ASSESSING STRUCTURE OF AGED NATURAL SEDIMENTARY CLAYS

SATORU SHIBUYA¹⁾

ABSTRACT

Soils become stiffer as they age. In this paper, attempts were made to assess the in-situ structure of natural clays by calculating the current quasi-elastic shear modulus from shear wave velocity measurement, $G_{\max}$. The $G_{\max}$ value of reconstituted clays was employed as a yardstick against which the corresponding characteristic of natural sedimentary clays is examined, implying that the intrinsic properties of a non-aged reconstituted clay provide a basic frame of reference for assessing the in-situ structure of a natural clay and the influence of ageing (secondary compression and inter-particle bondings) on its in-situ properties. In a series of bender element (BE) tests on a reconstituted clay, it was first manifested that the relationship between the void ratio and $G_{\max}$ as subjected to one-dimensional compression exhibited the state boundary (SB) curve applicable to the short-term migration of effective stress. Second, it was demonstrated that the SB curve was violated when the clay aged by drained creep, and that the state of the aged clay rejoined the SB curve in a gradual manner on subsequent stressing applied over a short period. In the light of this characteristic time-dependent behaviour of $G_{\max}$, a measure termed metastability index $MI(G_{\max})$ has been newly proposed for assessing the structuration/destructuration of in-situ natural sedimentary clays which undergo effects of a long-term in-situ ageing. The performance of $MI(G_{\max})$, together with the non-dimensional soil constant reflecting structure, S (Jamiolkowski et al., 1994), was carefully examined in the BE tests using various natural and reconstituted clays. It was successfully demonstrated that: i) the proposed index $MI(G_{\max})$ as well as the S -value were both equally capable of expressing structuration/destructuration of clays as examined in tests on Ariake (Japan), Bangkok (Thailand) and Louiseville (Canada) clays, each comprising a pair of a natural and its reconstituted sample; ii) two different formations of soil structure, i.e., an on-depositional structure formed at sedimentation and the evolving post-depositional structure enriched due to ageing, should both be addressed for a proper understanding of the current structure of natural clays; iii) the S values were similar among a total of nine non-aged reconstituted clays, despite a wide spread of plasticity index as well as compressibility; and iv) the S -value of each reconstituted clay is the lower bound against which the in-situ structure of the natural clay can be quantified at the relevant effective stress. A time-dependent soil structuration was also predicted by introducing a soil constant for metastabilization, C_{β} .

Key words: ageing, bonds, clay, density, elastic modulus, laboratory test, liquidity index, structure (IGC: D5/D6/D7/C2/C6/B1)

INTRODUCTION

The term 'soil structure' is ambiguous. Among practicing engineers as well as researchers, this term is often referred to, for instance when describing different mechanical behaviours observed between natural and reconstituted samples of a clay, the erratic behaviour of comparable natural clay samples caused by some artificial disturbances, and so on. In most such situations, we rather abuse this term by simply having a vague image of micro-structure in mind. The particles of a natural clay are arranged in edge-to-edge and edge-to-face associations, whereas the arrangement of a reconstituted clay corresponds to the state close to the densest possible packing of particles at the relevant effective stress (Mitchell, 1993). However, the designation of soil microstructure to an explanatory mechanism is seemingly qualitative, im-

plying that this kind of examination is unlikely to provide much help in predicting different mechanical behaviour reflecting numerous micro-structures and fabrics of a cited clay. Engineers as well as researchers working in the field of geotechnical and geological engineering must now have a common measure in describing the soil structure. As a result of this, a better engineering judgement may be made, for example in evaluating the quality of soil samples in the laboratory (e.g., Toki et al., 1994).

This paper attempts to assess the in-situ structure of natural sedimentary clays by using a clay property that can be readily measured in-situ and/or in the laboratory. In dealing with natural sedimentary clays in particular, we should pay attention to the development of soil structure due to the continuous action of ageing over the entire geological formation (Hight and Higgins, 1994). In this respect, the time-dependent response of clays due to

¹⁾ Associate Professor, Graduate School of Engineering, Hokkaido University, Sapporo 060-8626.

Manuscript was received for review on March 18, 1999.

Written discussions on this paper should be submitted before January 1, 2001 to the Japanese Geotechnical Society, Sugayama Bldg. 4F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101-0063, Japan. Upon request the closing date may be extended one month.

ageing is highlighted by substantial increases of both stress and strain at apparent yielding(s), σ'_y and ϵ_y , involved with the creep-strain and/or bonding induced expansion of the yield surface(s) (Jardine, 1992; Leroueil and Vaughan, 1990; Tatsuoka et al., 1997; Asaoka et al., 2000). Similarly, the quasi-elastic shear modulus from shear wave velocity measurement, $G_{\max}$, of clays exhibits continuous increase with time during drained creep (Afifi and Richart, 1973; Anderson and Woods, 1975; Kokusho, 1987; Anderson and Stokoe, 1987). This time-dependent behaviour of $G_{\max}$ cannot be accounted for by increased density alone. Accordingly, the use of quasi-elastic soil properties such as σ'_y or $G_{\max}$ appears to be intrinsically suitable for portraying the degree of structuration of in-situ natural clays, since these properties are closely linked with soil ageing involved with concurrent development of soil structure.

The term 'quasi-elastic' is employed in this paper by considering the rate-dependent characteristic of each property, for which the σ'_y value is significantly rate-dependent (Leroueil and Marques, 1996), whereas $G_{\max}$ associated with the shear strain level of about 0.0001% is more or less exempt from rate effects, but involved with a very small (i.e., not exactly zero) hysteretic damping ratio (Tatsuoka and Shibuya, 1992; Shibuya et al., 1995a). It should be mentioned that experimental data are now abundant for $G_{\max}$ from shear wave velocity to coincide with the corresponding stiffness in stress-strain relationship at the relevant small-strains as observed in monotonic as well as cyclic loading tests in the laboratory (e.g., see Shibuya et al., 1992).

CLAY SAMPLES AND LABORATORY TEST

Natural Clay Samples

Three soft marine clays, Ariake from Japan, Bangkok from the Kingdom of Thailand and Louiseville (or Champlain) from Canada, were provided for laboratory testing. Both the Ariake and Louiseville clays were retrieved by using a large-diameter ($\Phi=208$ mm) Laval sampler (La Rochell et al., 1981), whereas a fixed-piston Japanese thin-wall sampler ($\Phi=76$ mm) was employed for the soil sampling in Bangkok. All three clay samples can be categorized into normally consolidated aged clay by indicating no evidence of geological overconsolidation at each sampling site (Bjerrum, 1973).

For Ariake clay, the results of in-situ tests as well as the soil samplings performed at the exact site have been described in detail by Tanaka et al. (1996) and Shibuya and Tanaka (1996). The values of water content at Atterberg limits, w_L and w_P , are each relatively large compared to other marine clays in Asian countries. The in-situ water content w_n exceeds w_L , implying a high sensitivity ratio (refer also to Ohtsubo et al., 1995). The undrained shear strength c_u from field vane increased more or less linearly with depth, showing a value of about 20 kPa at 10 m depth. The overconsolidation ratio (OCR) as examined in a constant-rate-of strain (CRS) consolidation test using the rate of 0.01%/min exhibited values ranging be-

tween 1.2 and 1.4 due to ageing.

