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A study on small strain shear modulus of undisturbed soft marine clays and its correlations with other soil parameters

Etude du module de cisaillement à faibles déformations d'argiles marines molles et de leurs corrélations avec d'autres paraméres des sols

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ABSTRACT: A study on small strain shear modulus (G_0) of naturally deposited fine grained soft marine soils on five soil deposits, namely, Ariake and Hachirogata of Japan, Pusan of Korea, Bangkok of Thailand and Bothkennar of UK, has been done using field seismic cone tests and also laboratory bender element tests at insitu stress states as well as isotropically normally consolidated stress states. Based on these and other data obtained from several other sites, general characteristics of G_0 and its correlations with other soil parameters have been examined. Finally, relation of the G_0 with soil parameters measured by the field vane, piezocone and dilatometer tests have been investigated using high quality insitu data acquired from twelve different fine grained soils deposits from various parts of the earth.

RÉSUMÉ: Une étude sur le module de cisaillement à faibles déformations (G_0) a été réalisée sur des sols argileux d'origine marine, tels que ceux de Ariake et Hachirogata (Japon), Pusan (Corée), Bangkok (Thaïlande), et Bothkenar (UK). L'étude comportait l'utilisation d'un cône sismique de terrain ainsi que des essais en laboratoire réalisés, à l'aide languettes piézoélectriques, à des contraintes insitu ainsi qu'à des conditions de contraintes de consolidation isotrope. À partir des résultats obtenus, les caractéristiques générales du G_0 ainsi que ses corrélations avec d'autres paramètres des sols ont été examinées. Finalement, la relation entre G_0 et les paramètres des sols mesurés au scissomètre de chantier, au piézocone, ainsi qu'au dilatomètre, a été analysée par l'utilisation de données insitu d'excellente qualité et qui ont été acquises de douze sites différents de dépôts fins localisés dans diverses régions du monde.

1 INTRODUCTION

Small strain shear stiffness (G_0) is a fundamental soil parameter in characterizing non-linear stress-strain behavior of a soil, both in the monotonic and dynamic loadings and related shear deformation problems. Particularly, geotechnical works subjected to dynamic loading such as earthquake, wave-loads, traffic-loads and machine foundations demand accurate estimate of small strain shear deformation. When shear strain is smaller than the elastic threshold strain of about 10^{-5}, shear modulus (G) is practically constant and is independent of shear strain level (Hardin, 1978; Burland, 1989; Tatsuoka and Shibuya, 1991). Various types of dynamic tests are available in measuring small strain shear modulus of a soil. It has been established that G_0 measured using field seismic cone and bender element test yield small strain shear modulus (i.e. $G = G_0$) of a soil. G_0 of a soil can be measured in field and laboratory by passing elastic shear wave of velocity v_s through soil having length L, density ρ and recording its travel time t, by using the following relationship:

$$G_0 = \rho\, v_s^2 = \rho\, (L/t)^2 \qquad (1)$$

This paper investigates G_0 in the following naturally sedimented undisturbed soft marine clays using field seismic cone tests, and laboratory bender element tests at the insitu stress state and also at isotropically normally consolidated stress state: Ariake and Hachirogata (Japan), Pusan (Korea), Bangkok (Thailand) and Bothkennar (United Kingdom). Based on the data obtained from these tests, and also field seismic cone data from several other sites, general characteristics of G_0 and its correlations with other soil parameters have been examined. Furthermore, correlation between G_0 and other relevant soil parameters measured by the field vane, piezocone and dilatometer tests and index tests have been investigated using high quality data acquired from twelve different fine grained soil deposits from various parts of the world.

2 EXPERIMENTAL INVESTIGATIONS

Over the past decade, Geotechnical Investigation Laboratory of Port and Harbour Research Institute (PHRI) has conducted extensive site investigations works of soft soil deposits both inside Japan (such as Ariake, Hachirogata, Yamashita, Kurihama, Ogishima etc.), as well as outside Japan (such as Korea (Pusan), Thailand (Bangkok), UK (Bothkennar), Canada (Louiseville), Norway (Drammen), Singapore etc.) while observing the unified and identical conditions for sampling, testing and interpretation of the results. Commonly performed investigations include field tests such as seismic cone, field vane, piezocone (CPTU), dilatometer, etc. Besides, comprehensive laboratory tests have been performed on these soils. Investigations at some of the sites have been done in collaboration with corresponding local institute(s). Field data for Louiseville clay have been referred from the works of Laval University (Roy, 1995), while that of Singapore has been obtained as a part of the collaborative research with National University of Singapore. Similarly, the data for the Bangkok clay are part of collaborative research works between PHRI and Asian Institute of Technology. Except for the bender element test data, other data for the Bothkennar site has been referred from Nash *et al.* (1992). This paper focuses on small strain shear modulus and its correlations with relevant soil parameters obtained from various field tests.

