, measured from frozen samples, was not available since the frozen sampling method was not used at that site. The magnitude of D_r from both in situ tests can be compared in Fig. 16, indicating that predicted D_r values are of the same order.

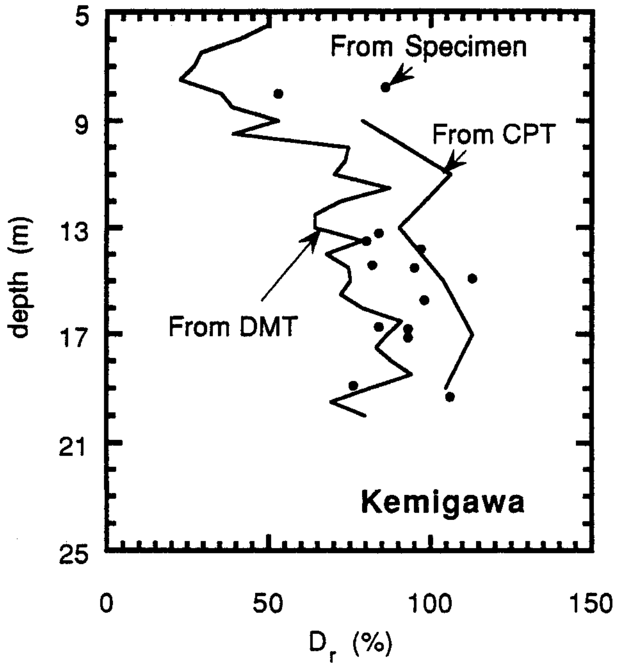


Fig. 15. Relative density against depth at Kemigawa

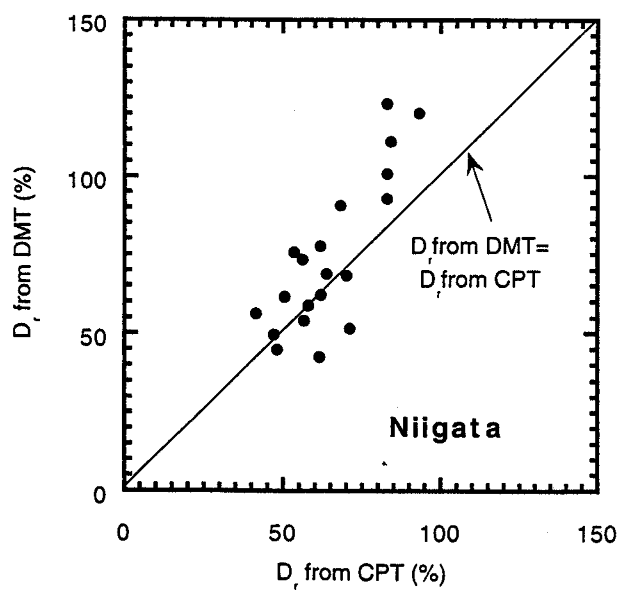


Fig. 16. Comparison of relative density predicted by CPT and DMT at Niigata

SHEAR MODULUS

It is well known that the shear modulus G of soil is dependent on strain level; that is, the stress-strain relation is considerably non-linear even at small strain. However, it is found that G at strain smaller than 10^{-5} is independent of strain level and strain rate, and is equivalent to the value obtained by seismic methods such as the seismic cone (Tatsuoka and Shibuya, 1992). It may be considered that G at these very small strain levels, usually denoted G_{max}, or G_0 in some references, may be one of the most fundamental parameters governed by state parameters such as the confining stresses and the void ratio. It is thus interesting to correlate G_{max} with parameters measured by CPT or DMT.

Tanaka et al. (1994) showed that there is a unique relation between the shear modulus obtained from seismic cone tests, G_{sc} ($=G_{max}$), and the parameter (q_t-p_{v0}) for normally or slightly overconsolidated clays. The relation proposed by Tanaka et al. (1994) is $G_{sc}=50$ (q_t-p_{v0}) in which q_t is the tip resistance deduced from CPT and p_{v0} is the total overburden pressure (Fig. 17). Tanaka et al. (1994) presented a similar relation between G_{sc} and the DMT modulus E_D in clays; $G_{sc}=7.5E_D$ (Fig. 18). However, such a linear relation of G_{sc} is not observed in sand. Figure 19 presents the relationship between the ra-