The Bangkok clay sample was obtained at Nong Ngu Hao (NNH) site, which has long been planned for the second international airport in Bangkok. Soil properties at the exact site have been thoroughly examined by a research group at the Asian Institute of Technology (AIT) (Balasubramaniam et al., 1996; Shibuya et al., 1998). At this site, so-called soft Bangkok clay extends down to 15.5 m depth. In this soft marine clay layer, w_n reaches values in excess of 100% being close to w_L . The c_u value from field vane tests increased with depth from 13 kPa at 4 m depth to 27 kPa at the bottom. The OCR value ranged from 1.0 to 1.3 as examined in the CRS consolidation test.

The OCR value at the Louiseville site exhibited gradual decrease with depth by showing that the subtraction of effective overburden stress $\sigma'_{v(in-situ)}$ from σ'_{vy} at each depth remains at a roughly constant value (Leroueil et al., 1991). This pattern of the OCR profile with depth strongly suggests apparent overconsolidation due to the development of interparticle bondings, which has recently been confirmed in terms of the c_u profile with depth (Tsuji et al., 1998).

Reconstituted Clay Samples

Reconstituted samples of Ariake, Bangkok and Louiseville clays were each prepared from the above-mentioned natural clay sample, so that each pair shared intrinsic properties such as Atterberg limits (w_L and w_P), density of soil particles, ρ_s , and mineralogical contents in common. In addition, a total of nine reconstituted clay samples were prepared, for which the basic properties, except for Bangkok clay were given in Table 1 of Shibuya et al. (1997). In preparing these reconstituted clay samples, clay flakes at trimming were thoroughly mixed with deaired water in a partially vacuumed chamber to give the slurry an initial water content of about twice w_L . Prior to preconsolidation in the perspex chamber, any shells or coarse grained soils in the slurry were removed by using a 420 μ m sieve. The slurry was gently poured into the chamber, implying that the liquidity index I_L was approximately two at deposition for all the reconstituted clay samples. The slurry of each clay was subjected to one-dimensional preconsolidation by using a preconsolidation pressure which ranged from 70 to 150 kPa depending on the clay tested.

Laboratory Bender Element (BE) Test

A laboratory technique using bender elements was employed for measuring the shear body wave velocity, V_s , propagating through soil specimens (Dyvik and Madhus, 1985). The $G_{\max}$ value is given by

$$G_{\max} = \rho_t V_s^2 \quad (1)$$

where ρ_t denotes bulk density of soil. Most of the tests described in this paper were performed in a consolidometer equipped with a set of bender elements (refer to Fig. 1 of Shibuya et al., 1997). This apparatus (developed at Hokkaido University), together with the experimental

techniques are described in detail in Shibuya et al. (1997). The disk-shaped specimens had dimensions of 60 mm in diameter and 20 mm in height. The vertical stress on the specimen, σ'_v , was measured by means of a load cell mounted on the base, the measurement of which was thus unaffected by any mechanical friction in the axial loading system. The horizontal stress was also measured by monitoring hoop strain in the thin section of the consolidation ring. The travel distance of the shear wave was taken as the tip-to-tip distance between the bender elements, each protruded into the clay sample by 4 mm. The arrival time was determined at the first clear peak as measured by the receiver (Viggiani and Atkinson, 1995a). A similar consolidometer, but without the horizontal stress measurement was employed in tests on Louiseville and Bangkok samples (Lohani et al., 1999).

In tests on three natural clay samples, σ'_v was increased in steps to $\sigma'_{v(in-situ)}$. This process is called in this paper 'recompression'. The specimen was subsequently subjected to unload-reload cycle(s), for which the first loading stage exceeding $\sigma'_{v(in-situ)}$ is termed 'virgin compression'. It should be mentioned that drainage was allowed at the top surface of the specimen only. On reaching each prescribed σ'_v value, the stresses, together with V_s were measured at an instant when the excess pore water pressure at the bottom of the specimen was fully dissipated. This rather unusual technique was employed in an attempt to minimize generation of creep-strain throughout testing, for which reason the term 'compression' rather than 'consolidation' was intentionally used in this paper.

OBSERVATIONS INTO THE BEHAVIOUR OF G_{max}

State Boundary (SB) Curve of Non-aged Reconstituted Clay

Figure 1 shows the results of a series of BE tests performed on a reconstituted clay. This NSF-clay is a powder clay on retail, which features high repeatability in producing samples having a similar density at a given effective stress (for details, see Shibuya et al., 1995a). Four specimens were all prepared from a single preconsolidated block sample using the preconsolidation pressure of 150 kPa. Three specimens were each subjected to one-dimensional compression to the prescribed σ'_v of 150, 200 and 300 kPa, respectively, and afterwards σ'_v was reduced in steps by following the technique described earlier. Note that the $e - \log G_{max}$ relationship in a state of normal compression (NC) was successfully expressed using a straight line, the line $\beta^* - \beta^*$ in Fig. 1(b). On over-compression (OC), the state moved to the left of this line by showing also a linear $e - \log G_{max}$ relationship in each sample.

According to the framework of critical state soil mechanics, the NC line in the corresponding $e - \log \sigma'_v$ relationship (i.e., the line $\alpha^* - \alpha^*$ in Fig. 1(a)) represents a boundary between possible states to the left and impossible states to the right, bearing in mind that the NC line is the state boundary (SB) curve. In a similar fashion, the results shown in Fig. 1(b) strongly suggest that the NC

Fig. 1. Behaviour of four reconstituted NSF clay specimens subjected to one-dimensional compression: a) e versus σ'_v , and b) e versus G_{max}

line (i.e., the line $\beta^* - \beta^*$) is the SB curve applicable to the relationship between e and G_{max} .

Violation of SB Curve due to Short-term Ageing in the Laboratory

The fourth specimen was aged for two weeks while σ'_v was maintained at 200 kPa. Afterwards, it was subjected to incremental loading by using the compression technique as described earlier. At the end of primary (EOP) compression denoted by point A*, the state of (e , σ'_v , G_{max}) was in the close proximity to the NC lines. As the specimen undertook sustained drained creep, the state in the (e , G_{max}) space moved away to the right of the SB line (i.e., the line $\beta^* - \beta^*$ in Fig. 1(b)), ending up at point B. The SB curve was certainly violated as the clay aged. This means that during the sustained drained creep, the rate of increase in G_{max} with increased density was higher compared to the corresponding rate when subjected to stress-

Fig. 2. Variations of e and $G_{\max}$ with σ'_v of aged NSF clay sample subjected to normal compression after drained creep for two weeks

ing without significant ageing. Contrary to the behaviour of $G_{\max}$, the state of (e, σ'_v) migrated well below the corresponding SB line of $\alpha^*-\alpha^*$ (see Fig. 1(a)).