Soil sampling at all the sites have been done using the standard Japanese fixed piston samplers and Japanese sampling technique. Thus, by using the identical and high quality Japanese sampling method at all the sites, the variation in sample quality owing to the sampling technique and sampler type has been eliminated (Tanaka *et al.*, 1996). In all the site investigation works, soil samples were transported to PHRI from the investigation sites to conduct further laboratory tests. Soil samples from outside Japan were transported by air cargo. It has been confirmed that samples transported in this way do not get disturbed during the transportation (Tanaka H. & Tanaka M., 1997).

Table 1. Typical basic characteristics of the soil deposits studied

Soil Deposit	w_n (%)	w_L (%)	I_p	ρ_s	%< 2μm	OCR	G0 (MPa)
Hachirougata [J]	130-205	135-235	85-150	2.39-2.65	30	1.0-1.2	5
Ariake [J]	90-180	80-175	50-115	2.60-2.66	45	1.2-1.7	8
Kurihama [J]	75-95	75-115	45-75	2.67-2.70	52	1.1-1.8	12
Izumo [J]	70-110	75-150	40-105	2.61-2.72	45	0.8-1.1	13
Yamashita [J]	50-100	75-110	40-75	2.65-2.70	35	1.8-2.0	18
Ogishima [J]	45-75	50-95	30-55	2.65-2.68	28	1.2-1.5	43
Bangkok	55-80	45-85	30-70	2.72-2.77	50	1.3-1.8	15
Pusan	45-65	55-70	30-45	2.70-2.73	44	0.9-1.2	18
Louiseville [R]	65-75	70-80	45-55	2.75-2.79	55	2.2-2.4	20
Bothkennar	50-70	55-75	35-45	2.69-2.72	28	1.8-2.3	24 [N]
Drammen	30-40	35-45	16-24	2.79-2.81	40	1.1-2.0	30 [I]
Singapore	45-60	60-75	40-60	2.76-2.78	65	1.1-1.4	44
Mexico City	500 [M]	500 [M]	350 [M]	2.35 [M]	25 [M]	1.5 [M]	7.5 [D]

J: Japanese soils; [N]: Nash *et al.* (1992); [M]: Mesri *et al.* (1975); [D]: Diaz-Rodriguez *et al.* (1998); [I]: Larsson & Mulabdic (1991); [R]: Roy (1995)

All seismic cone tests performed by PHRI, are down-hole type, in which shear wave is generated in the field by hitting a wooden plank on the ground surface with a hammer and by measuring the resulting shear wave detected by receivers installed at the tip of the cone (Tanaka *et al.*, 1994).

Laboratory bender element tests were performed in a special triaxial apparatus, which houses bender elements developed by Norwegian Geotechnical Institute (Dyvik and Madshus, 1985). Bender elements were set into the upper cap and the lower pedestal of the triaxial apparatus, with the offset of 4.5 mm at each end. Nominal size of specimen was 50 mm in diameter and 55 mm in length. Load cell was fitted inside the triaxial cell to avoid the influence of piston friction while measuring axial pressure.

Soil specimen in the triaxial cell was subjected to anisotropic insitu stress state, assuming K_0 of 0.5. Backpressure of 200 kPa was applied to soil specimen throughout the test to assure the high degree of saturation. At the end of the insitu stress application, specimen was subjected to incremental isotropic consolidation pressure of upto 600 kPa, with a step loading of 100 kPa. For each step of loading, wave velocity measurements were done at the end of primary consolidation.

Square wave was generated at a frequency of 50 Hz having 10 V amplitude and G_0 was calculated using the equation 1 described earlier by using the length between tips of the bender elements as the effective length for shear wave calculation, as recommended by Viggiani and Atkinson (1995). Arrival of shear wave was taken based on the first major reversal of polarity of the received signal.

3 G_0 FROM FIELD SEISMIC CONE TESTS AND LABORATORY BENDER ELEMENT TESTS

3.1 *Site characteristics of tested soil deposits*

Table 1 shows the typical characteristics of various soils studied.

Ariake site (Figure 1), is one of the most extensively studied soft clay deposits in Japan, whose details can be found from Hanzawa, *et al.* (1990); Ohtsubo, *et al.* (1995) and Tanaka *et al.*, (1996). This site has liquidity index of about unity, and is rich in diatom microfossils.

