Deformation and strength properties of peat

Propriétés de déformation et résistance de la tourbe

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SYNOPSIS The both vertical and horizontal samples of undisturbed fibrous peat, which were obtained by maintaining the axes of thin-wall tubes parallel to the vertical and horizontal directions in the peat ground, were used. These samples contained the amount of organic matters more than 55 %. A series of the undrained triaxial compression and extension tests with pore water pressure measurement were performed on the isotropically normally and over-consolidated specimens, and the influence of fabric anisotropy on the undrained shear properties of peat was investigated. Moreover, based on the test results, the authors proposed a method of predicting stress - strain - strength relationships of peat under triaxial compression and extension conditions.

INTRODUCTION

It has been said that the peat, which is mainly composed of fibrous organic matters, i.e. partly decomposed plants such as leaves and stems, shows special shear properties in comparison with those of inorganic soils such as clay and sandy soils. Also, it is generally said that it is the material which shows strongly strength anisotropy. However, there are very few systematical studies on the shear properties of peat. Therefore, there are many problems which must be solved on the basic shear properties. In this paper, the influence of fabric anisotropy on the undrained shear properties of peat is presented. Also, based on the experimental facts and Cambrigde theory, the authors propose a new method to explain systematically the stress - strain behavior of normally consolidated peat under triaxial compression and extension conditions. Moreover, a method to estimate the strength parameters of over-consolidated peat by using the data obtained from undrained tests on normally consolidated peat is also presented.

PEAT TESTED AND TESTING PROCEDURE

The sample used in this study is fibrous peat with the physical properties shown in Table I. This saturated undisturbed sample was brought from the river side near Ohmiya city, Saitama, Japan. The sample contained a considerable amount of vegetal fibers. Then, we could not determine the values of liquid and plastic limits. The amount of organic matters is indicated by the ignition loss Lig. It is defined as the ratio of the weight lost by heating at a temperature of 800 °C to the total dry weight of peat sample. In sampling, in order to ensure the least variation among individual samples, they were sampled with the thin-wall tubes from the depth of 0.7 m to 1.5 m below the ground surface in same vicinity, and a care was taken to maintain the axes of samples parallel to the vertical and horizontal directions in the peat ground. These samples were designated as vertical (V) and horizontal (H) samples. V and H specimens, 50 mm in diameter and 125 mm in height, were trimmed from the V and H samples, respectively. These specimens were isotropically normally and over-consolidated in the triaxial cell with the initial back pressure of 100 kPa to insure saturation, and then sheared under the undrained compression and extension conditions. The shear tests with the rate of axial strain of 0.05 %/min were performed by increasing or decreasing axial stress σ_a while radial stress σ_r was maintained constant. The pore water pressure was measured at the bottom of the specimen.

TABLE I Physical Properties of Peat

Properties	Amounts
Natural void ratio e_n	10.5 - 14.2
Specific gravity G_p	1.55 - 1.63
Ignition loss Lig(%)	55 - 78
Decomposition D(%)	45 - 53
Saturation S_r(%)	100
pH	5 - 7
Carbon content C(%)	35 - 45
Total unit weight γ_t(KN/m³)	10 - 12
Preconsolidation stress p_y(kPa)	10 - 15
Liquid limit L.L.(%)	-
Plastic limit P.L.(%)	-

TEST RESULTS AND DISCUSSION

Volume Change Behavior Prior to Shear

Typical volume change behavior during isotropic compression and swelling prior to shear is plotted in Fig. 1, where Δln e and Δln p' are natural logarithmic void ratio and effective stress increments from the beginning of normal consolidation. These results were obtained from the three V specimens with the same Lig of about 60 % but the different preconsolidation pressures of 100 kPa, 200 kPa and 350 kPa. It is clear from this figure that the normal compression and swelling lines can be approximated by the straight lines, respectively, and furthermore the

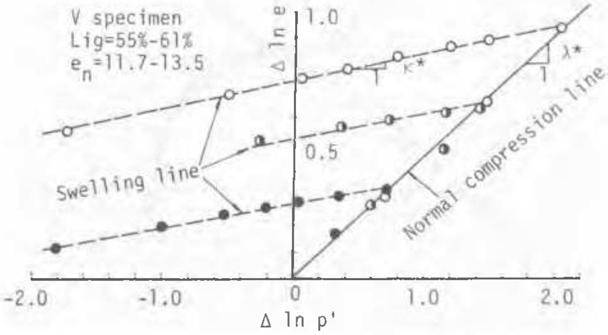

Fig. 1 Volume Change under Isotropic Stress Condition

three swelling lines show approximately parallel alignment. Therefore, the total void ratio change due to isotropic effective stress increment $(de)_c$ and its elastic and plastic components, $(de)_c^e$ and $(de)_c^p$ can be represented as follows.