The behaviour after the drained creep is examined in detail in Fig. 2. Immediately after the stressing as started at point B, a tendency was clear for $G_{\max}$ to decrease in value, whereas the density changed little. A structural breakdown, i.e. destructuration, was definitely observed at this early stage of NC. On subsequent loading, $G_{\max}$ increased monotonically accompanied by monotonous densification with σ'_v . Note that the state in $(e, G_{\max})$ space steadily came back to the SB line of $\beta^*-\beta^*$, eventually ending up completely rejoined with it. The entire behaviour on stressing after the drained creep can therefore be interpreted as sustained destructuration. Conversely, it was somewhat ambiguous whether or not the SB curve in the (e, σ'_v) space (i.e., the line $\alpha^*-\alpha^*$ in Fig. 1(a)) was violated in the event of this quick stressing. Since the NC curve including the apparent yield stress, σ'_{vy} , is affected by the rate of loading, it is difficult to draw a definite conclusion on the sustainability of the SB curve as examined with the short-term ageing.

It should be mentioned that similar behaviour to the above has been observed in the CRS consolidation test (Leroueil, 1995), and some reduction of $G_{\max}$ on stressing has been observed in an aged reconstituted clay at the very start of primary consolidation (Nakagawa et al., 1995).

Comparison between Natural and Reconstituted Samples

A comparison by using a pair of Ariake clay samples is shown in Fig. 3. On recompressing the natural clay sample to $\sigma'_{v(in-situ)}$ of 51 kPa (point B), the $G_{\max}$ value steadily increased against σ'_v concurrent with a continuous decrease in void ratio. The $G_{\max}$ value practically coincided with G_f from in-situ seismic survey at $\sigma'_{v(in-situ)}$, implying the quality of this natural clay sample (see Fig.

Fig. 3. Comparison of behaviour of natural and reconstituted samples of Ariake clay at 15.8 m depth: a) e versus σ'_v , b) e versus $G_{\max}$, and c) $G_{\max}$ versus σ'_v

Fig. 4. Comparison of behaviour of natural and reconstituted samples of Bangkok clay at 12.5 m depth in Nong Ngu Hao (NNH): a) e versus σ'_v , b) e versus G_{max} , and c) G_{max} versus σ'_v

Fig. 5. Comparison of behaviour of natural and reconstituted samples of Louisville clay at 11.5 m depth: a) e versus σ'_v , b) e versus G_{max} , and c) G_{max} versus σ'_v

Fig. 6. Scanning electron microscope pictures of natural and reconstituted samples of Ariake, Bangkok (NNH), and Louisville clays: a) Ariake (natural), b) Ariake (reconstituted), c) Bangkok (natural), d) Bangkok (reconstituted), e) Louisville (natural), and f) Louisville (reconstituted)

3(c)). The virgin compression exceeding $\sigma'_{v(in-situ)}$ showed σ'_{vy} at around 62 kPa (point C), resulting in an OCR value of 1.2. Beyond this apparent yielding, a tendency was clear for the state (e , σ'_v , G_{max}) of the natural clay sample to approach the relevant SB curves of the reconstituted sample (i.e., $\alpha^*-\alpha_{nc}^*$ in Fig. 3(a) and $\beta^*-\beta_{nc}^*$ in Fig. 3(b)). As described later, this behaviour is interpreted as continuous destructuration on virgin compression. Also interesting is the fact that some restructuration was observed in the event of two-day drained creep at $\sigma'_v=500$ kPa. Although the change in void ratio was small (see Fig. 3(a)), the G_{max} value exhibited a substantially large increase with time over this very short period of re-ageing in the laboratory.

The behaviour of Bangkok clay samples was remarkably similar in a qualitative sense (Fig. 4). However, the difference in the behaviour between the natural and reconstituted samples was less significant as compared to Ariake clay. An example is seen in the e -log G_{max} relationship, for which the rate of increase in G_{max} with decreased void ratio was slightly lower for the natural sample (see Fig. 4(b)).

The behaviour of Louisville clay samples was again similar to, but slightly different from the response of the other two clays (Fig. 5). Unlike the behaviour of natural samples of Ariake and Bangkok clays, the G_{max} value on recompression to $\sigma'_{v(in-situ)}$ of 66 kPa remained more or less constant at values close to G_f , and with a small amount of volume change. The coincidence between G_{max} and G_f at $\sigma'_{v(in-situ)}$ as well as this remarkably high compressibility at the stage of recompression would ensure the quality of the natural clay sample tested. The different behaviour on recompression compared to the other two clays is presumably attributed to the site-specific formation of the aged structure; i.e., the dominance of secondary compression for both Ariake and Bangkok clays and of interparticle bondings for Louisville clay, respectively. The virgin compression exceeding $\sigma'_{v(in-situ)}$ manifested a σ'_{vy} value of about 170 kPa (point C). Destructuration dominated the subsequent behaviour, for which the state (e , σ'_v , G_{max}) of the natural sample eventually rejoined that of the reconstituted sample at a σ'_v of about 300 kPa (point E). Surprisingly, the G_{max} value after unloading at point U coincided with G_{max} on the recompression at $\sigma'_v=20$ kPa (point R), despite marked differences in both void ratio and OCR value. The stress-independent behaviour of G_{max} on recompression suggests that the interparticle bondings had existed at the stage of recompression. Contrary to this, a breakdown of the interparticle bondings is certainly hinted at the behaviour on unloading that both (e and G_{max}) were conversely dependent on σ'_v after the joint of the state (e , σ'_v , G_{max}) to the fully-destructured sample.

An examination into microstructures was also carried out by using a scanning electron microscope (SEM) (Fig. 6). The clay particles of Ariake clay are definitely larger in size and more elongated in shape compared to Louisville clay. Microfossils (or diatoms) were abundantly found in Ariake, whereas Bangkok clay com-

prised little. It may be speculated that these primary properties of Ariake clay allow a wider variation of void ratio seen at deposition as compared to the other two clays. Note also that the reconstituted sample was more densely packed compared to the natural sample in Louisville clay, whereas this kind of densification was not obvious by sight for Bangkok clay.

DISCUSSIONS ON IN-SITU STRUCTURE OF NATURAL SEDIMENTARY CLAYS

On-depositional and Post-depositional Effects on Current Soil Structure

The author interprets the current structure of natural sedimentary clays by referring to two different formations. One is a characteristic soil structure formed at deposition, whereas the other is an evolving structure enriched by post-depositional ageing involving secondary compression and/or the growth of interparticle bondings. These are now termed by the author 'on-depositional' and 'post-depositional' effects on the current soil structure, respectively.

An on-depositional structure is seemingly characterized by the amount of contained water in voids at deposition, which is presumably governed by depositional circumstances such as temperature, salt content, rate of deposition, etc. For the sake of the following discussions, it is of great importance to realize that the relationship between w and σ' as subjected to one-dimensional compression is not unique even for a single cited clay, but it varies over a low stress range depending on the initial water content at deposition (Leroueil and Vaughan, 1990; Imai, 1995; Tsuchida and Gympo, 1995). In addition, 'aged-structure' must comply with post-depositional effects, so this terminology is exclusively applicable to clays that have been subjected to ageing.