Hachirogata site (Figure 2) is located in Akita prefecture of Japan and is characterized by the abundance of cylindrical shaped diatoms, resulting in relatively low unit weight and very large water bearing capacity (Shiwakoti *et al.*, 2001).

Pusan site (Figure 3) is located along the Yangsan delta of Pusan in Korea, and has been extensively studied by PHRI in collaboration with Pusan National University. Field testing and sampling of this site was done a few months after the ground was loaded with fill of about 1.8 m. This site is normally consolidated or slightly under-consolidated (Tanaka *et al.* 2001b).

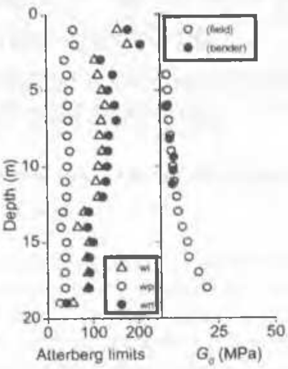

Figure 1. Site profile of Ariake

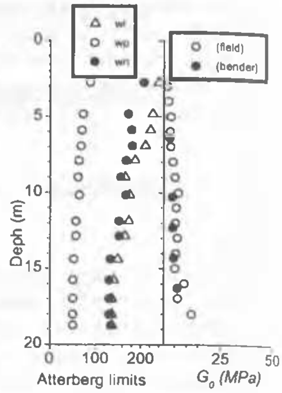

Figure 2. Site profile Hachirogata

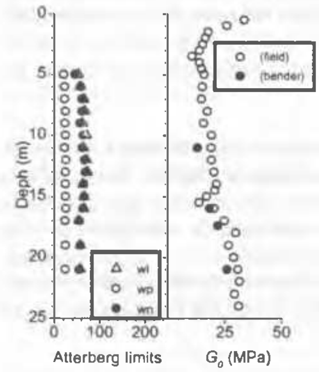

Figure 3. Site profile of Pusan

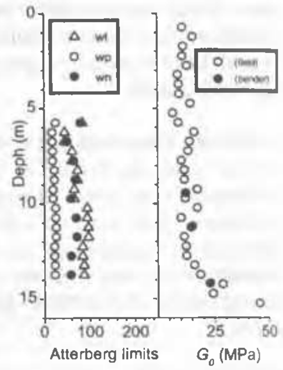

Figure 4. Site profile Bangkok

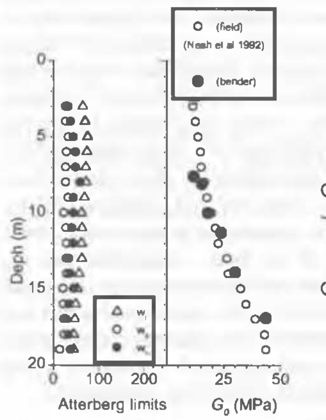

Figure 5. Site profile of Bothkennar

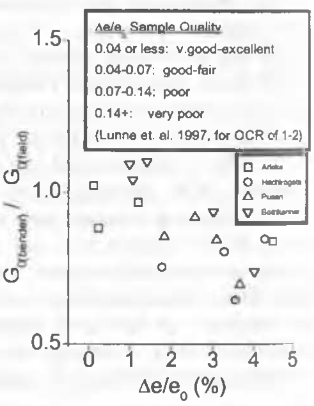

Figure 6. Influence of sample disturbance in G_0 measurement

Bangkok site (Figure 4) is located at Sutthisan of Bangkok, Thailand, and has been reported by Tanaka *et al.* (2001a). Like the sites of Ariake, grain composition its consists of very fine particles.

Bothkennar site (Figure 5) has been thoroughly investigated, and the results have been published in volume 42 of Geotechnique. Details of this site can be found from Nash *et al.*, 1992 & Tanaka (2000).

For details on Louiseville & Drammen deposits, readers may refer to Roy (1995) and Hamouche et al. (1995), and Larsson & Mulabdic (1991), respectively.

3.2 *G_0 profiles from insitu seismic cone tests*

Figures 1-5 show the profiles of insitu seismic test results for all the five sites. For the Ariake clay (Figure 1), G_0 increases with depth continuously. At a depth of 3 m, G_0 is less than 3 MPa, which increase to 20 MPa at the depth of 18 m, with mean value of about 8 MPa for the depth range of 3 m to 18 m.

Hachirogata site also shows general increase in G_0 with depth (Figure 2), where G_0 varies from as low as 1.6 MPa at the depth of 3 m, to about 12 MPa at the depth of 18 m, with the average value of 5.3 MPa for the same depth range. Hachirogata site and the Ariake site have relatively low value of G_0 compared to other three sites, which is due to the presence of significant proportion of diatom microfossils (Shiwakoti *et al.* 2001), which will be discussed in more detail later.