$$(de)_c = -\lambda^* e_o \left(\frac{p'}{p'_o}\right)^{-\lambda^*} \frac{dp'}{p'} \tag{1}$$

$$(de)_c^e = e_o\left[(\lambda^*-\kappa^*)\left(\frac{p'}{p'_o}\right)^{-(\lambda^*-\kappa^*)} - \lambda^*\left(\frac{p'}{p'_o}\right)^{-\lambda^*}\right]\frac{dp'}{p'} \tag{2}$$

$$(de)_c^p = -(\lambda^*-\kappa^*)e_o\left(\frac{p'}{p'_o}\right)^{-(\lambda^*-\kappa^*)} \frac{dp'}{p'} \tag{3}$$

where p'_o is normal compression pressure at the beginning of stress increment and e_o is void ratio at p'_o.

Dilatancy Behavior

Presuming that the total volumetric strain v of soil element during drained shear is represented by the sum of change due to the increment of effective mean stress v_c and that due to the increment of deviatoric stress v_d, then

$$v = v_c + v_d \tag{4}$$

For undrained shear, putting the condition v = 0 into Eq. (4) and integrating Eq.(1) with the initial condition $e = e_o$ at p'_o, the following equation is obtained.

$$v_d = -\frac{e_o}{1+e_o}\left[1 - \left(\frac{p'}{p'_o}\right)^{-\lambda^*}\right] \tag{5}$$

That is, Eq.(5) expresses the volumetric strain due to the increment of deviatoric stress during drained shear, so-called, dilatancy, in terms of the change of effective mean stress during undrained shear. Accordingly, based on the data of undrained shear tests performed without any substantial change of volume, the volumetric strain calculated from Eq.(5) can be denoted as equivalent dilatancy. v_d computed from the data of undrained compression and extension tests on V and H specimens is shown in terms of stress ratio $\eta = q/p'$ in Fig. 2, where $q = \sigma_a - \sigma_r$ and $p' = (\sigma'_a + 2\sigma'_r)/3$. These specimens, having the almost same Lig of about 70 %, were isotropically normally consolidated in a range of consolidation pressure p'_o between about 100 kPa and 350 kPa. In the each case of four types of test conditions, v_d vs. η relationships will not be hardly affected by the magnitude of p'_o and v_d is approximately considered to be a unique function of η. However, v_d vs. η relationships of V specimens in compression and extension tests are very different from those of H specimens in the same tests. Similary, for the same V or H specimens, these relations under compression condition also differ very much from those under extension condition. This matter implies that the anisotropic fabric of fibrous peat is still kept after the isotropic compression and its effect appears remarkably during shear. It is interesting to note that v_d vs. η relationships in the four types of test conditions are almost symmetric with respect to the origin. That is, in the cases of V and H specimens, of which sampling direction is different at an angle of 90° each other, it is said that the compression and extension behavior of V specimens are very similar to the extension and compression behavior of H specimens, respectively. The experimental v_d vs. η relationships can be approximated by the solid lines (see Fig. 2) represented by the following equation.

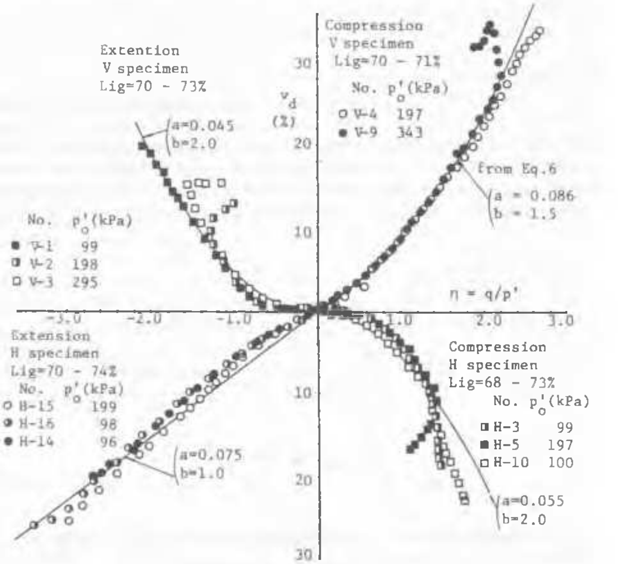

Fig. 2 Equivalent Dilatancy vs. Stress Ratio Relationships

$$v_d = F(\eta) = a\,\eta^b \tag{6}$$

where $F(\eta)$ is dilatancy function (Mitachi et al. 1979) and a and b are experimental coefficients.

