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A CASE STUDY FOR CHARACTERIZING UNDRAINED CYCLIC DEFORMATION PROPERTIES IN YOUNG SAND DEPOSIT FROM IN-SITU AND LABORATORY TESTS

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ABSTRACT

This paper describes a case study in which undrained cyclic deformation properties of a recent sand deposit in use for seismic response analysis was investigated through a comprehensive site investigation, coupled with cyclic loading tests in the laboratory. The test site comprised a loose sand deposit in Tokyo Bay which was reclaimed in 1960'. The objective of this investigation was to examine the applicability of profiling small-strain shear modulus and damping characteristics of the recently reclaimed sand deposit by performing undrained cyclic triaxial and/or torsional shear tests on disturbed sand samples retrieved by ordinary tube samplers, and also on fully reconstituted samples prepared with a void ratio similar to the in-situ subsoil. Two types of sand sample with high and low quality were obtained by means of in-situ ground freezing technique and by ordinary tube samplers, respectively. The quality of the samples was evaluated by comparing the pseudo-elastic shear modulus, G_{max}, from the laboratory cyclic loading tests to G_f from an in-situ downhole seismic cone test, which was involved with little disturbance to the original subsoil. On the basis of the test results, it was concluded that for this loose sand deposited only thirty years ago, the aging and structural effects were not significant with respect to the undrained cyclic deformation properties at small strains. It was also demonstrated that the undrained cyclic deformation properties can be evaluated with laboratory tests on isotropically consolidated specimens using disturbed or even fully reconstituted samples when the density profile with depth is pre-determined by means of appropriate in-situ tests.

Key words: elasticity, in-situ test, laboratory test, sand, shear modulus, small-strain, soil sampling **(IGC:** D7/D6/C6)

INTRODUCTION

Geotechnical engineers in Japan often encounter loose deposits of fine sand which resulted from land reclamation in the second half of this century. Most nearshore man-made islands in metropolitan areas comprise very recent deposits of sand which have neither been compacted nor chemically improved during and after the reclamation work. Seismic response of these deposits is hence the primary concern for practitioners when evaluating seismic stability of the foundations and super-structures.

In predicting the ground motion by means of seismic response analysis, the small-strain stiffness and damping of in-situ subsoil subjected to cyclic loading can be correctly characterized through a rational site investigation program which usually involves performance of laboratory cyclic loading tests on samples retrieved from the site. The strain range necessary for the seismic response analysis spans between 0.0001% (1×10^{-6}) and 0.1% (1×10^{-3}). For this small-strain region, the shear modulus which accompanies strains less than 0.001% (1×10^{-5}) can be employed as a fundamental soil stiffness, since the 'pseudo'-elastic shear (or Young's) modulus as observed in the laboratory, G_{max} (or E_{max}), is known to be close to the maximum value under given stress/strain conditions, and it is independent of shearing rate, type of loading, number of cycles, etc., but it accompanies a limiting value of hysteretic damping (Tatsuoka and Shibuya, 1992; Shibuya et al., 1992, 1995). The G_{max} profile with depth from laboratory tests performed under perfect conditions may therefore coincide with the profile of G_f ($=\rho_t V_s^2$, ρ_t: total soil density) determined from the measurement of in-situ seismic shear wave velocity, V_s.

The G_f value from in-situ seismic type measurements accommodates extremely small strains of about 0.0001%

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(1×10^{-6}), causing little disturbance to the current state of the in-situ subsoil, unless it is heterogeneous comprising large stones (Tanaka, Y. et al., 1994). Conversely, the measurement of G_{max} in the laboratory usually involves the effects of sample disturbance, stress relief, etc. In addition, conventional sampling techniques for cohesionless soils induce changes in the in-situ density and soil structure. Techniques are currently available to obtain high-quality samples of cohesionless soils, for example, by means of artificial ground freezing (Yoshimi et al., 1984, 1989, 1994). Quality of in-situ frozen samples subjected to coring and thawing has been intensively examined related to the drained and undrained strengths (Yoshimi et al., 1978, 1989; Singh et al., 1982; Hatanaka et al., 1985, 1995). Recently, Satitharan et al. (1994) have carried out a laboratory investigation into the effects of freezing and thawing on G_{max} of sand/clay mixtures. The work indicated that the percent fines content, together with the mineralogy of the fines is important; for sand containing more than 5% fines of expansive minerals, freezing and thawing could result in substantial reduction of G_{max}.

Despite that the in-situ ground freezing technique appears to be effective in providing high-quality sand samples in many respects, yet a better understanding is needed into the absolute necessity of this sophisticated sampling technique for utilization in ordinary site investigation programs, in particular, for recent sand deposits. In fact, it is a common practice to make use of reconstituted sand samples for profiling small-strain stiffness and damping of non-aged sandy deposits, although this methodology is at present not supported by any proper supporting evidence.

A case study described in this paper was carried out in order to evaluate the applicability of this approach to very recent sand deposits in which aging effects are not anticipated to be dominant to govern the soil behavior at small strains. In this study, the quality of fine sand samples retrieved by in-situ freezing technique as well as by ordinary tube samplers was carefully examined by comparing G_{max} in the laboratory to G_f from in-situ seismic surveys.

EFFECTS OF SAMPLE DISTURBANCE ON SMALL STRAIN BEHAVIOR OF RECENT SAND DEPOSITS-GENERAL

Provided that i) in-situ subsoil is reasonably uniform in terms of the particle size distribution, ii) it exhibits isotropy in G_f, iii) the G_{max} value of each laboratory specimen is obtained for a stress system similar to the current state of in-situ subsoil, and iv) the laboratory specimens are perfectly undisturbed showing also isotropy in G_{max}, the G_{max} value from the laboratory test should coincide with G_f from in-situ shear wave velocity measurement.

Figure 1 depicts measurements of G_f and G_{max} from in-situ seismic surveys and laboratory cyclic loading tests, respectively, for soil showing a cross-anisotropy in stiffness. Both downhole seismic survey and cyclic torsional

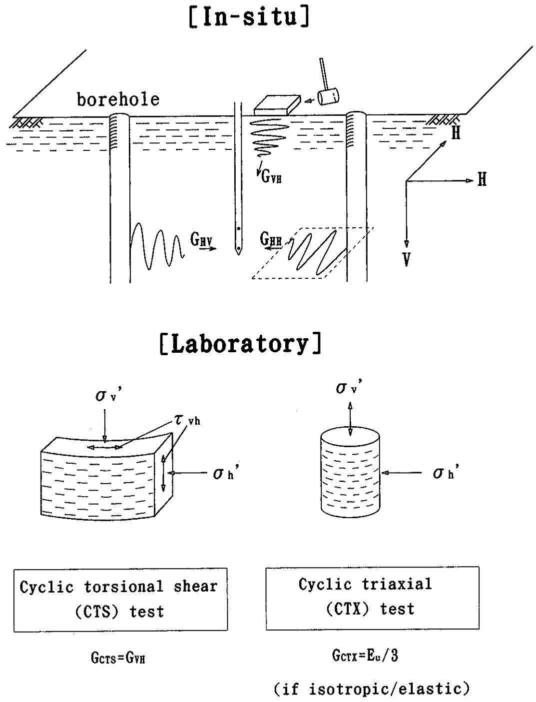

Fig. 1. Shear moduli from in-situ and laboratory tests for cross-anisotropic medium

shear (CTS) tests measured G_{vh}, whereas G_{hh} was measured in a cross-hole seismic survey only. Note that the first and second subscripts for G denote the directions of shear wave propagation and polarization, respectively. When the subsoil is isotropic implying G_{vh} equal to G_{hv} and to G_{hh}, the shear modulus can also be determined in an undrained cyclic triaxial (CTX) test through G_{max} equal to $E_{max}/3$. It should be pointed out that the CTX test is simpler in testing and more versatile in accommodating a wider spectrum of geomaterials to be tested as compared to the CTS test.

