Consolidation Phenomena of Normally Consolidated Clays Due to the Disturbance Caused by Undrained Shear

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Consolidation settlement can occur in surrounding ground under its own weight due to clay disturbance following the decrease of effective stress, since it is inevitable that the adjacent clay suffers undrained shear deformation during various civil engineering works in soft clay ground. In this paper such consolidation phenomena of normally consolidated clays due to their disturbance are investigated and it is concluded from experiments and theoretical considerations that the volume decrease of normally consolidated clays necessary to predict the ground subsidence behavior is obtained by using the magnitude of shear strain during undrained shear and clay sensitivity and plasticity index, which related to the susceptibility to clay disturbance.

Clay disturbance is defined as the decrease of effective stress derived from the change of clay structure, which means the collapse of solid contacts between peds (or aggregates of clay platelets) and coarser particles by shear deformation, under constant total stress condition. The disturbance ratio R defined as the change of effective stress is adopted as the measure of clay disturbance, for the change of mechanical characteristics of clays due to their disturbance is essentially caused by the change of effective stress. The disturbance ratio R or its increment r is defined by Eq. (1).

$$R=\sigma_0'/\sigma_r', \quad r=R-1=(\sigma_0'-\sigma_r')/\sigma_r' \cdots\cdots(1)$$

in which σ_0'; effective stress at undisturbed state (corresponding to the initial isotropic consolidation effective stress in the experiments described later)

σ_r'; residual effective stress at disturbed state to some degree.

However it is assumed that the unique relationship should exist between this disturbance ratio and the magnitude of shear strain, because clay disturbance is originally derived from the change of clay structure by shear deformation. And it is reasonable to evaluate the disturbance ratio from the magnitude of ground displacement or shear strain comparatively easily obtained, for example, by finite element analysis, since it is difficult to predict directly the change of effective stress (or excess pore water pressure in saturated clay) when the change of total stress condition is indeterminate. So the relationship between the disturbance ratio and the magnitude of shear strain during undrained shear (undrained shear stress loading) is deduced according to Anisotropic Cam Clay model (Ohta et al. (1975))[1] in case of undrained shear stress loading and unloading for isotropically normally consolidated clay as in the experiments described later and represented by Eq. (2).

$$\left.\begin{array}{l} R=\exp(A-(1-\exp(-B\cdot\gamma_{oct}))) \\ A=1-(\kappa/\lambda)^2, \quad B=3\lambda/\kappa\cdot\mu \end{array}\right\} \cdots\cdots(2)$$

in which γ_{oct}; octahedral shear strain $(=\sqrt{(\epsilon_1-\epsilon_2)^2+(\epsilon_2-\epsilon_3)^2+(\epsilon_3-\epsilon_1)^2/3})$, λ; compression index, κ; swelling index, μ; dilatancy constant.

Using this R versus γ_{oct} relationship R can be obtained from the magnitude of shear strain during undrained shear, but this may be approximated as linear relationship until the magnitude of shear strain attains to 1.5% (Fig. 1). So the relationship between R (or r) and γ_{oct} (or γ) is assumed to be given by Eq. (3).

$$r=R-1=k'\cdot\gamma_{oct} \quad \text{or} \quad r=k\cdot\gamma \quad \cdots\cdots\cdots\cdots(3)$$

in which $k=(\sqrt{2}/3)k'$, $\gamma=\epsilon_1-\epsilon_3$ (%).

This constant k contained in Eq. (3) is the parameter related to the susceptibility to clay disturbance, which controls the magnitude of r in case of suffering given magnitude of shear strain, dependent on clay type or difference in clay structure.

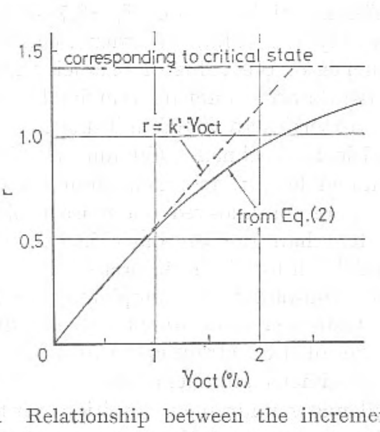

Fig. 1 Relationship between the increment of disturbance ratio r and the magnitude of shear strain γ_{oct} during undrained shear stress loading.