Relationship among e , σ' and G_{max} of Non-aged Clay

Mikasa (1964) stated that the effective stress of unbonded saturated clays is determined by the soil type, w and the aged-structure. This premise may also be applicable to the mechanical properties, for example G_{max} . Thus, for a given soil;

$$\begin{aligned}\sigma' &= F_1(w, \text{aged-structure})_{\text{on-depositional structure}}, \text{ and} \\ G_{max} &= F_2(w, \text{aged-structure})_{\text{on-depositional structure}}\end{aligned}\quad (2)$$

Since the effective stress as well as G_{max} of a non-aged clay prepared at a specific w on deposition is each a monotonous and smooth function of w , the current water content of the non-aged clay is expressed as;

$$\begin{aligned}w &= \Phi_1(\sigma')_{\text{on-depositional structure}}, \text{ and} \\ w &= \Phi_2(G_{max})_{\text{on-depositional structure}}\end{aligned}\quad (3)$$

The G_{max} value of a non-aged clay subjected to NC is uniquely determined by w , which is in turn uniquely related to σ' . The behaviour expressed by Eq. (3) has already been successfully demonstrated in tests on a reconstituted clay with w at deposition of twice w_L (see Fig. 1).

As illustrated in Fig. 7, the NC curves serve as the SB curves of a non-aged clay, which may be approximated by

$$e = \Gamma - \lambda \ln [\sigma'_v / \sigma'_{v(\text{reference})}] \quad (\lambda > 0) \quad (4)$$

$$e = \Delta - v \ln [G_{\max} / G_{\max(\text{reference})}] \quad (v > 0) \quad (5)$$

in which Γ and Δ stand for void ratio at references of σ'_v and $G_{\max}$ at unity, respectively. Similarly, the behaviour on OC is expressed by the linear relationship with the slopes of $-\kappa$ ($\kappa > 0$) and $-\mu$ ($\mu > 0$) for the relations of $e - \ln \sigma'_v$ and $e - \ln G_{\max}$, respectively (see also Fig. 1).

Relationship among I_L (or e), σ'_v and $G_{\max}$ of Aged Natural Clays

The combined expressions of Eqs. (2) and (3) are a very basic concept adopted in this paper for interpreting the in-situ structure of aged natural sedimentary clays. When based on these grounds, underlying information necessary for assessing the in-situ structure of natural clays is exclusively obtainable by comparing the relationship among w , σ'_v , and $G_{\max}$ of a natural clay sample to the corresponding behaviour of its reconstituted (or fully-destructured) sample prepared in the laboratory.

Such a soil-element based comparison is confronted with two obstacles. The first obstacle refers to the technical problem in the laboratory of simulating in-situ clay sedimentation in terms of the water content at deposition. By using I_L , that is

$$I_L = (w_n - w_p) / (w_L - w_p) \quad (6)$$

it is widely accepted that the deposition with I_L equal to about four corresponds closely to the instant of actual marine clay sedimentation in the cited source ground (e.g., Tsuchida and Gymyo, 1995), whereas the prevailing practice worldwide of preparing the reconstituted

clay sample in the laboratory refers to the corresponding condition with I_L of about two (Burland, 1990). Bearing in mind the fact that the NC curve of non-aged clay is influenced by the I_L value at deposition, this widespread laboratory practice prevents us from singling out the aged structure of natural clay samples in the laboratory.

The second obstacle is attributed to the heterogeneous feature common to any natural clay deposit that the intrinsic properties such as w_L and w_p , ρ_s , and the mineralogical contents show subtle variations with depth. A couple of solutions capable of overcoming this hiccup are: i) to prepare a pair of comparative natural and reconstituted samples at a single depth by using a large-diameter quality sample (e.g., Oka et al., 1996); and ii) to make use of I_L , instead of w_n , by which interpretation of one-site test results using several samples from different depths is properly made on the same cooking table. The use of I_L , like relative density for sands, may also be helpful when comparing the degree of soil structure of various clays from different origins.

Figure 8 illustrates formations of the in-situ structure of a natural clay as examined in terms of the interrelationship among I_L , σ'_v and $G_{\max}$, the idea of which is based on the results of the short-term ageing test (refer to Figs. 1, 2 and 7). Both α - α and α^* - α^* , refer to the non-aged NC curves of two reconstituted clay samples prepared in the laboratory with $I_L=4$ and $I_L=2$ at deposition, respectively (see Fig. 8(a)). These initial states correspond roughly to the loosest and densest possible packing at deposition, bearing in mind the fact that the state denoted by a square refers to the conceptually densest possible packing at deposition having an I_L value of unity (Mikasa, 1964). The natural clay may have been subjected to the path from A to B during in-situ ageing. When the source ground is subjected to 'fast' loading with the rate of stressing equivalent to the α - α curve, the state of

Fig. 7. Migration of state (e , σ'_v , $G_{\max}$) in a reconstituted clay subjected to sustained drained creep and subsequent stressing

Fig. 8. In-situ structuration and destructuration in aged normally consolidated sedimentary clay

I_L , σ'_v is likely to overshoot and afterwards rejoin the α - α line by following the path B-C-D-E-F. The traverse of the SB curve in-between the start of loading (point B) and the apparent yielding (point C) may take place in the aged clay. The OCR value, the ratio of σ'_{vy} to the maximum past stress, σ'_{vmax} ($=\sigma'_{v(in-situ)}$) shows an apparent increase in excess of unity. On stressing exceeding σ'_{vy} , destructuration surpasses the subsequent behaviour by forcing the state of the clay to rejoin the α - α curve at point E, and it eventually joins the fully destructured state denoted by point F.

The curves, β - β and β^* - β^* in Fig. 8(b) represent the cor-

responding behaviour of G_{max} of the non-aged clay samples subjected to NC. The ageing involves continuous increase in G_{max} in an accelerated manner against decreased I_L . The stiffness state moves away from the comparable SB curve β - β ending up at point B, indicating the development of an aged-structure. When the aged clay is subjected to 'fast' loading, it responds elastically by showing an insignificant increase in G_{max} up to the apparent yielding at point C. The structural damage occurs at subsequent stressing, resulting in the stiffness state to rejoin the relevant SB curve of β - β at point E. The two SB curves of β - β and β^* - β^* eventually meet together at point F. The

corresponding behaviour for the relationship between $G_{\max}$ and σ'_v is shown in Fig. 8(c), in which the curves γ - γ and γ^* - γ^* correspond also to the referenced relationship of those two reconstituted samples.

Metastable (or Structure-permitted) Regions

In dealing with the behaviour of an NC clay as defined in terms of effective stress history, the shaded areas in Fig. 8 are regarded by the author as primary (or on-depositional) metastable (or structure-permitted) regions formed at deposition. On the other hand, the hatched zones refer to secondary (or post-depositional) metastable (or structure-permitted) regions enriched due to ageing (see also Fig. 7). The former exhibits the degree of metastability of a cited non-aged clay sample by comparing the current condition to the equivalent state of the fully destructured sample, whereas the latter can be identified by the existence of the aged structure. The current clay structure is taken as a consequence of the combined effects of on-depositional and post-depositional formations.