In the literature, it has been defined that G_{hh} is slightly larger than G_{vh} even for normally consolidated soils, and the ratio of G_{hh} to G_{vh} increases from about 1.2 as the K_0-value increases (Lo Presti and O'Neil, 1991; Jamiolkowski et al., 1994). It has also been demonstrated that highly-structured overconsolidated clays exhibited G_{hh} far greater than G_{vh} due to the effects of macro-fabric on stiffness (e.g., Butcher and Powell, 1995). Conversely, the results of in-situ bender element test show that uniform, uncemented and young, Holocene sand deposits exhibited G_f which was closely isotropic (Nishio, 1995).

Considering a sandy soil that is neither cemented nor significantly aged without any interparticle bonding, the quality of the laboratory samples may be assessed in a simple way by comparing G_{max} with G_f; G_{max} is greater than G_f when the void ratio of the laboratory specimen, e_{lab}, is smaller than $e_{in\text{-}situ}$, and vice versus when e_{lab} is larger than $e_{in\text{-}situ}$ (Fig. 2). This concept is valid for situations where the four conditions specified at the beginning of this section are all satisfied. The quantification of sample disturbance, however, should be accompanied by a

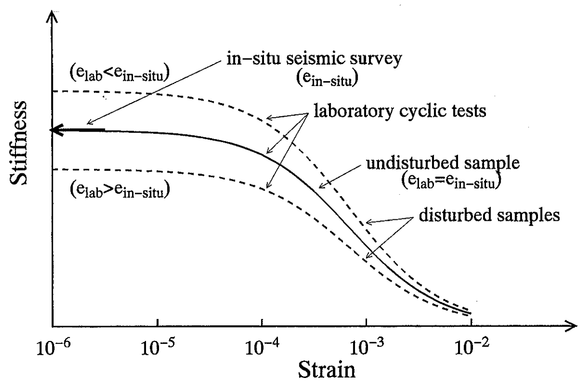

Fig. 2. Effects of sample disturbance on small strain stiffness in laboratory tests

precise estimate of void ratio profile in the subsoil that is being evaluated, through adequate site investigation technique(s) and/or the examination of high-quality samples from the subsoil profile.

When comparing G_{max} to G_f, the differences in soil density as well as the stress state between the laboratory specimen and in-situ subsoil profile ought to be correctly measured or reasonably estimated. These should then be properly taken into account in a quantitative manner, for instance, using the following expression given by Hardin and Richart (1963);

$$G=A\cdot f(e)\cdot g(K)\cdot p_r'^{-m}\cdot p'^{m} \qquad (1)$$

where A is a constant having the same unit as G. The symbols, p' and p_r', refer to mean effective stress and its reference pressure, respectively. $f(e)$ represents void ratio function, where $f(e)$ of $(2.17-e)^2/(1+e)$, for example, is considered to be valid for sandy soils. The exponent, m, ranges between 0.4 and 0.5 for a wide spectrum of clean sands (e.g., Iwasaki et al., 1978). The effects of stress ratio are not significant for a range of effective principal stress ratio, K, of σ_h'/σ_v' between 0.5 and 2.0, resulting in $g(K)=1.0$ (Tatsuoka et al., 1979).

When the strain-level dependent properties of stiffness and damping are insensitive to the effects of sample disturbance and soil fabric, which is convenient since they can be measured in tests on disturbed or even fully reconstituted samples. This can be possible for conditions where the shear modulus degradation with strain as determined by G_{max}, together with the variation of damping with strain, is not influenced by sample disturbance as much as the soil structure.

TEST SITE AND SOIL SAMPLINGS

The case study was carried out on a very young deposit of uncemented sand in Higashi-Ohgishima, a man-made island located offshore from Kawasaki city in Tokyo Bay. The test borehole records show that the site consists of an upper sand layer to a depth of 14 m below the ground surface, overlaying Holocene deposits of silt and clay. The current sea water level is about 2 m below the ground surface. The upper sand layer was placed by reclamation in 1960'. It consists of fine Sengenyama sand having sub-angular grain shape (for details, *see* Iai and Kurata, 1991; Shibuya and Tanaka, 1996). The results of grain distribution analysis are shown in Fig. 3 and in Table 1, for which the percent fines content exceeds 5 below a depth of 5 m.

Figure 4 shows soil samples obtained for this study, together with the results of SPT N-values. Two frozen sample cores, F1 and F2, each 15 cm in diameter and 7 m

Table 1. Gradation of sands

Depth (m)	D_{max} (mm)	D_{50} (mm)	U_c	F.C. (%)
3.0	9.50	0.341	2.58	4.5
5.0	9.50	0.249	2.31	6.3
7.0	4.75	0.223	2.49	7.9
10.4	4.75	0.234	2.93	9.1

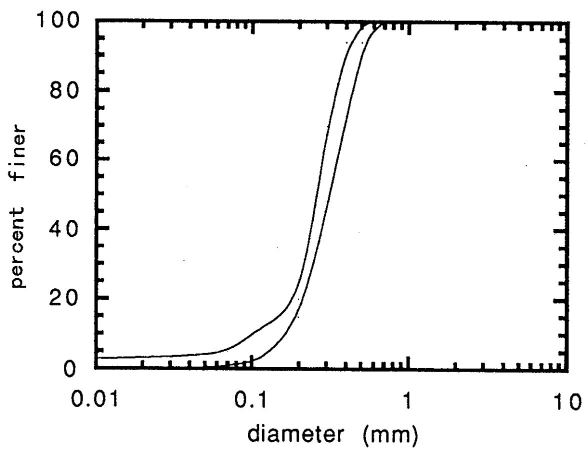

Fig. 3. Grain size distribution curves

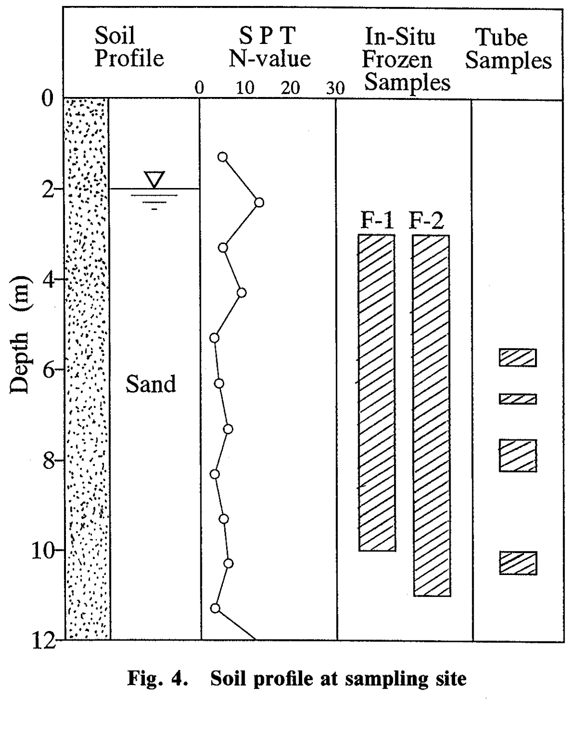

Fig. 4. Soil profile at sampling site

and 8 m long, respectively, were retrieved using of the in-situ ground freezing method (IFM). Since the basic properties were similar, no distinction was made for these cores of F1 and F2 when interpreting the test results. In the adjacent area, more sand samples were retrieved by using ordinary tube samplers; a thin-wall sampler and a triple-tube sampler. Immediately after this retrieval, the pore water in the samples was drained, and they were stored in a refrigerator, and later transported to the laboratory.