Undrained Strength Parameters

Based on the test data of normally consolidated V and H specimens with Lig of about 70 % described in previous section, the states of the stress at failure, which were determined from the deviator stress maximum criterion, are plotted on q_f vs. p'_f plane in Fig. 3(a) and undrained shear strength $C_u = |q_f/2|$ and pore pressure coefficient A_f are plotted against consolidation pressure p'_o in Fig. 3 (b) and (c), where suffix f shows the state of failure. Symbols M and m in Fig. 3(a) are slope and vertical intercept of q_f vs. p'_f relationship approximated by a straight line. The effective angle of shear resistance ϕ' and cohesion intercept c' are given as follows.

$$\phi' = \sin^{-1}\left|\frac{3M}{6+M}\right|, \quad c' = \frac{3 \mp \sin\phi'}{6\cos\phi'}|m| \tag{7}$$

where the values of M and m in extension tests are negative. Since the straight lines in Fig. 3(a) have vertical intercepts, $q_f/2$ vs. p'_o relationships in Fig. 3(b) also have vertical intercepts but can be represented approximately by the straight lines. The absolute values of their slopes $\overline{C_u/p'_o}$ are corresponding to the rate of increase in undrained shear strength due to consolidation. Then, the various strength parameters obtained from undrained compression and extension tests are summarized in Table II. As seen in this table, the fibrous peat shows remarkably anisotropic strength properties. In compression tests, ϕ' of 52° of V specimens was very large value comparing with 35° of H specimens, and there was a difference of about 10° between compression and extension tests on V specimens. Also, the large values of $\overline{C_u/p'_o}$ more than 0.5 were measured and especially, the value of 0.8 from extension tests on H specimens was extremely large value. It is also expected that the normally consolidated peat has not only the large values of ϕ' and $\overline{C_u/p'_o}$ but also shows cohesion intercept c' due to the development of tension in fibers extending across the failure plane. The values of A_f in compression tests on V and H specimens and in extension tests on H specimens were closer to unity. However, this value from compression tests on H specimens was about 57 % from extension tests. Such the large values of peat concerning with ϕ' and C_u/p'_o have been also reported by the other research workers. In consolidated undrained compression tests on the undisturbed samples of Muck, the values of $\phi' = 78.3°$ and $C_u/p'_o = 0.63$ were indicated by Oikawa et al.(1980) and Adams (1962, 1965) and Edil et al. (1981) also obtained ϕ' of about 50°. The value of c' from compression tests on normally consolidated Muskeg after