It should be emphasized that the effects of on-depositional clay structure relating to a primary metastable region are no longer evident as the current σ'_v exceeds the value at point F in Fig. 8, for which the effects of the aged structure alone are the matter of consideration. The author's interpretation of metastable regions is similar to, but slightly different from, the one described by Soga and Mitchell (1996), which in turn differs from the concept disregarding the primary region by Leroueil and Vaughan (1990).

Metastability Index (MI)

A metastability index (MI) previously proposed by Soga and Mitchell (1996) may be expressed in the following form (see Fig. 8(a));

$$MI(\sigma'_v) = (I_L - I_L^{\alpha^*})_{\text{current } \sigma'_v} \quad (7)$$

Obviously, the use of $MI(\sigma'_v)$ suffers from the fact that σ'_v is affected by the rate of stressing or straining. In addition, σ'_v was found to be unreceptive in reflecting clay structuration (refer to Fig. 1(a)).

On the contrary, the behaviour of $G_{\max}$ seems to reflect-ing concurrent structuration/destructuration in a more responsive and systematic way (see Figs. 1, 2, 3(b), 4(b) and 5(b)). Therefore, an alternative measure, $MI(G_{\max})$, is now proposed in the following;

$$MI(G_{\max})^e = (e - e^{\beta^* - \beta^*})_{\text{current } G_{\max}} \quad (8)$$

$$MI(G_{\max})^{I_L} = (I_L - I_L^{\beta^* - \beta^*})_{\text{current } G_{\max}} \quad (8')$$

The metastability index, $MI(G_{\max})^e$ is a structural measure of soil applicable to clay samples from the same origin which exhibit intrinsic properties in common (refer to Figs. 3(b), 4(b), 5(b) and 7). Alternatively, the use of $MI(G_{\max})^{I_L}$ is inherently more versatile (refer to Fig. 8(b)). The measure $MI(G_{\max})^{I_L}$ may be employed at two different levels; i.e., it is applicable to a series of clay samples at different depths and at different locations in a specific

Fig. 9. A demonstration of metastability index, $MI(G_{\max})^e$ in assessing structuration and destructuration in a reconstituted NSF clay sample

source ground which show slight variations in the intrinsic properties, and to clay samples from different sources.

The performance of $MI(G_{\max})^e$ in evaluating concurrent structuration/destructuration is demonstrated in Fig. 9, in which $MI(G_{\max})^e$ in the short-term ageing test on NSF clay (see Fig. 1) is examined with respect to the current $G_{\max}$. The $MI(G_{\max})^e$ value increased monotonically in accordance with increased $G_{\max}$ during the drained creep by showing the maximum at the end (point B), by which the sustained structuration due to ageing was well characterized. Consecutive destructuration on stressing was also successfully expressed by $MI(G_{\max})^e$ that showed a continuous decrease in value.

The performance of $MI(G_{\max})^{I_L}$ is shown in Fig. 10, in which the results of tests on Ariake, Bangkok and Louiseville clays were examined by means of the difference in I_L between each pair of the natural and reconstituted samples at the relevant $G_{\max}$ (see Figs. 3(a), 4(b) and 5(b)). The $MI(G_{\max})^{I_L}$ value for Ariake clay was considered at two different phases of recompression and virgin compression. Regarding the characteristics of the e - $\log G_{\max}$ relationship observed in Fig. 1(b), the straight lines, $(\beta^* - \beta^*)_{oc}$ and $(\beta^* - \beta^*)_{nc}$ in Fig. 3(b) were taken as a reference for the behaviour of the natural sample on the OC and NC states, respectively. Note that the $MI(G_{\max})^{I_L}$ value on recompression stayed at around 0.4~0.5, implying insignificant destructuration. Destructuration took place continuously as σ'_v increased in virgin compression involving a steady decrease in $MI(G_{\max})^{I_L}$. Restructuration at $\sigma'_v = 500$ kPa was clearly sensed by exhibiting an abrupt increase in $MI(G_{\max})^{I_L}$. The behavioural trend was similar for $MI(G_{\max})^{I_L}$ in tests on Bangkok clay, but the absolute value was much smaller compared to Ariake. In tests on Louiseville clay, $MI(G_{\max})^{I_L}$ remained also at a constant value of about 0.3 on recompression. As σ'_v exceeded σ'_{vy} , destructuration dominated the behaviour,

Fig. 10. A demonstration of metastability index, $MI(G_{\max})^L$ in assessing destructuration and restructuration in tests on Ariake, Bangkok and Louiseville clays

and the natural clay sample eventually reached the state of full destructuration by showing $MI(G_{\max})^L$ of zero.

In comparing the behaviour between the natural and reconstituted samples, the destructuration as well as the restructuration in a test on Ariake clay was successfully expressed by means of $MI(G_{\max})^L$. On the other hand, the difference was clearly observed in the magnitude of $MI(G_{\max})^L$ among these three clays. As stated earlier, Ariake clay, with its characteristic fabrics, and the high sensitivity is likely to build up larger primary metastable regions compared to the other two clays (refer to Fig. 6), which in turn will result in a larger in-situ $MI(G_{\max})^L$ value (refer to Mitchell, 1993). Note also that the natural Louiseville clay sample exhibited a complete structural breakdown when σ_v' reached four times $\sigma_v'(\text{in-situ})$, whereas the state of full destructuration was never met in tests on Ariake and Bangkok clays. The results may suggest that weakly-bonded fabrics can be readily destroyed by simply increasing the stress level. Conversely, the on-depositional effect is sustainable against stressing as observed in Ariake and Bangkok clays.

Soil Constant Reflecting Structure (S)-General

Another measure examined in this paper is a non-dimensional soil constant reflecting structure, S_{vh} (Jamiolkowski et al., 1994). At the outset, we should note that anisotropy in $G_{\max}$ is evident in most natural soils. If the shear wave propagates in the vertical direction, with the particle motion in a horizontal direction, $G_{\max}$ may be expressed by the following relationship;

$$G_{\max} = S_{vh} f(e) \sigma_r'^{(1-nv-nh)} \sigma_v'^{nv} \sigma_h'^{nh} \quad (9)$$

where

$f(e)$: void ratio function,
 σ_v' and σ_h' : current effective vertical and horizontal stresses,

σ_r' : a reference pressure (=98.1 kPa), and
 nv and nh : exponents.

Previous works suggest the equality of the two exponents (i.e., $n = nv = nh$) and also that $G_{\max}$ is independent of the magnitude of normal stress on the plane through which the shear body wave propagates (e.g., Roesler, 1979). In laboratory tests using vertically cut specimens, the material constant, S_{vh} , is thus obtained in the following form by fitting the result of tests in which the variation of $G_{\max}$ with the stress system (σ_v', σ_h') is examined;

$$S_{vh} = [G_{\max} / f(e)]_{\text{extrapolated at } (\sigma_v', \sigma_h') = 1} / \sigma_r'^{(1-2n)} \quad (10)$$

The function $f(e)$ employed in this paper is shown in Eq. (11), which has recently been proposed by Shibuya et al. (1997);

$$f(e) = (1 + e)^{-2.4} \quad (11)$$

This $f(e)$ has been derived by examining the relationship among e , $G_{\max}$ and σ_v' of various non-aged reconstituted clays as subjected to NC and OC in the BE test.