Figure 3 shows that G_0 does not increase with depth along several sections of the Pusan clay. The change in G_0 with depth is somewhat complicated, a sharp kink is observed at around the depth of 15.5 m. Compared to the sites of Ariake and Hachirogata, this site has larger value of G_0 at a given insitu stress. For the depth of 3 to 18 m, its G_0 varies from about 14 to 30 MPa, with the average value of 18 MPa.

The Bangkok clay site possesses complicated geological history owing to desiccation of crust and up and down of water table due to the over-pumping of underlying layers (Figure 4). Therefore, this site also has several fluctuations in G_0 profile with depth, average value of G_0 being approximately 15 MPa.

Field data of the Bothkennar site, taken from Nash *et al.* (1992), shows G_0 increases from about 11.5 MPa at the depth of 3 m to about 43 MPa at the depth of 18 m (Figure 5).

3.3 *Comparison of G_0 from insitu seismic cone tests and laboratory bender element tests*

Laboratory bender element test results for all the five investigation sites have also been plotted in respective site profiles of Figures 1-5. Comparison of data reveals a slight site/depth specific discrepancy between seismic cone and bender element tests for all the soil types. In general, bender element yields slightly lower values of G_0, compared to the seismic cone tests. However, for almost all the practical purposes, the results are comparable. Jamiolkowshi *et al.* (1994) have also done similar tests for several Italian clays and have found that the G_0 measured by seismic cone are only slightly higher than those resulting from bender element tests, when performed at effective stresses comparable to those existing in the field.

3.4 *Influence of sample disturbance in measured G_0*

The degree of disturbance of a sample tested in laboratory may be accessed by subjecting a specimen to its insitu stress state and observing the change in its void ratio or volumetric strain.

A plot of the ratio of G_0 from bender and seismic cone test ($G_{0(bender)}/G_{0(field)}$) versus the change in void ratio of a sample in recompressing the sample to its insitu stress state, is shown in Figure 6, where, e_0 represents the in-situ void ratio of a soil specimen, which is estimated based on its natural water content and the particle density of soil solids (specific gravity), while e is the void ratio of the soil specimen when recompressed to its natural stress state, assuming the coefficient of earth pressure at rest (K_0) of 0.5.

Figure 6 reveals that the sample disturbance and the discrepancy between the field and laboratory test results are inversely correlated. In general, with an increase in sample disturbance, as measured by a decrease in initial void ratio, $G_{0(bender)}/G_{0(field)}$ decreases.

Also included in the figure is the criteria for sample quality evaluation based on $\Delta e/e_0$ proposed by Lunne *et al.* (1997). It can be seen that almost all the samples tested have $\Delta e/e_0$ less than 4%, indicating that samples used were of very good to excellent quality.

4 CORRELATION OF G_0 WITH SOME BASIC SOIL PARAMETERS

Several researchers have tried to describe the characteristics of small strain shear modulus in terms of other fundamental soil parameters. All seem to agree that G_0 is a function of some power of insitu effective stress, may it be mean, vertical, or confining stress. However, there is a dispute on whether and how much G_0 depends on OCR, I_p and e. Viggiani and Atkinson (1995), for example, have shown that G_0 is related to I_p in addition to OCR and insitu stress, but have ignored the explicit influence of e on G_0. However, Jamiolkowski *et al.* (1994) have insisted that, if appropriate function of e is used, OCR does not have any influence on G_0 and have accordingly correlated G_0 with e, instead of with I_p or OCR. Shibuya & Tanaka (1996), and Shibuya *et al.* (1997, 1999) have also proposed prediction models similar to Jamiolkowski group. In the following sections, relations of G_0 with some of these parameters have been examined.

4.1 *Relation between G_0/σ_{v0}', OCR & Void ratio*

Figure 7 shows the plot between $G_{0(field)}/\sigma_{v0}'$ and e_0 for eleven different soils. Trend of increase in $G_{0(field)}/\sigma_{v0}'$ with a decrease in e_0 is apparent. However, the scatter in data is notable, which is partly so because the value of G_0 might also be affected by other factors such as OCR, soil fabric bonding, and so on.