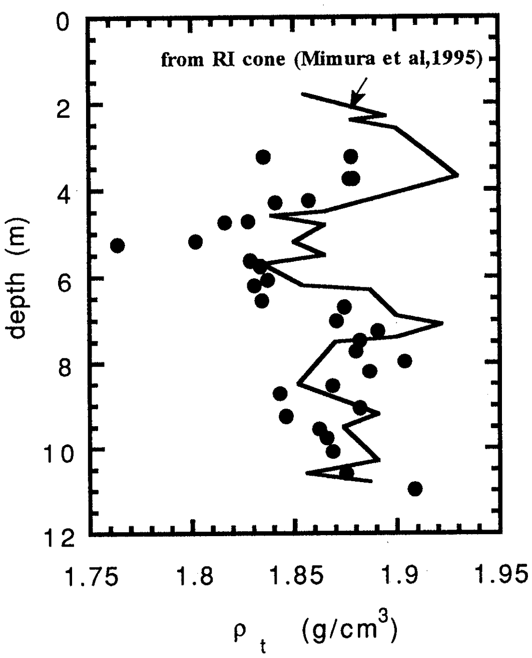

Fig. 5. Predicted and measured total density ρ_t (Mimura et al., 1995)

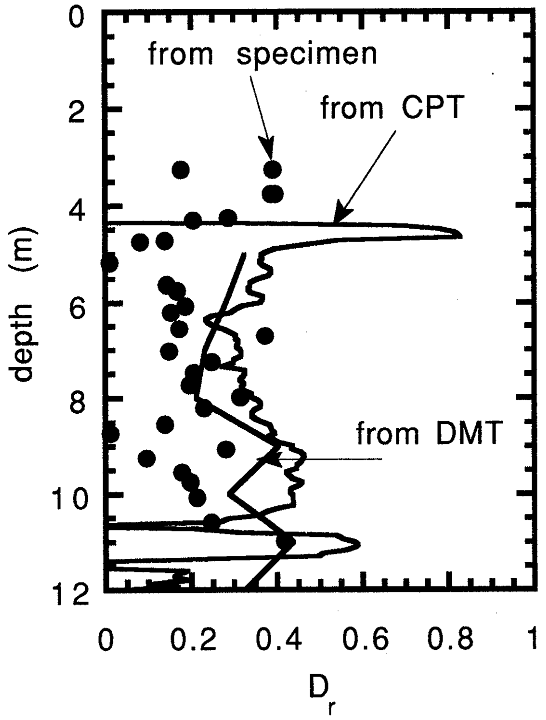

Fig. 6. Prediction of relative density using results of in-sit tests

The profile of total density, ρ_t, with depth is shown in Fig. 5, in which solid circles denote the results of direct measurement on in-situ frozen cores F1 and F2. The solid line represents the profile estimated by means of in-situ radio-isotope (RI) cone test (Shibata et al., 1994; Mimura et al., 1995). It is demonstrated that except for shallow depths about 5 m, the RI cone is capable of providing a reasonable estimate for continuous profile of ρ_t with depth. Figure 6 shows the variation of relative density, D_r, with depth. It should be noted that the deposit is very loose with a D_r ranging between 0.1 and 0.4. In this figure, the solid lines refer to the profile estimated from a flat dilatometer test (DMT, Marchetti, 1980) and from CPT test; the former is derived on the basis of the K_D value using the relationship proposed by Robertson and Campanella (1986), and the latter using the relationship of $D_r = -98 + 66 \log_{10} [0.1q_c/(0.1\sigma_v')^{0.5}]$ (c.f., q_c: cone tip resistance (in kPa), σ_v': in kPa) proposed by Lancellotta (1983). It should be noted that these in-situ estimates formed the upper bound for the results of the direct measurement.

IN-SITU AND LABORATORY TESTS PERFORMED

A downhole seismic cone test (Campanella et al., 1986) was performed using a cone with a diameter of 35.7 mm. It comprised two receivers installed one meter apart from each other. The V_s value was determined by measuring the arrival time lag of the shear wave propagation between these two receivers. The shear wave was generated on the ground surface by hammering a plank (Tanaka, H. et al., 1994).

Two types of laboratory tests; i.e., CTX and CTS tests, were performed on intact samples retrieved by IFM, and on possibly disturbed samples retrieved by the tube samplers. In addition, fully reconstituted samples were prepared by are-pluviation method, each using soil from an intact specimen recycled after testing. The CTS specimen was 3 cm and 7 cm in inner and outer diameters, and 7 cm high. A cylindrical specimen 5 cm in diameter and 10 cm high was used in the CTX test. The void ratio of the frozen specimen prior to recompression is denoted as e_0 in this paper. Each intact specimen was subject to thawing in the cell under an isotropic pressure of 30 kPa, one test case resulted in 20 kPa in the CTS test (*see* Fig. 14, $K=0.5$). It was recompressed against a back-pressure of 98 kPa to in-situ effective overburden pressure, σ_v', while K was maintained at a constant value in each test. The void ratio just prior to undrained shear is denoted as e_c. The change in volume during recompression, that is the difference between e_0 and e_c, was not significant, showing an increase of only 0.02. Cyclic loading was imposed in a stress-controlled fashion using a fixed frequency of 0.5 Hz and 0.1 Hz for the CTS and CTX tests, respectively. It should be noted that the effects of loading frequency are small for the sand behavior at small strains (Iwasaki et al., 1978). For one stage of cyclic loading, the specimen was subjected to undrained loading with a fixed amplitude of cyclic stress. The number of cycles imposed for each stage was eleven. Before the next stage with a higher magnitude of cyclic stress, the specimen was fully drained. The details of the CTX and CTS testings have been described by Yamashita et al. (1994) and Shibuya et al. (1995), respectively.

The peak-to-peak scant shear or Young's modulus, G_{eq} or E_{eq}, together with the hysteretic damping ratio, h, was examined for the tenth hysteresis loop at each stage. The definitions of these parameters have been described elsewhere (for example, *see* Toki et al., 1995).

RESULTS OF IN-SITU AND LABORATORY TESTS

Results of in-situ seismic surveys are shown in Fig. 7, in which the G_f profile with depth from the down-hole survey (i.e., G_{vh}) is denoted using squares. In this figure, the variations of G_{hv} ($=G_{vh}$ for cross-anisotropic soil) and G_{hh} from cross-hole measurements performed by Nishio (1995) are also plotted for comparison. It should be noted that for depths below 4 m, G_{hh} obtained using horizontally polarized shear wave in the cross-hole test was slightly larger than G_{hv}. The G_{hh} value, however, was in the vicinity of G_{vh} from the down-hole measurement. Based on the self-contained results of the cross-hole test, the subsoil may be anisotropic in respect of G_f, but the value of G_{hh}/G_{vh} was just slightly over unity.