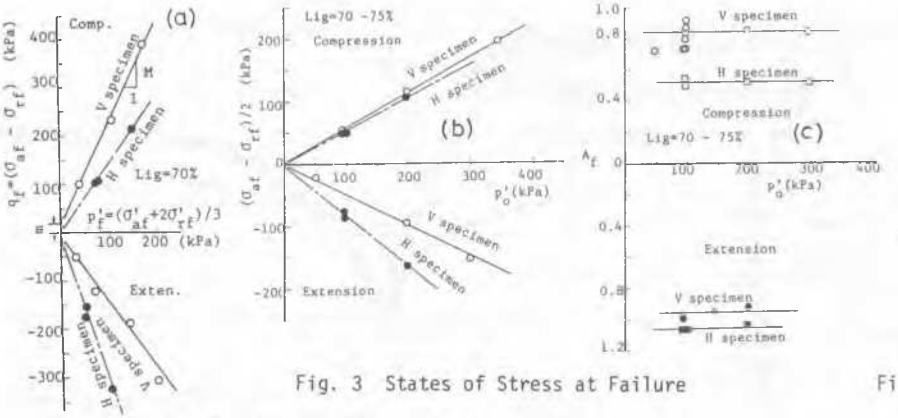

Fig. 3 States of Stress at Failure

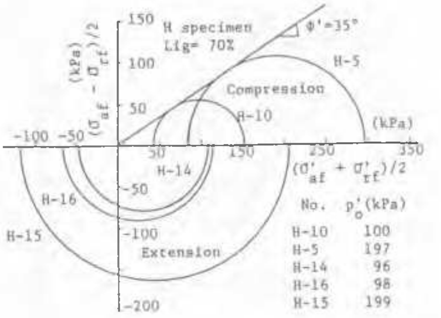

Fig. 4 Effective Mohr Circles at Failure

TABLE II Various Strength Parameters

Strength prameters	V specimen		H specimen	
	Comp.	Exten.	Comp.	Exten.
M	2.14	-1.36	1.43	-2.97
m (kPa)	7	-4	1	-5
ϕ' (°)	51.0	61.6	35.3	-
c' (kPa)	4.0	5.4	0.5	-
$\overline{C_u/p'_o}$	0.55	0.57	0.53	0.80
A_f	0.84	0.92	0.52	1.06

Adams (1962) was 14 kPa.

Now, as the value of M obtained from the extension tests on H specimens was -2.97, so, by substituting the values of M and m into Eq.(7), we could not decide both the values of ϕ' and c'. Because the range of M value corresponding to the condition $0° < \phi' < 90°$ must be from -1.5 to 3. Fig. 4 shows the effective Mohr circles obtained from compression and extension tests on H specimens. In the extension tests, the magnitudes of pore water pressure at failure become larger than those of minimum principal stress (axial stress σ_{af}). That is, the negative effective stress σ'_{af} is measured, and as the consolidation pressure p'_o increases this σ'_{af} shows further larger negative value. Therefore, it was impossible to decide the values of ϕ' and c' of H specimens under extension condition.

ANALYSIS

The stress - strain equations of normally consolidated peat under symmetrical triaxial compression and extension derived by the present authors are based on the theory of Roscoe et al.(1968) and the methods of Mitachi et al.(1979) and Yamaguchi et al.(1983) which make use of dilatancy function, which represents approximately the experimental dilatancy vs. stress ratio relationships of soils. Then, the assumptions of the present authors are essentially the same as those of them. As shown in Table II, the normally consolidated peat has cohesion intercept c' but in order to simplify the equations, we let c' equal to zero and assume that the critical state line (c.s.l.) becomes the straight line through the origin with the modified slope of M' on q vs. p' plane. Referring to the existing theories, and by using Eqs.(1)-(6), the basic equations of incremental stress - strain relationships for normally consolidated peat are given as follows (Yamaguchi et al. 1984, a).

$$d\varepsilon_s = -\frac{1+e_o}{1+e}\left[\frac{e_o(\lambda^*-\kappa^*)}{1+e_o}\left(\frac{p'}{p'_o}\right)^{-(\lambda^*-\kappa^*)}\frac{dp'}{p'} + F'(\eta)d\eta\right]x$$
$$\left[\frac{F'(\eta)}{F'(\eta)\eta - \frac{e_o(\lambda^*-\kappa^*)}{1+e_o}\left(\frac{p'}{p'_o}\right)^{-(\lambda^*-\kappa^*)}}\right] \qquad (8)$$

$$dv = \frac{1+e_o}{1+e}\left[\frac{\lambda^* e_c}{1+e_o}\left(\frac{p'}{p'_o}\right)^{-\lambda^*}\frac{dp'}{p'} + F'(\eta)d\eta\right] \qquad (9)$$

where $d\varepsilon_s = d\varepsilon_a - dv/3$, $d\varepsilon_a$ and dv are axial strain and volumetric strain increments and $F'(\eta)$ is differential form of $F(\eta)$. In undrained tests, applying the conditions dv = 0 and $e = e_o$, the equations of effective stress path and incremental stress - strain relationship become as follows.

$$\left(\frac{p'}{p'_o}\right) = \left[1 + \left(\frac{1+e_o}{e_o}\right)F(\eta)\right]^{\frac{1}{-\lambda^*}} \qquad (10)$$

$$\frac{d\varepsilon_s}{d\eta} = \frac{\left[\left(\frac{\lambda^*-\kappa^*}{\lambda^*}\right)\left\{1+\left(\frac{1+e_o}{e_o}\right)F(\eta)\right\}^{-\kappa^*/\lambda^*} - 1\right] F'(\eta)^2}{\eta F'(\eta) - \frac{e_o(\lambda^*-\kappa^*)}{1+e_o}\left[1+\left(\frac{1+e_o}{e_o}\right)F(\eta)\right]^{\frac{\lambda^*-\kappa^*}{\lambda^*}}} \qquad (11)$$