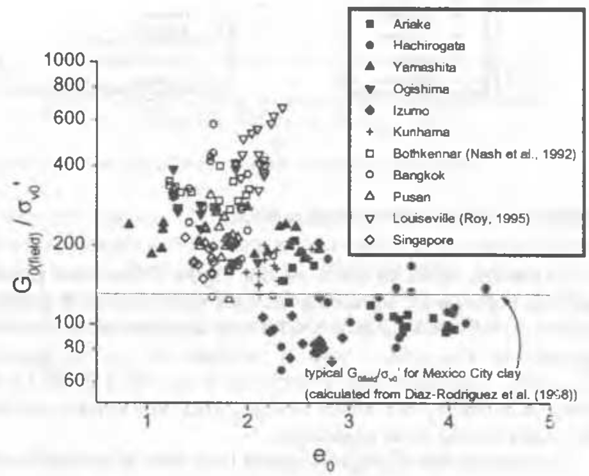

Figure 7. Relation between G_0 / σ_{v0}' and e_0

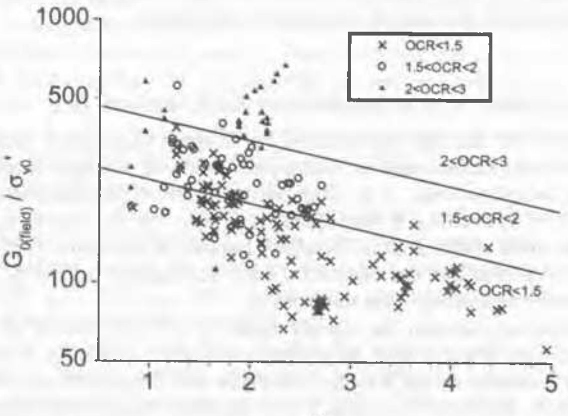

Figure 8. Relation between G_0 / σ_{v0}', e_0 and OCR

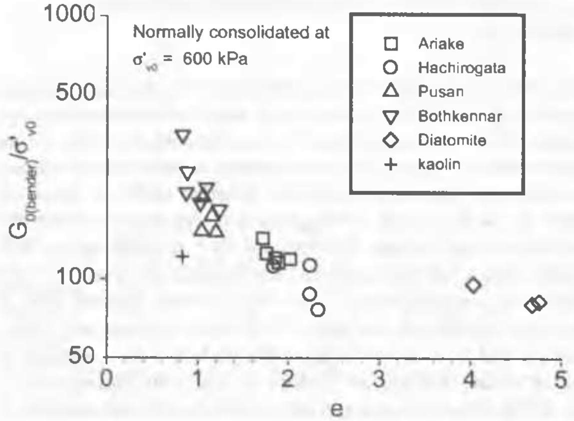

Figure 9. Relation between G_0/σ_{v0}' & e at 600 kPa consolidation pressure

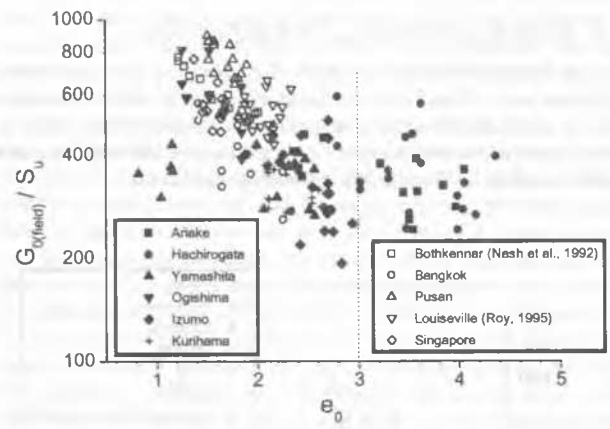

Figure 10(a). Relation between G_0/S_u and e_0

To see the effect of OCR on G_0, Figure 7 has been plotted again as Figure 8 by separating the data into three OCR groups: OCR<1.5, 1.5<OCR<2 & 2<OCR<3. In addition to the trend of increase in $G_{0(field)}/\sigma_{v0}$' with a decrease in e_0, in general, $G_{0(field)}/\sigma_{v0}$' increases with a decrease in e_0, for a given OCR. Thus, OCR and e_0 are related to $G_{0(field)}/\sigma_{v0}$' and certain correlation exists among these parameters.

To examine how G_0/σ_{v0}' is related to e, even at normally consolidated state, at which all the stress histories get erased and bonding structures get destroyed, bender element tests were carried out on various soil specimens by consolidating them isotropically to 600 kPa. Figure 9 shows the resulting relation between $G_{(bender)}/\sigma_v$' and e. The results from this figure confirm the existence of fairly good correlation between G_0/σ_v' and e even at consolidation pressure far beyond the yield stress.