Figures 8 and 9 show variations of G_{eq} and h with single amplitude cyclic shear strain, $(\gamma)_{SA}$. The values of G_{eq} and $(\gamma)_{SA}$ in the CTX tests (*see* Fig. 8) were determined as $E_{eq}/3$ and $1.5\times(\varepsilon_a)_{SA}$, respectively, based on the premise of stiffness isotropy. The following may be noted for the results of tests on in-situ frozen samples that were isotropically recompressed to in-situ σ_v':

i) Similarly to the CTX and CTS tests, the stiffness of each intact sample decreased, which was noticeable beyond $(\gamma)_{SA}$ of about 1×10^{-5},

ii) below $(\gamma)_{SA}$ of 1×10^{-5}, h showed the values less than 2%, and

iii) the relationship between h and $(\gamma)_{SA}$ was scarcely influenced by the consolidation pressure.

On the basis of these observations, the mean G_{eq} value associated with the apparent plateau with $(\gamma)_{SA}$ less than 1×10^{-5} in each test was conveniently taken as G_{max} of the specimen.

The stiffness degradation curves normalized by G_{max} for each specimen are shown in Figs. 10 and 11 for groups of CTX and CTS tests, respectively. The stiffness decreased to about a half of the initial value at $(\gamma)_{SA}$ of about 1×10^{-3}. In both of the tests, the degradation rate with strain tended to be slightly slower for specimens subjected to higher σ_v'. This difference may not be significant, however, in a quantitative sense when the results are used for seismic response analysis.

A comparison is made in Fig. 12, in which the relationship between G_{eq}/G_{max} and $(\gamma)_{SA}$ is shown for the results of the CTX and CTS tests. It should be noted that the difference in stiffness degradation with shear strain was not significant between the CTX and CTS tests.

The effects of effective stress ratio attained by following different consolidation stress paths can be seen in Figs. 13 and 14. In the CTX tests, the effects on the

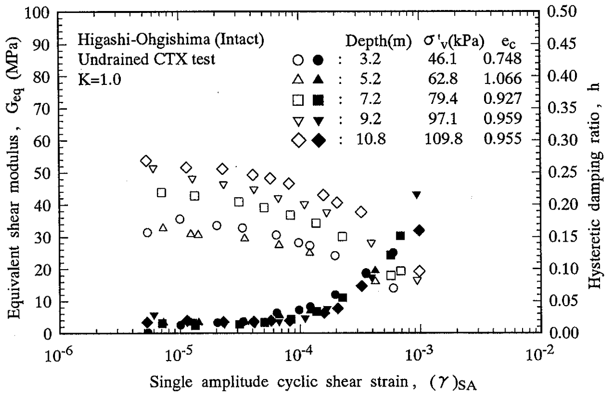

Fig. 8. Results of undrained cyclic triaxial (CTX) test on IFM samples

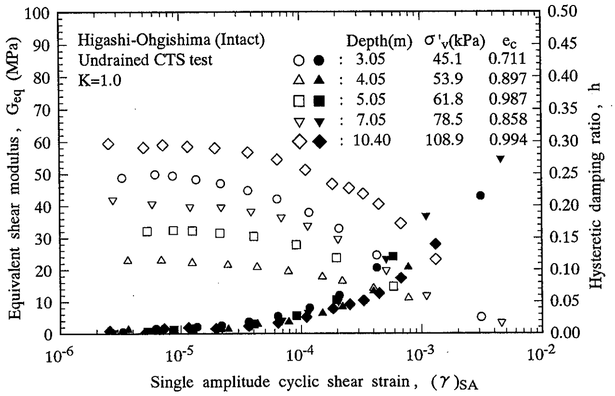

Fig. 9. Results of undrained cyclic torsional shear (CTS) test on IFM samples

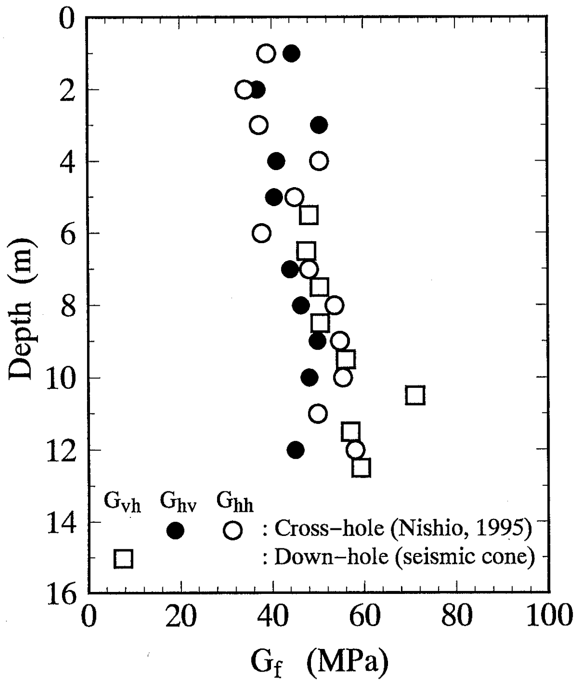

Fig. 7. Stiffness anisotropy of the fill sand from in-situ seismic surveys

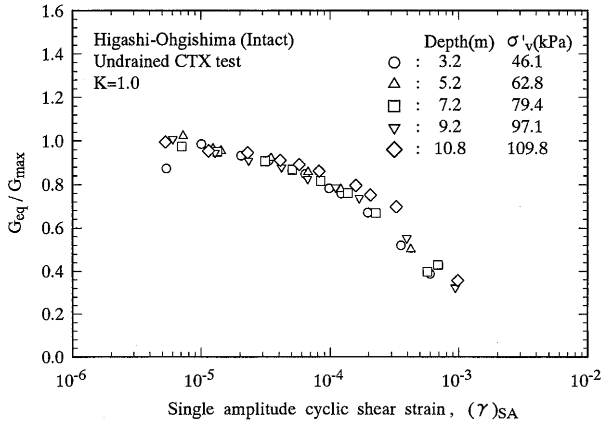

Fig. 10. Shear moduli normalized by G_{max} with single amplitude cyclic shear strain in CTX test

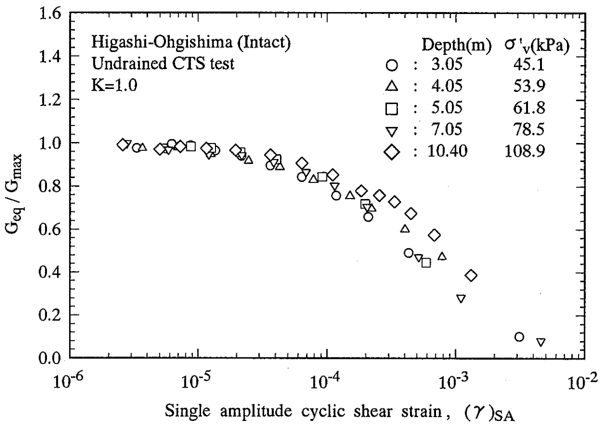

Fig. 11. Shear moduli normalized by G_{max} with single amplitude cyclic shear strain in CTS test

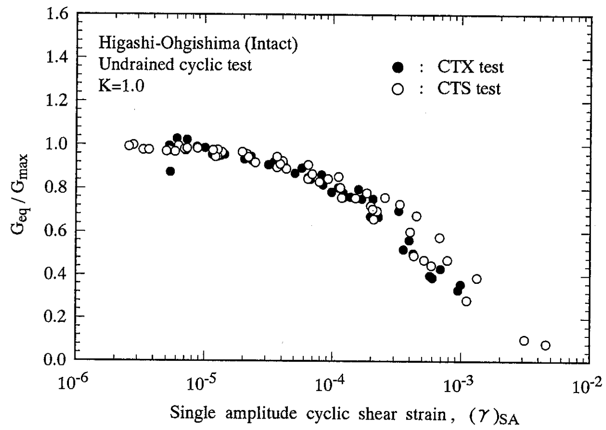

Fig. 12. Relationship between G_{eq}/G_{max} and log $(\gamma)_{SA}$ for CTX and CTS tests