S-value of Non-aged Reconstituted Clays

The e -log σ_v' relationship of the nine reconstituted clays, each prepared at $I_L = 2$ at deposition is shown in Fig. 11. The corresponding e -log $G_{\max}$ relationship is shown in Fig. 12. Note that the behaviour of each clay followed the pattern shown in Fig. 7 (see also Eqs. (4) and (5)).

The S_{vh} -value, together with the exponent n , in each clay (see Eqs. (9) and (10)) was derived on the basis of data fitting shown in Fig. 13. The results are examined with respect to the plasticity index, I_p of each clay (Fig. 14). A few characteristic features observed were;

- the relationship between $G_{\max} / f(e)$ and (σ_v', σ_h') was in each clay scarcely affected by the stress history (i.e., NC or OC),
- the S_{vh} values, except for Ariake clay, were similar among the non-aged reconstituted clays, which all showed a wide spread of both compressibility and I_p (i.e., S_{vh} stayed within a narrow range between 1,600 and 2,400), and
- the scope of exponent n was also tight ranging from 0.23 to 0.33.

These observations strongly suggest that the soil structure can be evaluated by the S_{vh} value in a manner unaffected by intrinsic properties such as I_p (refer to Viggiani and Atkinson, 1995b).

S-value of Aged Natural Clays

Figure 15 illustrates how to determine the S_{vh} of an aged natural clay with reference to the S_{vh} of a non-aged reconstituted clay. The use of $f(e)$ may be most beneficial in respect that the variation of $G_{\max} / f(e)$ with the stress system of (σ_v', σ_h') becomes unique for a series of non-aged reconstituted samples of a given clay showing similar shape in the $G_{\max}$ - σ_v' relation (c.f., the curves γ - γ and γ^* - γ^* in Fig. 8(c)). This feature leads to a cancellation of on-depositional structural effects on $G_{\max}$, resulting in a sin-

Fig. 11. Relationship between e and σ'_v of nine non-aged reconstituted clays prepared at $I_L=2$ at deposition as subjected to one-dimensional compression (a) and (b)

Fig. 12. Relationship between e and $G_{\max}$ of nine non-aged reconstituted clays prepared at $I_L=2$ at deposition as subjected to one-dimensional compression (a) and (b)

gle S_{vh} value applicable to the non-aged reconstituted samples prepared at different I_L values at deposition. Accordingly, any deviation of S_{vh} compared to S_{vh} of the non-aged reconstituted samples is simply attributed to post-depositional aged structure.

The path from A to B stands for a long-term in-situ structuration due to ageing reflected in $G_{\max}$. As previously demonstrated (see Figs. 2, 3(b), 4(b) and 5(b)), the virgin compression beyond σ'_{vy} will involve some destructuration (path C-D-E). Conversely, any creep-strain will bring about further restructuration of the laboratory sample (path D-D') (see also Figs. 3(c) and 10). As these occur in the laboratory sample, the S_{vh} value of in-situ natural clay may be conversely over- and under-esti-

mated, respectively, as a consequence of careless data fitting. An example is shown in Fig. 16, in which the results of tests on the natural and reconstituted samples of Ariake clay are examined (refer to Fig. 3). The difference in S_{vh} reflects the in-situ structure of the natural sample with reference to the non-aged reconstituted sample (c.f., S_{vh} of 6,100 and 3,700 for the natural and reconstituted samples, respectively). On the premise that a single value of the exponent is applicable to the behaviour of an unbonded clay (Shibuya et al., 1997), an increase in S_{vh} from 6,100 to 6,800 reflected the restructuration which occurred due to two-day drained creep. On the contrary, a data fitting denoted by the dashed line resulted in S_{vh} of 6,100, indicating that the S_{vh} value was conversely

Fig. 13. Variation of $G_{\max}$ with respect to stress system ($\sigma'_v \sigma'_h$) of non-aged reconstituted clays (a) and (b)

Fig. 14. S -parameter with power, n , in bender element tests on non-aged reconstituted clays

Fig. 16. Determination of S_{vh} for pair of natural and reconstituted samples of Ariake clay sampled at 15.8 m depth

Fig. 15. Determination of S -parameter in laboratory testing

underestimated by mistake.

The relationship between $G_{\max}/f(e)$ and ($\sigma'_v \sigma'_h$) of the non-aged reconstituted samples is the lower bound against which the aged structure alone can be singled out (see Figs. 15 and 16). In an attempt to obtain in-situ value of S_{vh} , care must be taken so as to minimize effects of both destructuration and restructuration of the clay sample in the laboratory testing.

Predicting Time-dependent Behaviour of Soil Structuration

On the basis of the two-week ageing test performed on NSF clay (refer to Fig. 1), an attempt is made to predict the time-dependent behaviour of creep-strain induced structuration.

The variations of void ratio and $G_{\max}$ with time are

Fig. 17. Coefficient of secondary compression in short-term ageing test on NSF clay

Fig. 18. Power N_G in the short-term ageing test on NSF clay

shown in Figs. 17 and 18, respectively. It is widely acknowledged that the time-dependent behaviour of void ratio during drained creep at constant σ_v' can be expressed in the following form;

$$\Delta e_{cr} = C_\alpha \ln(t/t_{EOP}) \quad (C_\alpha > 0, t > t_{EOP}) \quad (12)$$

where $\Delta e_{cr}(>0)$ refers to accumulated change in void ratio after the EOP compression, and C_α denotes the coefficient of secondary compression (see Fig. 17). Similarly, G_{max} increases with time also in an exponential form (Afifi and Richart, 1973; Ishihara, 1996). The expression proposed by Shibuya et al. (1995b) takes the form of

$$(G_{max})_t / (G_{max})_{EOP} = (t/t_{EOP})^{N_G} \quad (t > t_{EOP}) \quad (13)$$

where $N_G(>0)$ is a power (see Fig. 18). The rate of increase in G_{max} with time is thus related to Δe_{cr} as expressed in the following (see Eqs. (12) and (13));

$$(G_{max})_t / (G_{max})_{EOP} = \exp(\Delta e_{cr} N_G / C_\alpha) \quad (14)$$

Fig. 19. Soil constant for metastabilization, C_β

Fig. 20. Prediction of development of G_{max} in NSF clay sample as subjected to long-term secondary compression

On the basis of Eqs. (5) and (14), the metastability index, $MI(G_{max})^e$ can be related to Δe_{cr} as expressed in the following (refer to Fig. 7);

$$MI(G_{max})^e / \Delta e_{cr} = [C_\beta / (1 - C_\beta)] \quad (15)$$

$$C_\beta = 1 - [C_\alpha / (v N_G)] \quad (0 < C_\beta < 1) \quad (16)$$

Note that C_β represents the ratio of slope A*-B to the slope of the relevant SB line of $\beta^* - \beta^*$ as shown in Fig. 7. By considering this feature, C_β is termed by the author the soil constant for metastabilization. By using C_β , the rate of increase in G_{max} associated with two different phenomenological effects; i.e., drained creep at constant stress (i.e., soil structuration) and stressing (i.e., increase in effective stress without creep-strain), can be interrelated. Provided that the rate parameter C_β stays constant throughout ageing, the ratio of $MI(G_{max})^e$ to Δe_{cr} of a

given soil is uniquely determined by C_β (Fig. 19). Incidentally, the condition of $C_\beta=1$ refers to a type of soil ageing involved with sudden development of interparticle bondings alone. A prediction of the time dependent behaviour of $G_{\max}$ is shown in Fig. 20, in which the result of the test on NSF clay was examined by applying the measured values of $t_{EOP}=270$ min, and the powers of $N_G=0.03$, $C_\alpha=0.002$ and $v=0.221$, hence C_β of 0.70 to Eqs. (14) and (15). It should be pointed out that the development of $G_{\max}$ during drained creep was successfully approximated by the straight line with the slope of $-(C_\alpha/N_G)$ in the e - $\ln G_{\max}$ diagram. The consecutive structuration with time, involved with the increase in $MI(G_{\max})^e$ was also predicted for a long-term ageing due to secondary compression.