4.2 *Relation between rigidity index (G_0/S_u) and e or I_p*

Despite the fact that undrained shear strength (S_u) of soil varies with many factors such as failure path followed, strength anisotropy and strain rate, S_u is often used as normalizing parameter. Merit of S_u over σ_{v0}' is that, unlike σ_{v0}', S_u is the function of σ_{v0}', yield stress and soil fabric bonding. Larsson & Mulabdic (1991) have reported that correlation between normalized G_0 and I_p or w_l shows less scatter than that with e_0.

Relation between the rigidity index, I_R (=G_0/S_u), and e or I_p for eleven different sites are plotted in Figure 10, where S_u is the vane strength, except for the Yamashita and Singapore soil, for which S_u has been taken as $q_u/2$ from uniaxial test and peak shear strength (τ_{max}) from constant volume direct shear test, respectively. In the figure, no correction factors have been applied to the measured vane strengths.

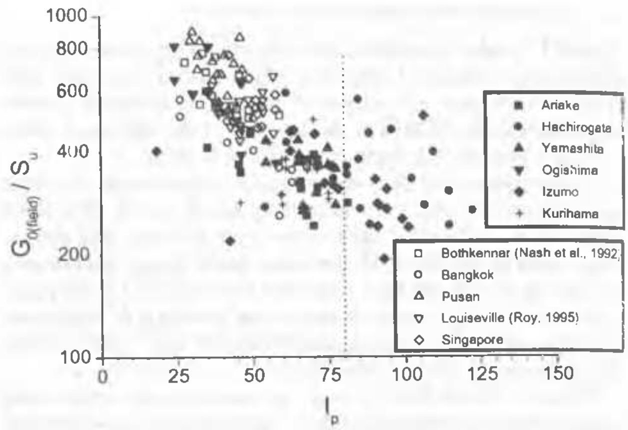

Figure 10(b). Relation between G_0/S_u and I_p

Figure 10, shows that, despite some scatter in data, I_R in log scale is inversely proportional to I_p (and e_0), for I_p less than 80 (and for e_0 less than 3). Comparing Figure 10 with Figure 7 or 8, it can be seen that the use of I_R considerably reduces the scatters in data seen with the use of G_0/σ_{v0}'. Therefore, the use of I_R is recommended over the use of G_0/σ_{v0}' while characterizing G_0 in non-dimensional form.

It may also be said that the scatters in data implies that no unique correlation can be derived from these parameters alone for the entire data range. Alternatively, this could imply that some G_0 dependant factors might still be missing.

4.3 *Relationship between G_0 and diatom microfossils*

It is well recognized that Japanese soils have unique characteristics such as large effective friction angle (ϕ') and large value of S_u/σ_v', despite having relatively large w_l and I_p (Tanaka, 2001). Similarly, relatively low clay size fractions, relatively low particle density, relatively large coefficient of compression as well as relatively large coefficient of permeability are also the distinct characteristics of Japanese soils. Recently, it has come into light that Japanese soil deposits contain significant proportion of diatom microfossils (see for example, Tanaka and Locat, 1999). Further investigation has confirmed that the presence of diatom microfossils imparts significant influences on consistency indices as well as mechanical properties such as C_c, ϕ', S_u/σ_v' of a soil (for details see Shiwakoti *et al.*, 1999, 2001). Therefore, it is important to take into consideration the possible influence of diatom microfossils, while examining the G_0 characteristics of Japanese soils.

Careful examinations of Figures 7 and 10 reveal that Japanese soils have three distinct characteristics compared to the non-Japanese soils. Firstly, Japanese soils have relatively low values of G_0, and secondly the correlation between G_0 and e_0 (or I_p) is not as good as that for other soils. Furthermore, Japanese soils have relatively large e_0 and I_p compared to that of other soils (Table 1). The data from Ogishima shows exception to this trend, which is mainly due to its high content of silts and sands.

Similarly, Figure 9 shows relatively low values of G_0 for Japanese soils even at normally consolidated state. Also included in the figure is the data for diatomite and kaolin. It is clear from this figure that diatomite has the lowest value of G_0 as well as the highest value of e_0. Therefore, a soil rich in diatom microfossils (*i.e.* diatomaceous soils) can be expected to have relatively low G_0 as well as high e_0 (and also high I_p). Recent quantitative study has established that Hachirougata and Ariake soils are very rich in diatom microfossils (Shiwakoti *et al.*, 2001). Similarly, most of the other Japanese soils investigated in this research also are either confirmed or suspected to have significant quantity of diatom microfossils.