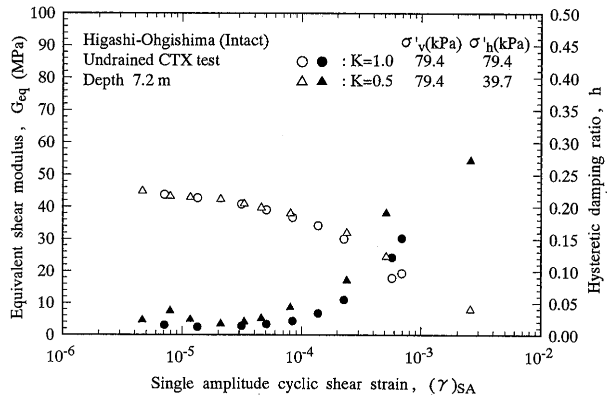

Fig. 13. Effects of stress state on shear modulus and damping ratio in CTX test

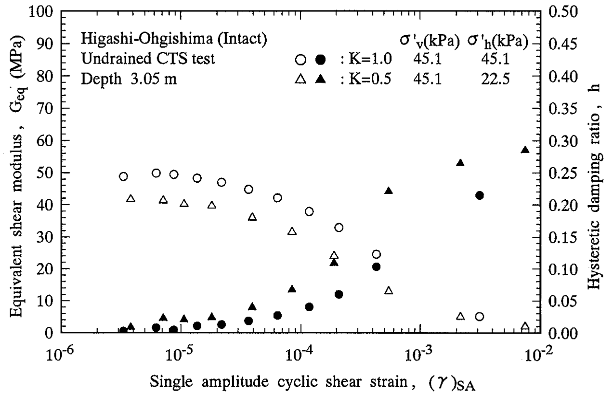

Fig. 14. Effects of stress state on shear modulus and damping ratio in CTS test

relationship between G_{eq} and $(\gamma)_{SA}$ for strains between 10^{-5} and 10^{-3} was marginally small as observed for each set of comparative specimens having a similar density, but subjected to isotropic and anisotropic consolidation histories with $K=1$ and 0.5, respectively (*see* Fig. 13). In most of the CTS tests, however, the effects were noticeable with respect to the relationship between G_{eq} and $(\gamma)_{SA}$ for the anisotropically consolidated specimen positioned below the results of the isotropically consolidated specimen (*see* Fig. 14). For both of the CTX and CTS tests, h was larger in the anisotropically consolidated specimen than in the isotropically consolidated specimen, the tendency of which was observed for $(\gamma)_{SA}$ beyond 5×10^{-5}.

The small-strain behavior in the CTX tests on tube-samples are shown in Fig. 15, in which the results of tests on in-situ frozen samples at similar depths are also shown for comparison. The void ratio of the tube samples was slightly smaller than for the in-situ frozen sample at the relevant depth, possibly due to densification occurring during the tube sampling. As a result, the G_{eq} value of the tube sample was apparently larger for the entire strain levels examined (*see* Fig. 15(a)). The stiffness degradation curves when examined using each G_{max}, however, were similar between the tube samples and the in-situ frozen sample for the overall depths examined (*see* Fig. 15(b)). Figure 16 shows a comparison of the small-strain behavior between in-situ frozen sample and the fully reconstituted sample. It should be noted that the small-strain behavior was quantitatively similar for these samples. It should be noted that the void ratio of these samples was very close to the average e_{max} of 1.085 for which the difference in e by 0.06 is expected to not greatly influence the stiffness. If in-situ frozen samples are per-

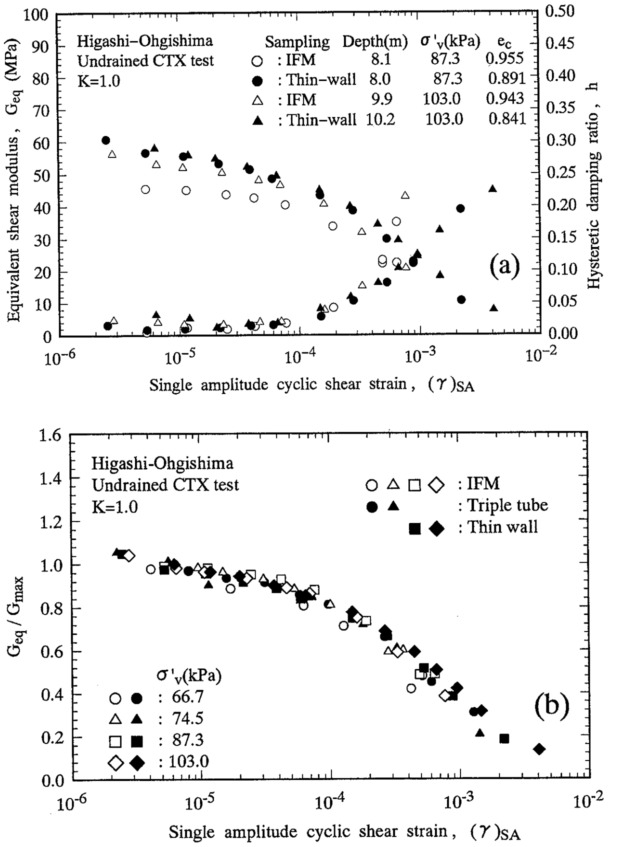

Fig. 15. Effects of sampling method on cyclic deformation properties; (a) G_{eq} vs. log $(\gamma)_{SA}$, (b) G_{eq}/G_{max} vs. log $(\gamma)_{SA}$

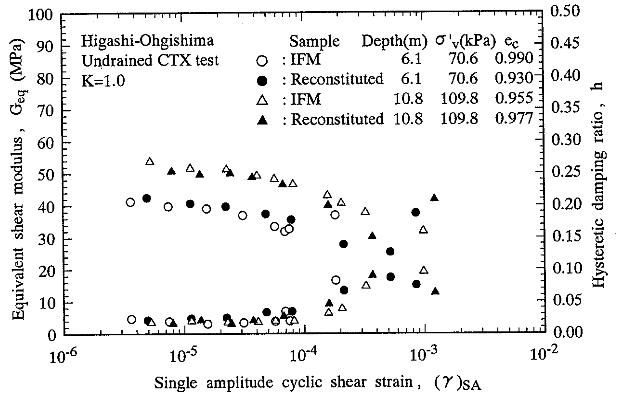

Fig. 16. Comparisons of in-situ frozen and the reconstituted samples

fectly undisturbed, the results imply that aging effects are not significant for the small strain behavior of this recent sand deposit only about thirty years old.

The test results shown in Figs. 8 through to 16 clearly indicate that the relationship between G_{eq}/G_{max} and $(\gamma)_{SA}$ was remarkably stable for this sand deposit, which was hardly influenced by the structural and density changes associated with the soil sampling, and also resulting from the consolidation history. In the following discussion, the quality of the laboratory samples is evaluated through comparisons between G_{max} and G_f, with attention paid to the difference in density, together with stiffness anisotropy.