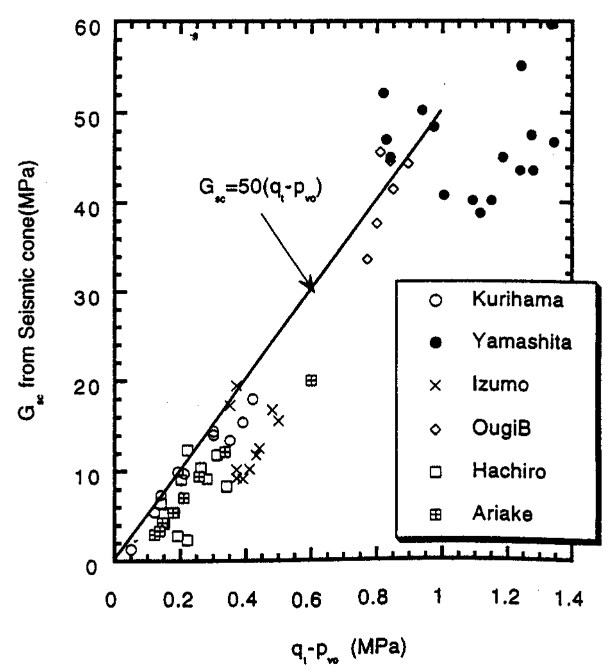


Fig. 17. Correlation of G_{sc} from seismic cone and (q_t-p_{v0}) from CPT for clays after Tanaka et al. (1994) (some data are added to the original diagram)

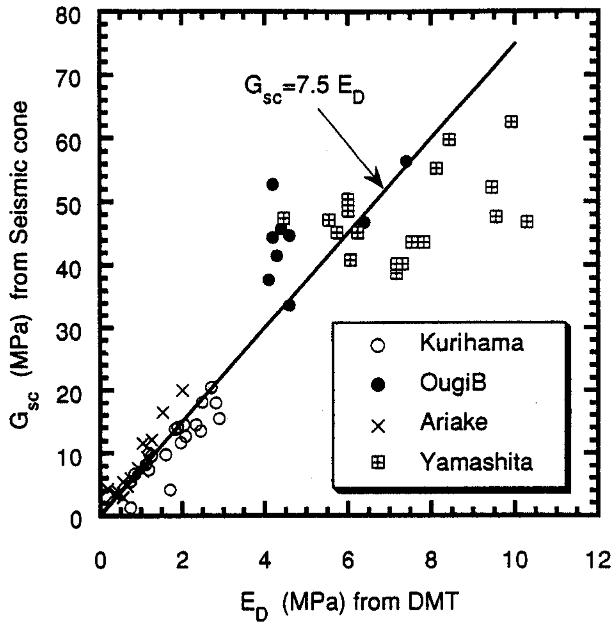


Fig. 18. **Correlation of G_{sc} from seismic cone and E_D from DMT for clays after Tanaka et al. (1994) (some data are added to the original diagram)**