The finding of this study that diatomaceous soils have low G_0 is further supported by similar characteristics of Mexico City clay, which reportedly contains more than 50% of diatom micro-

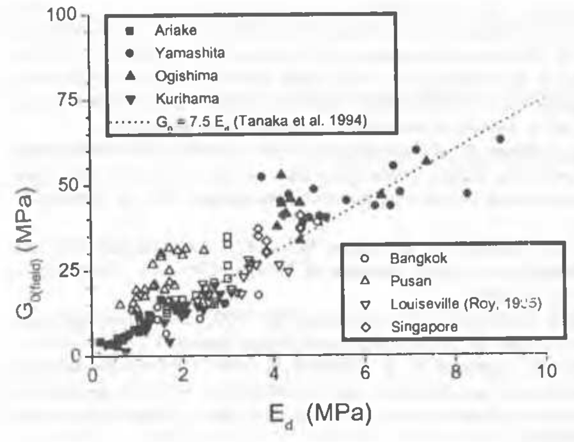

Figure 11. Relation between G_0 and E_D

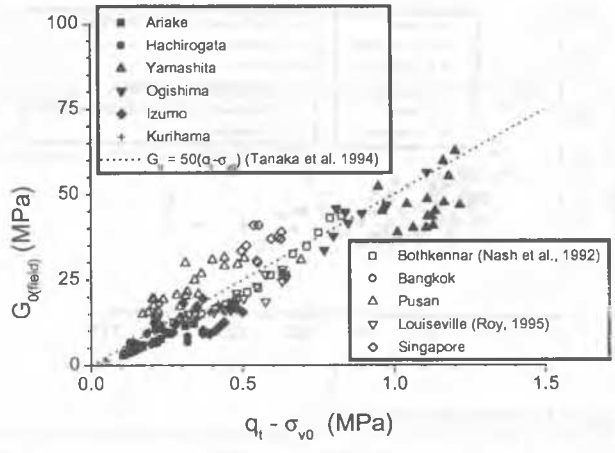

Figure 12. Relation between G_0 and $q_t - \sigma_{v0}$

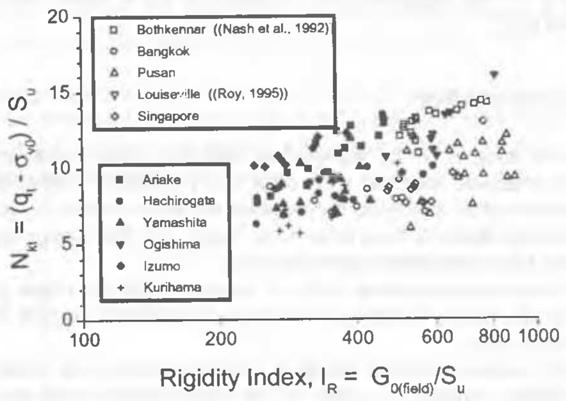

Figure 13. Relation between cone factor and rigidity index

fossils (Mesri et al., 1975). This clay has average shear wave velocity of 81 m/s, resulting in very low G_0 of about 7.5 MPa at a depth of 30 m, as also indicated in Figure 7 (Diaz-Rodriguez *et al.*, 1998).

Therefore, relatively low values of G_0, and high values of e_0 as well as I_p of Japanese soils can be well explained by considering the existence of diatom microfossils in these soils.

5 CORRELATION BETWEEN G_0 AND RELEVANT SOIL PARAMETERS FROM OTHER INSITU TESTS

If some simple correlations of relevant soil parameters obtained from different tests can be established, such relations could be useful in providing preliminary estimation and crosscheck of soil parameters obtained from other tests. In this section, correlation of G_0 obtained from seismic cone test will be examined with relevant soil parameters obtained from field vane test, Marchetti's dilatometer test and piezocone (CPTU) test.

5.1 *Relation between G_0 and E_D*

Among the correlations frequently sought with G_0 is the dilatometer modulus (E_D) obtained from Marchetti's dilatometer test. Here, one must keep in mind that not only the working mechanisms of these two instruments are totally different, but also the directions of measurement of modulus by these instruments are often perpendicular with each other. For example, E_D is invariably measured in horizontal direction (in vertical plane), while, G_0 is most often measured in vertical direction (down-whole tests). Furthermore, shear strain induced while measuring E_D by dilatometer could be significantly higher than that while measuring G_0 using seismic cone. Despite these facts, empirical correlations between these two parameters can be useful in practice.