DISCUSSIONS

Comparison of Pseudo-Elastic Shear Modulus between In-Situ and Laboratory

Figure 17 shows the values of $G_f/f(e_0)$ and $G_{max}/f(e_c)$ plotted against in-situ σ_v' and the consolidation pressure σ_c', respectively. It should be noted that G_f was determined from an averaged ρ_t of 1.85 g/cm^3 in the subsoil over the entire depths studied. Similarly, the in-situ void ratio for G_f was derived from the direct measurement of ρ_t on cores of in-situ frozen sample (*see* data points in Fig. 5), together with an average value of soil particle density ρ_s of 2.71 g/cm^3. The void ratio of the laboratory specimens prior to shearing, e_c, was determined from the local values of ρ_t and ρ_s along the depth for each sample. As previously stated, the decrease in void ratio during consolidation of the laboratory specimens was at most 0.02, which included apparent contraction due to membrane penetration effects. The upper solid line in Fig. 17 represents the relationship between G_f and σ_v', which was derived as a result of a linear regression applied to the results of in-situ seismic surveys. This is given by;

$$G_f=12{,}500(2.17-e)^2/(1+e)\cdot\sigma_v'^{0.4} \quad \text{(in kPa).} \tag{2}$$

The G_{max} values in laboratory tests show substantial scatter. The lower broken line in Fig. 17 was obtained in a similar manner for the results of CTX tests; represented by the equation:

$$G_{max}=8{,}500(2.17-e)^2/(1+e)\cdot\sigma_c'^{0.45} \quad \text{(in kPa).} \tag{3}$$

The CTX test underestimated G_f by 10 to 20 percent. Obviously, the data shows significant scatter for σ_{vlab}' below 60 kPa (see Fig. 17).

The comparison described above was made for the results of tests performed on isotropically consolidated

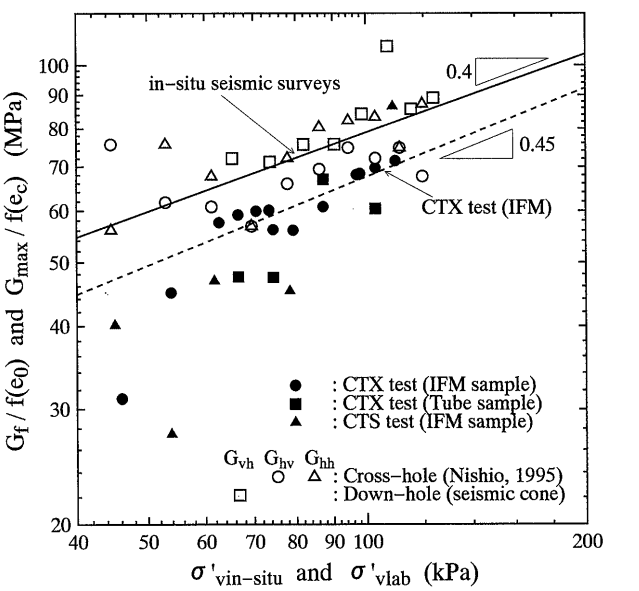

Fig. 17. Comparisons of G_f from in-situ seismic surveys and G_{max} from laboratory tests with σ_v'

specimens. Figure 18 shows E_{max} in undrained CTX tests for two groups of specimens consolidated isotropically with $K=1.0$ and anisotropically with $K=0.5$. Each data point represents the result of comparative specimens at the similar depth subjected to consolidation to σ'_v in common. For both in-situ frozen and fully reconstituted samples, the effect of the consolidation state was not significant on the undrained E_{max}. By contrast in the undrained CTS test, G_{max} of the anisotropically consolidated specimens tended to be larger than G_{max} of isotropically consolidated specimens (Fig. 19). Considering Eq. (1), the ratio of shear modulus between the isotropically consolidated specimen and the anisotropically consolidated specimen, $G_{(K=0.5)}/G_{(K=1)}$, should be $[(1+2K)/3]^m$ when σ'_v is in common. Noting that $m=0.45$ (see Fig. 17), the ratio of $G_{(K=0.5)}/G_{(K=1)}$ is expected to be approximately 0.83. The values of $G_{max(K=0.5)}/G_{max(K=1)}$ in CTS tests show scatter around 0.83 (*see* Fig. 19), whereas the similar results of CTX tests tend to be close to an $E_{max(K=0.5)}/E_{max(K=1)}$ value of unity.

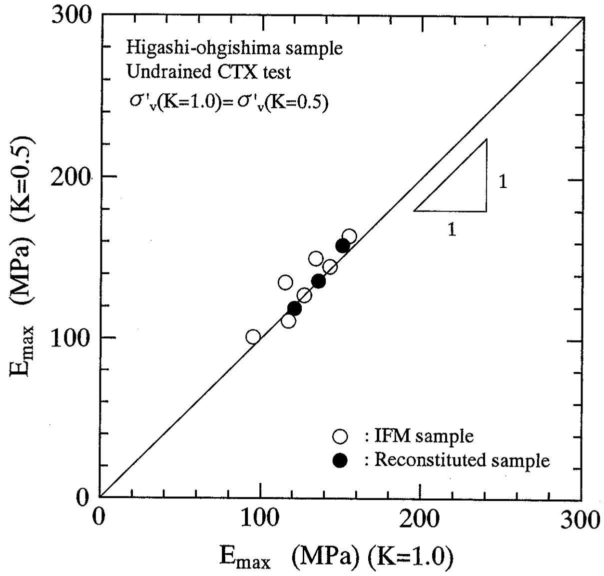

Fig. 18. Effects of stress state on E_{max} in CTX test

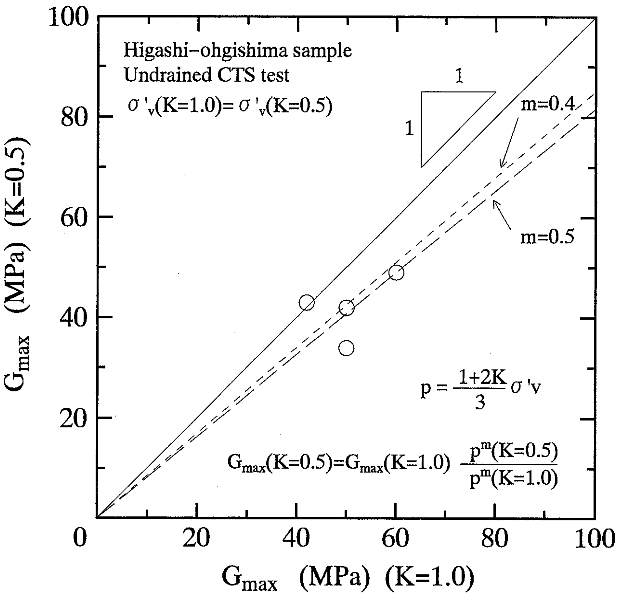

Fig. 19. Effects of stress state on G_{max} in CTS test

The results from the CTS and CTX tests appear to contradict each other when considering the effects of current stress state on G_{max}, for which G_{max} in the CTX test was determined on based on the assumption of stiffness isotropy. It should be pointed out, however, that G_{max} equivalent to $E_{max}/3$ in the CTX test was obviously larger than $G_{max}(=G_{vh})$ from the CTS test (*see* Fig. 17). This suggests stiffness anisotropy seen for isotropically consolidated specimens. If so, the coincidence of undrained E_{max} between the isotropically and anisotropically consolidated specimens in the CTX test does not imply $G_{max(K=0.5)}/G_{max(K=1)}$ of unity.