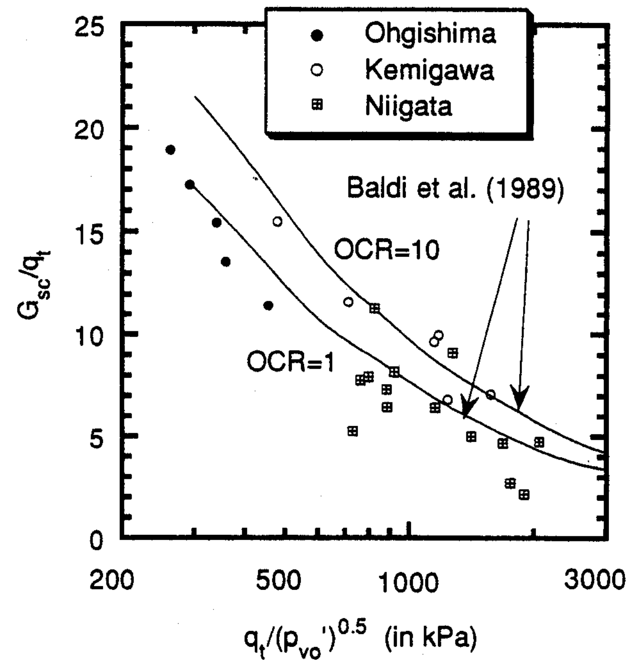


Fig. 20. **Correlation of G_{sc}/q_t and $q_t/(p'_{v0})^{0.5}$**

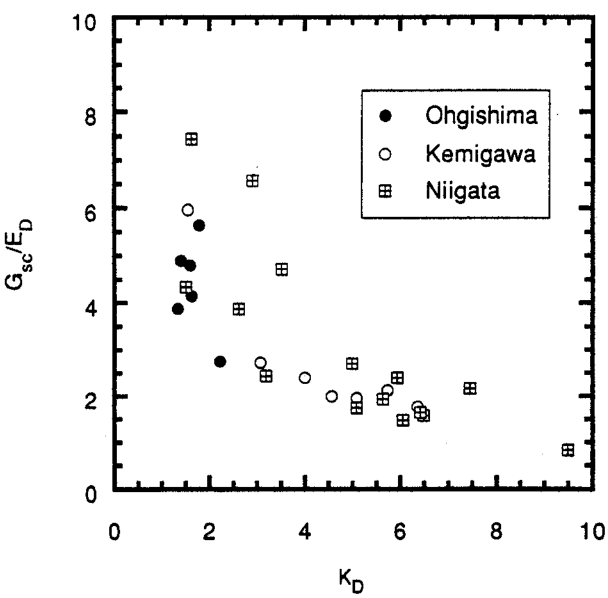


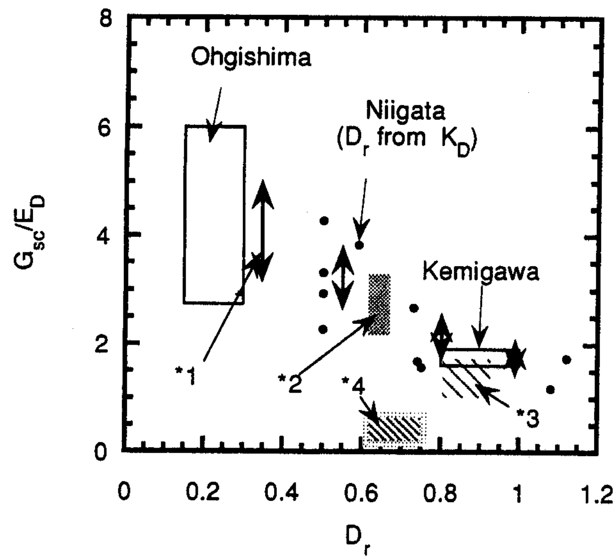
Fig. 19. **The ratio of G_{sc} from seismic cone to E_D from DMT against K_D (sandy soil)**

tio of the shear modulus over the dilatometer coefficient G_{sc}/E_D and the parameter K_D for the three tested soils. It can be seen in the Fig. 19 that the ratio G_{sc}/E_D is not constant but decreases with increasing values of K_D. It is interesting to know that the ratio of G_{sc}/E_D approaches to 7.5 at small K_D, which is the same ratio derived by Tanaka et al. (1994) for normally or slightly overconsolidated clays and that the K_D index is generally about 2 for these clays (Tanaka et al., 1996).

Baldi et al. (1989) proposed from a test series in a calibration chamber a relation between the ratio G_{max}/q_t and the ratio $q_t/(p'_{v0})^{0.5}$. The relation proposed by Baldi et al. is slightly influenced by the overconsolidated ratio (OCR) as shown in Fig. 20. It can be also seen in Fig. 20 that the test results from the present investigation follow the same trend and are thus consistent with Baldi et al.'s (1989) relation. Unlike clayey soils, the G_{sc}/q_t ratio is not constant for sandy soils but depends on the value of the ratio $q_t/(p'_{v0})^{0.5}$.

As shown in Fig. 13, K_D is a good indicator of the relative density D_r, and the $q_t/(p'_{v0})^{0.5}$ value is strongly related to the relative density D_r as shown in Fig. 12. Consequently, it can be expected that the value of both ratios G_{sc}/E_D and G_{sc}/q_t is influenced by D_r. Sully and Campanella (1989) showed that the G_{sc}/E_D ratio varies with D_r as shown in Fig. 21, where the data obtained in the present study are also plotted. The authors' data also support their correlation and it is clearly demonstrated that the G_{sc}/E_D ratio is affected by the relative density D_r.

So, in contrast to clays, where both ratios $G_{sc}/(q_t-p_{v0})$ and G_{sc}/E_D are constant, it is found that for sandy soils both ratios G_{sc}/q_t and G_{sc}/E_D decrease when the relative density D_r increases. Why is such a difference in behavior observed between sand and clay? K_D is an index strongly related to the coefficient of earth pressure at rest K_0, as proposed by Marchetti (1980). The relation between D_r and K_D in Fig. 13 can be explained by the fact that K_0 increases with D_r increasing, as proved by many measurements of lateral pressure in calibration chamber tests (for example, Bellotti et al., 1986). Subsequently E_D value, which corresponds to a deformation modulus in the horizontal direction, also increases when D_r increases, as shown in Figs. 2 and 14 for Ohgishima and in Figs. 8 and 15 for Kemigawa. If we consider a depth of 10 m, D_r value is 25% at Ohgishima and 90% at



*1 Tinco Sand (Bellotti et al., 1989)
*2 Po River Sand (Bellotti et al., 1989)
*3 UOM Silty Sand (Hryciw & Woods, 1988)
*4 LBS Sand (Sully &Campanella, 1989)

Fig. 21. Correlation with G_{sc}/E_D and D_r

Kemigawa. E_D at these depths is 2 MPa at Ohgishima and 40 MPa at Kemigawa, i.e., the dilatometer modulus E_D at Kemigawa is 20 times larger than that at Ohgishima for the same depth.