Figure 11 shows the relation between G_0 and E_D for nine different soil deposits. Fairly good correlation between G_0 and E_D can be observed. Also plotted in the figure is the correlation (2) proposed by Tanaka *et al.* (1994), which shows reasonably good agreement with the plotted data. Thus:

$$G_0 = 7.5\, E_D \tag{2}$$

5.2 *Relation between G_0 and $q_t - \sigma_{v0}$*

One important correlation commonly sought between the seismic cone and CPTU test is between G_0 and $q_t - \sigma_{v0}$, where q_t is the cone tip resistance corrected for pore pressure and σ_{v0} is the total insitu vertical stress. Some researchers have tried to correlate G_0 and $q_t - \sigma_{v0}$ with I_p, while others have tried to establish simple relation between the former two parameters (Tanaka *et al.* 1994).

Figure 12 shows G_0 plotted against $q_t - \sigma_{v0}$ for eleven different soils. Despite some scatter in data, fairly good correlation between the two parameters can be observed. Also plotted in the figure is the empirical relation proposed by Tanaka *et al.* (1994), which shows quite good agreement with the data. Thus:

$$G_0 = 50\, (q_t - \sigma_{v0}) \tag{3}$$

5.3 *Relation between $(q_t - \sigma_{v0})/S_u$ and G_0/S_u* (or I_p)

Piezocone test with pore pressure measurement (CPTU) is increasingly being used in practical works for site characterizations. However, major problem with the application of this test is the difficulty in accurate determination of cone factor, N_{kt} (= $(q_t - \sigma_{v0})/S_u$), which may vary in the range of 5-30. Researchers have tried to correlate N_{kt} with I_p, OCR, or even pore pressure ratio.

A frequently used relation, similar to equation 3 above, is the correlation between N_{kt} and I_R. Due to the fact that S_u of a soil is not unique parameter but depends on various factors such as failure path, strength anisotropy, strain rate, etc., it is possible that q_t is more fundamentally related to G_0 than that with S_u. In fact, Teh (1987) has theoretically shown that N_{kt} is a function of I_R, cone roughness and insitu stress.

Figure 13 shows the relation between N_{kt} and I_R for eleven different soil deposits. S_u from vane tests have been used throughout, except for the Yamashita and Singapore soils, for which $q_u/2$, and τ_{max} have been used, respectively. S_u has not been subjected to any correction factor.

The result reveals that, despite the scatter in data, N_{kt} increases linearly with the increase in logarithm of I_R, confirming the theoretical prediction of Teh (1987). Wider scatter in data from Pusan in believed to be due to the underestimation of its S_u.

Figure 14 shows the relation between N_{kt} and I_p for twelve different soil deposits. The data shows that, despite some appar-

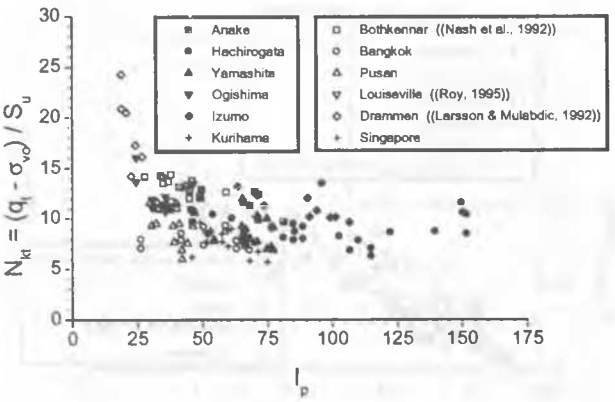

Figure 14. Relation between N_{kt} and I_p

ent relation between N_{kt} and I_p, the correlation is no good. Particularly for the Japanese soils, variation of N_{kt} is totally independent of I_p.

6 CONCLUSIONS

Characteristics of G_0 for undisturbed soils have been examined and its empirical relations with other soil parameters measured by various tests have been investigated on twelve naturally deposited soils deposits from around the world. The following conclusions have been drawn from this study.

1. Relatively low values of G_0 of Japanese soils are found to be due to the influence of diatom microfossils present in these soils.
2. For routine practical purposes, bender element test yields results comparable to that of the insitu seismic cone test, provided that good quality samples are used.
3. Empirical relations: $G_0 = 50\ (q_t - \sigma_{v0})$, and $G_0 = 7.5\ E_d$ are found to be valid for wide varieties of soft soils.
4. N_{kt} increases linearly with an increase in logarithm of G_0/S_u. However, N_{kt} is found to be independent of I_p.
5. G_0/S_u of soft soils in logarithmic scale decreases linearly with an increase in I_p (and e_0), for $I_p < 80$ (and $e_0 < 3$).
6. Although correlation exists between G_0/σ_{v0} and e_0, it is difficult to satisfy field data by using these parameters alone.

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