The underestimation of G_f from G_{max} in CTX and CTS tests does not mean inferior quality in-situ frozen samples, since some uncertainties were involved with density measurements of both in-situ subsoil and the laboratory specimens. The difference in value between G_f and G_{max} of about 20 percent may be due to the different strain levels examined; G_f corresponds to shear strain of 1×10^{-6} or less, whereas G_{max} is conveniently defined as the stiffness at $(\gamma)_{SA}$ of about 5×10^{-5} on an average. In fact, G_{eq} as measured in the laboratory indicated tendency to increase for a range of $(\gamma)_{SA}$ below 1×10^{-5} and above (*see* Figs. 8 and 9), in which the stiffness measurement on the laboratory specimen was difficult due to the limitation of the transducers' resolution. Above all such considerations, it is important to point out that the G_f profile with depth gives the upper bound for the maximum shear modulus when the density of each laboratory specimen is properly taken into account.

Characterizing Cyclic Deformation Properties of a Young Sand Deposit without Retrieving a High-Quality Sample

It has been shown in Fig.16 that the small-strain behavior was almost identical between in-situ frozen and reconstituted samples prepared with a void ratio similar to the frozen sample. It was also demonstrated in Fig. 15 that the strain-level dependencies of both G_{eq} and h in tests on disturbed sample isotropically consolidated to in-situ σ'_v were practically identical to those observed in tests on the in-situ frozen sample. These observations encourage exclusion of the high-quality sample to be provided for laboratory testing as a part of site characterization scheme. In such situations, the small-strain deformation properties can be measured or reasonably estimated through the procedure described as follows:

a) Estimate density profile with depth by means of in-situ tests, for example RI cone (*see* Fig. 5), DMT or CPT (*see* Fig. 6),

b) measure the G_f profile with depth by performing an in-situ seismic survey, or make an estimate of it using empirical relationship(s),

c) obtain sand samples using a conventional tube sampler,

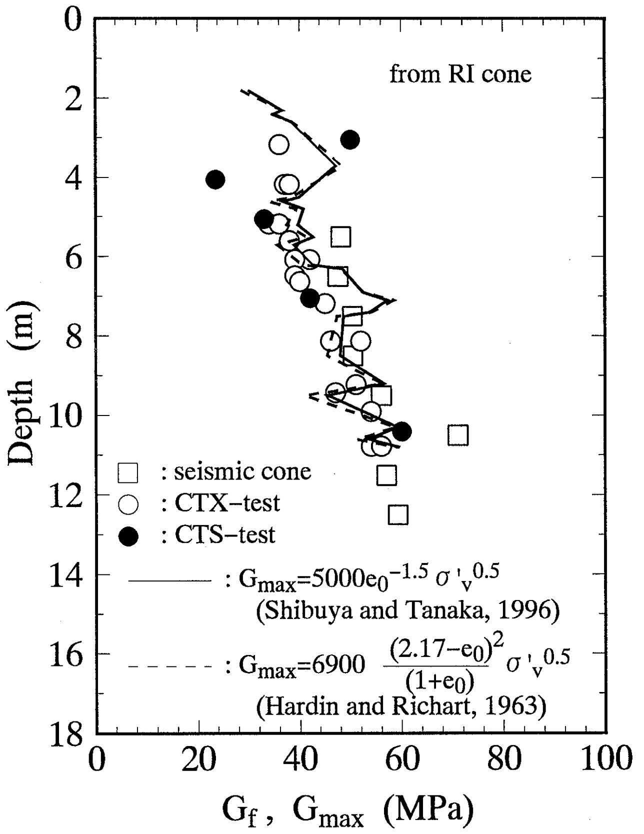

Fig. 20. Predictions of G_f using empirical equations proposed by Shibuya and Tanaka (1996) and Hardin and Richart (1963)

d) measure the variations of G_{eq} and h with $(\gamma)_{SA}$ including those at $(\gamma)_{SA}$ less than 1×10^{-5} in undrained CTX test or in CTS test (i.e., measure E_{max} or G_{max}) on reconstituted specimens, each having the similar density with in-situ soil, subjected to isotropic consolidation to in-situ σ_v',

e) obtain the variations between G_{eq}/G_{max} and h with $(\gamma)_{SA}$ at different depths, and

f) estimate in-situ shear modulus assuming $(G_{eq})_{in\text{-}situ}=(G_{eq}/G_{max})_{lab}\times G_f$.

Figure 20 shows the G_f profile with depth estimated using empirical relationships; i.e., $G_f=5{,}000e^{-1.5}\sigma_v'^{0.5}$ (in kPa) (Shibuya and Tanaka, 1996) and $G_f=6{,}900[(2.17-e)^2/(1+e)]\sigma_v'^{0.5}$ (in kPa), the version with σ_0' of which has been proposed by Hardin and Richart (1963). In these estimates, the void ratio profile with depth was determined from the results of RI cone tests (*see* Fig. 5). It was successfully demonstrated that the G_f profile estimated through steps a) and b) matches well with the results of an in-situ seismic survey. The outcome is as good as the results of careful laboratory tests using high-quality samples retrieved by in-situ ground freezing. This encourages the use of the proposed method in geotechnical engineering practice due to its cost-effectiveness.

CONCLUSIONS

Profiles of small-strain stiffness and damping of an uncemented sand deposit in Tokyo Bay was investigated with laboratory cyclic loading tests on samples retrieved by in-situ ground freezing, and also with conventional tube samplers. Examination into basic properties of the frozen core samples showed that the deposit was remarkably uniform with little variation of gradation with depth, and very loose with relative density ranging between 0.1 and 0.4. The percent fines content gradually increased from 5 adjacent to the surface to about 10 at a depth of 12 m.

Results of the in-situ seismic surveys showed that the deposit was slightly anisotropic in terms of the pseudo-elastic shear modulus, G_f. The G_f profile with depth was such that G_f increased with in-situ effective overburden pressure, σ_v', increase in value with a power of about 0.4; the value of the exponent was close to the exponent which describes stress-level dependency of G_{max} in non-aged clean sands.

Results of undrained cyclic triaxial (CTX) tests as well as undrained cyclic torsional shear (CTS) tests showed that the pseudo-elastic shear modulus, G_{max}, was observed for shear strains less than 0.001%, with limiting values of hysteretic damping ratio less than 2%. The tests performed on samples retrieved by in-situ ground freezing method underestimated G_f by 10 to 20 percent. The results were similar to tests on samples retrieved by ordinary tube samplers, implying that the G_f profile with depth formed the upper bound of G_{max} data when the density of each laboratory sample was properly taken into account.

It was also found, that the undrained cyclic deformation properties were almost identical between the in-situ frozen sample and its reconstituted sample that was prepared by are-pluviation to a void ratio similar to the frozen sample. In addition, the undrained CTX test on the reconstituted samples as well as on the tube samples, both isotropically consolidated to in-situ σ_v', resulted in strain-level dependencies of both G_{eq} and h that were similar to the results of the CTS test on high-quality in-situ frozen samples.

It was demonstrated that high-quality samples are not an absolute necessity when evaluating the undrained deformation properties of a recent uncemented sand deposit. Instead, procurement of grossly disturbed sand samples continuously with depth are required, each prepared with void ratio similar to in-situ subsoil, which can be tested either in triaxial or in torsional shear using the specimens isotropically consolidated to in-situ effective overburden pressure. With the known profile of void ratio from adequate in-situ tests, for example, RI cone test, the use of empirical equations for G_f worked successfully in estimating the G_f profile in the deposit.