CONCLUSIONS

Natural sedimentary clays are, without exception, structured due to in-situ ageing. A laboratory investigation, coupled with the results of relevant in-situ tests was carried out in an attempt to assess the in-situ structure of natural sedimentary clays based on the current elastic shear modulus from shear wave velocity measurement, $G_{\max}$.

- 1) In an assessment of the current soil structure, the $G_{\max}$ value of a fully-destructured (or reconstituted) clay can be employed as a yardstick against which the corresponding characteristic of the natural sedimentary clay is examined at the relevant effective stress.
- 2) The in-situ structure of natural clays comprises combined effects of on-depositional and post-depositional formations; the former refers to a characteristic structure formed at deposition, whereas the latter denotes the evolving structure enriched due to ageing involved with secondary compression and/or the growth of interparticle bondings.
- 3) A non-aged clay as subjected to one-dimensional compression exhibits the state boundary (SB) curve defined in 3D space of $(e, \sigma', G_{\max})$. Soil structuration involves violation of the SB curve in $(e, G_{\max})$ space, which is the key phenomenon with which the current structure of aged natural clay can be evaluated properly.
- 4) Similarly, destructuration in the aged clay is defined as continuous migration of the $(e, G_{\max})$ state towards the SB curve at the relevant effective stress. It was also observed that the aged clay showed some reduction in $G_{\max}$ on stressing involved with increased effective stress.
- 5) Based on the above-mentioned notions, together with the affirmative results of $G_{\max}$ measurements on various reconstituted as well as natural clay samples, a measure termed metastability index $MI(G_{\max})$ (refer to Eq. (8)) has been newly proposed by the author. This index using the time-dependent behaviour of $G_{\max}$ was found capable of representing structuration, destructuration and restructuration in clays in a systematic way when subjected to stressing and/or drained creep.
- 6) The soil constant reflecting structure, S -value (Jamiolkowski et al., 1994, see Eq. (9)) was also capable

of expressing concurrent structuration/destructuration in clays. When examined using $f(e)=(1+e)^{-2.4}$ (Shibuya et al., 1997), the S -value stayed within a narrow range between 1,600 and 2,400 for the eight non-aged reconstituted clays showing a wide spread in terms of both compressibility and I_p . In each clay, any deviation of the S -value, S_{aged} , against the S -value of the non-aged reconstituted clay, $S_{non-aged}$, was attributed to the presence of 'aged-structure', implying that the degree of clay structuration may also be evaluated by $S_{aged}/S_{non-aged}$, for example.

7) Soil constant for metastabilization, C_β , has been proposed by the author (see Eq. (16)). A prediction of time-dependent increase in $G_{\max}$ due to secondary compression involved with concurrent development of soil structure may be possible by using C_β , the capability of which was successfully demonstrated in tests using a reconstituted clay.

ACKNOWLEDGEMENTS

The author is grateful to Prof. T. Mitachi for his continuous encouragement and support during the course of this study. He is also grateful to Profs. F. Tatsuoka, D. W. Hight, S. Leroueil, G. Imai, A. S. Balasubramaniam, Dr. H. Tanaka and the late Dr. T. B. S. Pradhan with whom he has had free exchange of ideas on soil structure. Colleagues and students at Hokkaido University are also appreciated for their help in performing the laboratory tests. The natural clay samples in Ariake and Louiseville were provided by the Port and Harbour Research Institute.

REFERENCES

- 1) Adachi, T., Oka, F. and Mimura, M. (1996): "Modeling aspects associated with time dependent behavior of soils," *Measuring and Modeling Time Dependent Soil Behavior*, Sheahan, T. C. and Kaliakin, V. N. (eds.), ASCE GSP No. 61, pp. 61-95.
- 2) Afifi, S. E. A. and Richart, F. E. (1973): "Stress-history effects on shear modulus of soils," *Soils and Foundations*, Vol. 13, No. 1, pp. 77-95.
- 3) Anderson, D. G. and Woods, R. D. (1975): "Time dependent increase in shear modulus of clay," *J. Geotech. Engrg. Div., Proc. ASCE*, 102-GT5, pp. 525-537.
- 4) Anderson, D. G. and Stokoe, K. H. (1978): "Shear modulus: A time-dependent material property," *Dynamic Geotech. Testing*, ASTM STP654, pp. 69-90.
- 5) Asaoka, A., Nakano, M. and Noda, T. (2000): "Superloading yield surface concept for highly structured soil behavior," *Soils and Foundations*, Vol. 40, No. 2, pp. 99-110.
- 6) Balasubramaniam, A. S., Bergado, D. T., Long, P. V., Ashford, S. and Noppadol, P. (1996): "Development of ground improvement techniques in the Bangkok plain," *Proc. 12th SE Asian Geotech. Conf., Kuala Lumpur*, Vol. 2, pp. 71-92.
- 7) Bjerrum, L. (1973): "Problems of soil mechanics and construction on soft clays and structurally unstable soils (collapsible, expansive and others)," *Proc. of 8th Int. Conf. on SMFE, Stockholm*, Vol. 3, pp. 111-159.
- 8) Burland, J. B. (1990): "On the compressibility and shear strength of natural clays," *Géotechnique*, Vol. 40, No. 3, pp. 329-378.
- 9) Dyvik, R. and Madhus, C. (1985): "Laboratory measurements of $G_{\max}$ using bender elements," *Proc. of ASCE Convention, Detroit*, pp. 186-196.