The assumption that increasing D_r generates a larger K_0 is also supported by cone penetration tests used in calibration chamber conducted by Houlsby and Hitchman (1988). They concluded that the tip resistance of the cone penetrometer is mainly controlled by the lateral pressure and is independent of the vertical pressure. It can be explained from their experimental results that the increasing $q_t/(p'_{v0})^{0.5}$ values with D_r in Fig. 12 are due to an increase in the lateral pressure, i.e., an increase in K_0. Both values q_t and E_D are therefore strongly influenced by D_r and K_0 values.

Concerning the shear modulus G_{max}, its characteristics at small strain levels are somewhat different from those of E_D or q_t. Reviewing the past studies on G_{max}, and according to Viggiani and Atkinson (1995), G_{max} can be written in the following form:

$$G_{max}=F(e)\,p_a^{(1-n)}p_k'^{k}p_l'^{l}p_m'^{m} \tag{6}$$

where, $F(e)$ is a function of the void ratio, p_a is the atmospheric pressure, p'_k is the effective stress in the direction of wave propagation, p'_l is the effective stress in the direction of particle motion, p'_m is the effective stress in a direction perpendicular to the wave propagation, and $n=k+l+m$. From several studies, it has been reported in the literature that the influence of p'_m on G_{max} is nearly negligible; that is, $m=0$ and the exponents of p'_k and p'_l are the same ($k=l$) and with a value around 0.25. In the seismic cone, p'_k may correspond to the effective vertical stress, i.e., p'_{v0}: p'_l and p'_m are the effective horizontal stresses, i.e., $K_0p'_{v0}$. Using these experimental facts, the following equation can be derived.

$$G_{sc}=F(e)\,p_a^{(1-2k)}K_0^{k}p_{v0}'^{2k}=F(e)\,p_a^{0.5}K_0^{0.25}p_{v0}'^{0.5} \tag{7}$$

Equation 7 suggests that the shear modulus G_{sc} value is only slightly influenced by K_0 because the power of K_0 is half that of the vertical stress. So if we consider that K_0 is related to D_r, G_{sc} is only slightly influenced by the relative density D_r. As an example, G_{sc} is 70 MPa at 10 m depth at Kemigawa, which is only about 1.2 times larger than the G_{sc} value at the same depth at Ohgishima in spite of the fact that E_D and q_t at these sites are 20 and 2 times as large, respectively, as previously mentioned. Therefore, the decrease in G_{sc}/q_t or G_{sc}/E_D values, when the relative density or the coefficient of pressure at rest increase in sandy soils, is physically acceptable.

CONCLUSIONS

Three sandy soil sites were investigated using cone penetration (CPT), dilatometer (DMT) and seismic cone tests. At two sites, high quality specimens were sampled by a freezing method so that the relative density D_r could be measured accurately. Data obtained from these sites were analyzed to examine the correlation proposed from calibration chamber tests. It emerges from the present study that correlations obtained from tests conducted in a calibration chamber can be applied to the real ground. The main findings obtained in the present study can be summarized as follows:

1) Soil classification methods based on tip resistance, q_t, pore water pressure, u and friction ratio, f_s/q_t are applicable with good consistency to the present investigated sites except for the value of DMT material index I_D of 2.0, which is likely to be too large for sand.

2) D_r measured using frozen samples was compared with prediction methods using CPT proposed by Lancellotta and Tanizawa et al. These methods provide D_r values of the same order and have a tendency to overestimate the relative density D_r at small cone tip resistance q_t and underestimate it at large q_t.

3) The uniqueness of the relation between the relative density D_r and the dilatometer horizontal stress index K_D, as proposed by Robertson and Campanella is confirmed in the present study.

4) Using the seismic cone, the shear modulus under very small strain, G_{sc} was measured. From previous research conducted by the authors, it is known that for cohesive soils there are unique correlations between G_{sc} and E_D from DMT, or (q_t-p_{v0}) from CPT, where p_{v0} is the total burden pressure. It is found, however, that for sandy grounds these ratios are not constant but vary with D_r.

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