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REFERENCES

1) Butcher, A. P. and Powell, J. J. M. (1995):" The effects of geological history on the dynamic stiffness in soils," Proc. 11th European Regional Conf. on Soil Mech. and Found. Engrg., Vol. 1, pp. 27–36.
2) Campanella, R. G., Robertson, P. K. and Gillespie, D. G. (1986): "Seismic cone penetration test," Use of In Situ Tests in Geotech. Engrg., ASCE GSP 6, ASCE, New York, pp. 116–130.
3) Hardin, B. O. and Richart, F. E. (1963): "Elastic wave velocities in granular soils," J. of Soil Mech. and Found. Div., Proc. ASCE. Vol. 89, No. SM1, pp. 33–65.
4) Hatanaka, M., Sugimoto, M. and Suzuki, Y. (1985): "Liquefaction resistance of two alluvial volcanic soils sampled by in-situ freezing," Soils and Foundations, Vol. 25, No. 3, pp. 49–63.
5) Hatanaka, M., Uchida, A. and Oh-Oka, H. (1995): "Correlation between the liquefaction strengths of saturated sands obtained by in-situ freezing method and rotary-type triple tube method," Soils and Foundations, Vol. 35, No. 2, pp. 67–75.
6) Iai, S. and Kurata, E. (1991): "Pore pressures and ground motions measured during the 1987 Chiba-Toho-Oki Earthquake," Technical Note of the Port and Harbour Research Institute, No. 718, pp. 3–18 (in Japanese).
7) Iwasaki, T., Tatsuoka, F. and Takagi, Y. (1978): "Shear moduli of sands under cyclic torsional shear loading," Soils and Foundations, Vol. 18, No. 1, pp. 39–56.
8) Jamiolkowski, M., Lancellotta, R. and Lo Presti, D. C. F. (1994): "Remarks on the stiffness at small strains of six Italian clays," Prefailure Deformation of Geomaterials, Shibuya, S., Mitachi, T. and Miura, S. (eds.), Balkema, Vol. 2, pp. 817–836.
9) Lancellotta, R. (1983): "Analisi di affidabilita in ingegneria geotecnica. Atti Instituto Scienza Costruzioni," No. 625—Politecnico di Torino (in Italian).
10) Lo Presti, D. C. F. and O'Neil, D. A. (1991):" Laboratory investigation of small strain modulus anisotropy in sands," Proc. ISOCCT1, Clarkson Univ., Posdam, N.Y., Huang (ed.), Elsevier, pp. 213–224.
11) Mimura, M., Shibata, T. and Nobuyama, M. (1995): "Application of RI cone penetrometer to a reclaimed loose sand deposits and liquefaction assessment," Proc. 30th Japan National Conf. on Soil Mech. and Found., pp. 287–290 (in Japanese).
12) Marchetti, S. (1980): "In situ tests by flat dilatometer," J. of Geotech. Engrg. Div., ASCE, Vol. 106, No. GT3, pp. 299–321.
13) Nishio, S. (1995): "Shear wave anisotropy in crosshole seismic survey," Proc. 30th Japan National Conf. on Soil Mech. and Found., pp. 331–332 (in Japanese).
14) Robertson, P. K. and Campanella, R. G. (1986): "Estimating liquefaction potential of sands using the flat plate dilatometer," Can. Geotech. J., Vol. 9, No. 1, pp. 38–40.
15) Satitharan, S., Robertson, P.K. and Sego, D. C. (1994): "Sample disturbance from shear wave velocity measurement," Can. Geotech. Test. Jour., Vol. 31, pp. 119–124.
16) Shibata, T., Mimura, M. and Shrivastava, A. K. (1994): "Use of RI-cone penetrometer in foundation engineering," Proc. 13th Int. Conf. on Soil Mech. and Found., New Delhi, Vol. 1, pp. 147–150.
17) Shibuya, S., Tatsuoka, F., Teachavorasinskun, S., Kong, X. J., Abe, F., Kim, Y-S. and Park, C-S. (1992): "Elastic deformation properties of geomaterials," Soils and Foundations. Vol. 32, No. 3, pp. 26–46.
18) Shibuya, S., Mitachi, T., Fukuda, F. and Degoshi, T. (1995): "Strain rate effects on shear modulus and damping of normally consolidated clay," Geotech. Testing Jour., Vol. 18, No. 3, pp. 365–375.
19) Shibuya, S. and Tanaka, H. (1996): "Estimate of elastic shear modulus in Holocene soil deposits," Soils and Foundations, Vol. 36, No. 4, pp. 45–56.
20) Singh, S., Seed, H. B. and Chan, C. K. (1982): "Undisturbed sampling of saturated sands by freezing," J. of Geotech. Engrg. Div., ASCE, Vol. 108, No. GT2, pp. 247–264.
21) Tanaka, H., Tanaka, M., Iguchi, H. and Nishida, K. (1994): "Shear modulus of soft clay measured by various kinds of tests," Pre-failure Deformation of Geomaterials, Shibuya, S., Mitachi, T. and Miura, S. (eds.), Balkema, Vol. 1, pp. 235–240.
22) Tanaka, Y., Kudo, K., Nishi, K. and Okamoto, T. (1994): "Shear modulus and damping ratio of gravelly soils measured by several methods," Pre-failure Deformation of Geomaterials, Shibuya, S., Mitachi, T. and Miura, S. (eds.), Balkema, Vol. 1, pp. 47–54.
23) Tatsuoka, F., Iwasaki, T., Fukushima, S. and Sudo, H. (1979): "Stress conditions and stress histories affecting shear modulus and damping of sand under cyclic loading," Soils and Foundations, Vol. 19, No. 2, pp. 29–43.
24) Tatsuoka, F. and Shibuya, S. (1992): "Deformation characteristics of soils and rocks from field and laboratory tests," Keynote paper, Proc. 9th Asian Regional Conf. on Soil Mech. and Found. Engrg., Vol. 2, pp. 101–170.
25) Toki, S., Shibuya, S. and Yamashita, S. (1995): "Standardization of laboratory test methods to determine the cyclic deformation properties of geomaterials in Japan," Keynote Paper, Pre-failure Deformation of Geomaterials, Shibuya, S., Mitachi, T. and Miura, S. (eds.), Balkema, Vol. 2, pp. 741–784.
26) Yamashita, S. and Toki, S. (1994): "Cyclic deformation characteristics of sands in triaxial and torsional tests," Pre-failure Deformation of Geomaterials, Shibuya, S., Mitachi, T. and Miura, S. (eds.), Balkema, Vol. 1, pp. 31–36.
27) Yoshimi, Y., Hatanaka, M. and Oh-Oka, H. (1978): "Undisturbed sampling of saturated sands by freezing," Soils and Foundations, Vol. 18, No. 3, pp. 59–73.
28) Yoshimi, Y., Tokimatsu, K., Kaneko, O. and Makihara, Y. (1984): "Undrained cyclic shear strength of a dense Niigata sand," Soils and Foundations, Vol. 24, No. 4, pp. 131–145.
29) Yoshimi, Y., Tokimatsu, K. and Hosaka, Y. (1989): "Evaluation of liquefaction resistance of clean sands based on high-quality undisturbed samples," Soils and Foundations, Vol. 29, No. 1, pp. 93–104.
30) Yoshimi, Y., Tokimatsu, K. and Hosaka, Y. (1994): "In situ liquefaction resistance of clean sands over a wide density range," Géotechnique, Vol. 44, No. 3, pp. 479–494.