By applying the conditions at critical state, $d\eta/d\varepsilon_s = 0$ and $d\varepsilon_s >> 0$, to Eq.(11), the value of κ^* can be obtained from the following equation.

$$\kappa^* = \lambda^* + \frac{\lambda^*}{2F(M')}\left[\frac{e_o}{1+e_o} - \sqrt{\left(\frac{e_o}{1+e_o}\right)^2 + \frac{4M'F'(M')F(M')}{\lambda^*}}\right] \qquad (12)$$

where $F'(M')$ and $F(M')$ are $F'(\eta)$ and $F(\eta)$ at $\eta = M'$, respectively.

Next, based on the experimental fact which the states of effective stress at failure for normally and over consolidated peat can be illustrated as Fig. 5 (Yamaguchi et al., 1984, b), and further applying the failure criterion of Hvorslev, the undrained strength parameters of over-consolidated peat can be represented with those of normally consolidated peat as follows (Yamaguchi et al., 1984, b).

$$(C_u/p'_o)_{OP} = (\overline{C_u/p'_o})_{NP}\,(OCR)^{1-\alpha} \qquad (13)$$

$$(A_f)_{OP} = (A_f)_{NP} - \frac{1}{2(\overline{C_u/p'_o})_{NP}}\left[1 - (OCR)^{\alpha-1}\right] \qquad (14)$$

where suffixes NP and OP signify normally and over-consolidated peats, OCR is over-consolidation ratio and $\alpha = \kappa^*/\lambda^*$. The above equations can be derived based on the assumptions that the failure envelope with a slope of C_{nf} for normally consolidated peat is parallel to the normal consolidation line with a slope of λ^* and furthermore the slope of C_{nf} is approximately equal to the slope of fail-

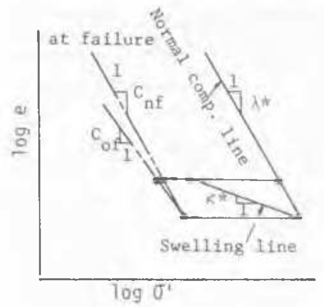

Fig. 5 Schematic Diagram of Stress States pricr to Shear and at Failure

TABLE III Values of Coefficients Used

No.	Lig (%)	p'_o (kPa)	e_o	λ*	a	b	M'
V-4	70	197	5.05	0.450	0.086	1.5	2.1
H-10	68	100	6.68	0.428	0.055	2.0	1.4
V-3	73	295	3.71	0.452	0.045	2.0	-1.6
H-16	74	98	6.41	0.401	0.075	1.0	-3.3