- 10) Hight, D. W. and Higgins, K. G. (1994): "An approach to the prediction of ground movements in engineering practice: Background and application," Pre-failure Deformation of Geomaterials, Shibuya, S. et al. (eds.), Sapporo, Balkema, Vol. 2, pp. 817-836.
- 11) Imai, G. (1995): "Analytical examination for the foundations to formulate consolidation phenomena with inherent time-dependence," Keynote paper, Proc. Int. Symp. on Compression and Consolidation of Clayey Soils, Yoshikuni, H. and Kusakabe, O. (eds.), Hiroshima, Balkema, Vol. 2, pp. 891-935.
- 12) Ishihara, K. (1996): "Soil behaviour in earthquake geotechnics," Oxford Science Publications, Chapter 6.
- 13) Jamiolkowski, M., Lancellotta, R. and Lo Presti, D. C. F. (1994): "Remarks on the stiffness at small strains of six Italian clays," Pre-failure Deformation of Geomaterials, Shibuya, S. et al. (eds.), Sapporo, Balkema, Vol. 2, pp. 817-836.
- 14) Jardine, R. J. (1992): "Some observations on the kinematic nature of soil stiffness," Soils and Foundations, Vol. 32, No. 2, pp. 111-124.
- 15) Kokusho, T. (1987): "In-situ dynamic properties and their evaluations," Proc. of 8th Asian Regional Conf. on SMFE, Kyoto, Vol. 2, pp. 215-240.
- 16) La Rochelle, P., Sarraillh, J., Tavenas, F., Roy, M. and Leroueil, S. (1981): "Causes of sampling disturbance and a design of a new sampler for sensitive soils," Can. Geotech. J., Vol. 18, No. 1, pp. 52-66.
- 17) Leroueil, S. and Vaughan, P. R. (1990): "The general and congruent effects of structure in natural soils and weak rocks," Géotechnique, Vol. 40, No. 3, pp. 467-488.
- 18) Leroueil, S., Tardy, J., Roy, M., La Rochelle, P. and Konrad, J. M. (1991): "Effect of frost on the mechanical behaviour of Champlain sea clay," Can. Geotech. J., Vol. 28, pp. 690-697.
- 19) Leroueil, S. (1995): "Could it be that clays have no unique way of behaving during consolidation?," Proc. Int. Symp. on Compression and Consolidation of Clayey Soils, Yoshikuni, H. and Kusakabe, O. (eds.), Balkema, Vol. 2, pp. 1039-1048.
- 20) Leroueil, S. and Marques, M. E. S. (1996): "Importance of strain rate and temperature effects in geotechnical engineering," Measuring and Modeling Time Dependent Soil Behaviour, Sheahan, T. C. and Kaliakin, V. N. (eds.), ASCE GSP No. 61, pp. 1-60.
- 21) Lohani, T. N., Imai, G. and Shibuya, S. (1999): "Determination of shear wave velocity in bender element test," Proc. 2nd Int. Conf. on Earthquake Geotech. Engrg., Lisbon, Balkema, Vol. 1, pp. 101-106.
- 22) Mikasa, M. (1964): "A classification chart for engineering properties of soils," Tsuchi-to-Kiso, JGS, Vol. 12, No. 4, pp. 17-24 (in Japanese).
- 23) Mitchell, J. K. (1993): Fundamentals of Soil Behavior, 2nd Edition, John Wiley & Sons, New York.
- 24) Nakagawa, K., Mitchell, J. K., Soga, K. and Sadek, S. (1995): "Soil structure changes during and after consolidation as indicated by shear wave velocity and electrical conductivity measurements," Proc. Int. Symp. on Compression and Consolidation of Clayey Soils, Yoshikuni, H. and Kusakabe, O. (eds.), Hiroshima, Balkema, Vol. 2, pp. 1069-1074.
- 25) Oka, F., Yashima, A., Hashimoto, T. and Amemiya, M. (1996): "Application of Laval type large diameter sampler to soft clay in Japan," Soils and Foundations, Vol. 36, No. 3, pp. 99-111.
- 26) Ohtsubo, M., Egarashira, K. and Kashima, K. (1995): "Depositional and post-depositional geochemistry, and its correlation with the geotechnical properties of marine clays in Ariake Bay," Géotechnique, Vol. 45, No. 3, pp. 509-524.
- 27) Roesler, S. K. (1978): "Anisotropic shear modulus due to stress-anisotropy," J. Geotech. Engrg. Div., ASCE, Vol. 105, No. GT7, pp. 871-880.
- 28) 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.
- 29) Shibuya, S., Mitachi, T., Fukuda, F. and Degoshi, T. (1995a): "Strain rate effects on shear modulus and damping of normally consolidated clays," Geotech. Testing J., Vol. 18, No. 3, pp. 365-375.
- 30) Shibuya, S., Mitachi, T., Yamashita, S. and Tanaka, H. (1995b): "Effects of sample disturbance on G_{max} of soils—a case study," Proc. 1st Int. Conf. on Earthquake Geotech. Engrg., Ishihara, K. (ed.), Tokyo, Vol. 1, pp. 77-82.
- 31) Shibuya, S., Hwang, S. C. and Mitachi, T. (1997): "Elastic shear modulus of soft clays from shear wave velocity measurement," Géotechnique, Vol. 47, No. 3, pp. 593-601.
- 32) Shibuya, S., Hanh, L. T., Wilailak, K., Lohani, T. N., Tanaka, H. and Hamouche, K. (1998): "Characterizing stiffness and strength of soft Bangkok clay from in-situ and laboratory tests," Geotechnical Site Characterization, Robertson, P. K. and Mayne, P. W. (eds.), Atlanta, Balkema, Vol. 2, pp. 1361-1366.
- 33) Soga, K. and Mitchell, J. K. (1996): "Rate-dependent deformation of structured natural clays," Measuring and Modeling Time Dependent Soil Behavior, Sheahan, T. C. and Kaliakin, V. N. (eds.), ASCE GSP No. 61, pp. 243-257.
- 34) Tanaka, H., Sharma, P., Tsuchida, T. and Tanaka, M. (1996): "Comparative study on sample quality using several types of samplers," Soils and Foundations, Vol. 36, No. 2, pp. 57-68.
- 35) Toki, S., Shibuya, S. and Yamashita, S. (1994): "Standardization of laboratory test methods to determine the cyclic deformation properties of geomaterials in Japan," Prefailure Deformation of Geomaterials, Shibuya, S. et al. (eds.), Sapporo, Balkema, Vol. 2, pp. 741-784.
- 36) Tsuchida, T. and Gymyo, M. (1995): "Unified model of e -log p relationship with the consideration of the effect of initial void ratio," Compression and Consolidation of Clayey Soils, Yoshikuni, H. and Kusakabe, O. (eds.), Balkema, Vol. 1, pp. 379-384.
- 37) Tsuji, K., Loan, D. T. K. and Hanzawa, H. (1998): "Direct shear test results on Ariake, Bothkennar, Champlain and Drammen clays," J. of JSCE, No. 603/III-44, pp. 167-177 (in Japanese).
- 38) 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 SMFE, Bangkok, Vol. 2, pp. 101-170.
- 39) Tatsuoka, F., Jardine, R. J., Lo Presti, D. C. F., Di Benedetto, H. and Kodaka, T. (1997): "Characterizing the pre-failure properties of geomaterials," Theme Lecture for Plenary Session 1, Proc. of 13th Int. Conf. on SMGE, Hamburg, Vol. 4, pp. 2129-2164.
- 40) Viggiani, G. and Atkinson, J. H. (1995a): "Stiffness of fine grained soil at very small strains," Géotechnique, Vol. 45, No. 2, pp. 249-265.
- 41) Viggiani, G. and Atkinson, J. H. (1995b): "Stiffness of fine grained soil at very small strains," Géotechnique, Vol. 45, No. 2, pp. 249-265.