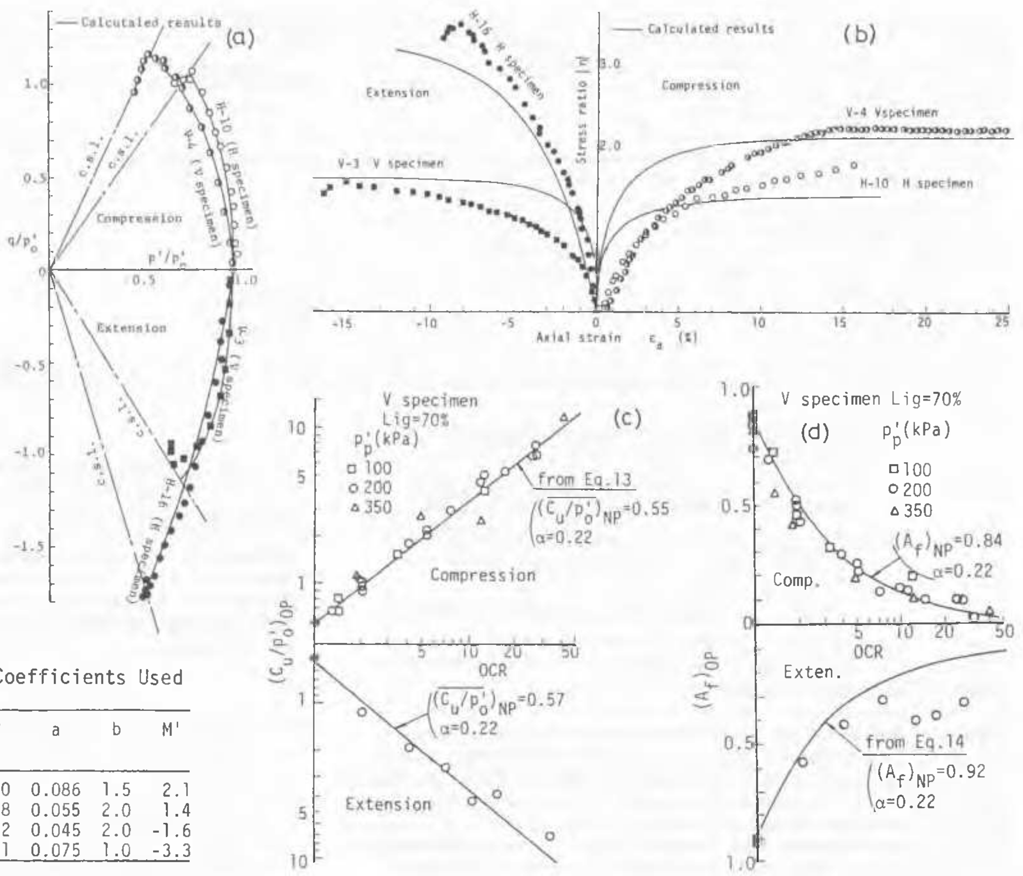

Fig. 6 Comparisons of the Experimental Results with the Calculated Results

ure envelope C_{of} for over-consolidated peat.

For the cases of compression and extension tests on normally consolidated V and H specimens mentioned previously, the typical comparisons of the experimental results with the calculated results (solid lines) from Eqs.(10) and (11) are shown in Figs. 6(a) and (b). The coefficients prepared for the calculation are indicated in Table III. Also, in Figs. 6(c) and (d), the undrained shear strength C_u/p'_o and pore pressure coefficient A_f obtained from the compression and extension tests on over-consolidated V specimens with the three different preconsolidation pressures p'_p of 100 kPa, 200 kPa and 350 kPa are compared with the calculated results (solid lines) from Eqs.(13) and (14). From these comparisons, as a first approximation, it may be seen that the present method will be very useful to understand systematically the undrained deformation - strength properties of peat.

CONCLUSIONS

i) The anisotropic fabric of peat is kept yet after the end of isotropic compression and the anisotropic shear properties are observed remarkebly.
ii) The method proposed by the present authors is very useful to discuss systematically and quantitatively the deformation - strength properties of peat under triaxial compression and extension conditions.

REFERENCES

Adams, J. I.(1962). Laboratory compression test on peat, Ontario Hydro Research News, third quarter, pp. 55 - 40.

Adams, J. I.(1965). The engineering behaviour of a Canadian Muskeg, Proc. of 6th ICSMFE, Vol. 1, pp. 3 - 9.

Edil, T. B. and Dhowian, A. W.(1981). At-rest lateral pressure of peat soils, Proc. ASCE, Vol. 107, No. GT 2, pp. 201 - 217.

Mitachi, T. and Kitago, S.(1979). The influence of stress history and stress system on the stress - strain - strength properties of saturated clay, S & F, Vol. 19, No. 2, pp. 45 - 61.

Oikawa, H. and Miyakawa, I.(1980). Undrained shear characteristics of peat, Jour. of JSSMFE, Vol. 20, No. 3, pp. 92 - 100 (in Japanese).

Roscoe, K. H. and Burland, J.B.(1968). On the generalized stress - strain behavior of wet clay, In Engineering Plasticity, Cambridge Univ. Press, pp. 535 - 609.

Yamaguchi, H., Mitachi, T. and Kitago, S.(1983). Dilatancy of clay during secondary consolidation, 7th Asian Regional Conf., SMFE, Vol. 1, pp. 387 - 392.

Yamaguchi, H., Ohira Y., Kogure, K. and Mori, S.(1984, a). Undrained stress vs. strain relationships of normally consolidated peat under triaxial compression and extension conditions, 19th Japan National Conf., SMFE, pp. 481 - 484 (in Japanese).

Yamaguchi, H., Mori, S. and Ohira, Y.(1984, b). Undrained shear strength of over-consolidated peat under compression and extension conditions, 19th Japan National Conf., SMFE, pp. 485 - 488 (in Japanese).