Table 1 Sample properties.

Sample		1	2	3	A	B	C	D	E
Specific Gravity	G_s	2.66	2.69	2.66	2.67	2.67	2.69	2.71	2.62
Liquid Limit	w_L (%)	63.5	43.9	47.1	23.6	33.9	38.4	55.0	74.5
Plasticity Index	I_p	26.6	22.7	18.2	3.7	8.6	15.2	24.9	44.3
Initial Effective Stress	σ_0' (kP_a)	150	44	71	105	78	78	78	78
Range of Void Ratio	e_0	1.72±0.05	1.27±0.06	1.42±0.06	0.67±0.03	0.74±0.03	0.88±0.02	1.15±0.01	1.21±0.03
Sensitivity	S_t	29.7	12.5	45.4	4.2	4.6	4.8	5.9	4.0
Compression Index	C_c	0.76	0.43	0.52	0.12	0.20	0.25	0.32	0.47

It is assumed that consolidation phenomena of normally consolidated clays due to such disturbance can occur when effective stress once decreased to σ_r' at disturbed state to some degree increases up to σ_0' at initial state under its own weight. In this case volume decrease of clay α can be represented by using the disturbance ratio R or the magnitude of shear strain γ as in Eq. (4).

$$\alpha=(C_c'/(1+e_0))\log R$$
$$=(C_c'/(1+e_0))\cdot\log(k\cdot\gamma+1) \quad \cdots\cdots\cdots\cdots\cdots\cdots (4)$$

in which e_0 is the initial void ratio before undrained shear and C_c' is the compression index of disturbed clay in the reconsolidation region (Fig. 2).

As it is considered that clay disturbance is influenced by type of clay or difference in clay structure, three of samples used in the experiments ($S_t=12.5-45.4$) were prepared by sampling clay blocks from three fields in Tokyo taking care lest they should suffer mechanical disturbance, named undisturbed samples. Other five samples were prepared by reconsolidating the alluvial clay—Toyoura sand mixtures having four different mixing ratios ($I_p=3.7-24.9$) and Kibushi clay ($I_p=44.3$) in order to investigate the influence of the value of plasticity index on clay disturbance, named remolded samples. Sample properties are listed in Table 1.

Cylindrical specimens (50 mm in diameter ×110 mm in height) trimmed from these block samples were consolidated under isotropic stress σ_0' greater than the preconsolidation stress in the triaxial cell for about 48 hours. After initial isotropic consolidation completed, undrained triaxial tests were performed to give different magnitudes of shear strain up to 10% axial strain to clay specimens and shear stress was gradually unloaded under undrained condition until initial isotropic stress state reached.

Then pore water pressures at that state Δu_r $(=\sigma_0'-\sigma_r')$ were measured and clay specimens were consolidated again under the same isotropic stress condition as in the initial consolidation. In this way the relationships between the change of effective stress or the volume decrease of specimens due to clay disturbance and the magnitude of shear strain during undrained shear stress loading were investigated.

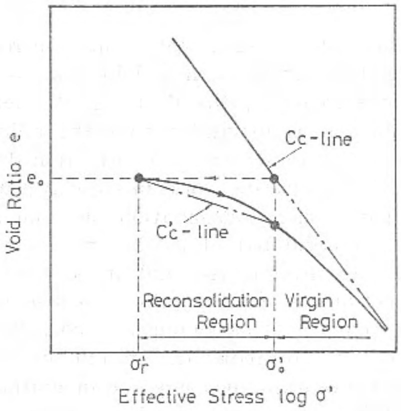

Fig. 2 Schematic diagram of the consolidation phenomena of normally consolidated clays due to the disturbance caused by undrained shear.

According to these test results, the increment of disturbance ratio r versus shear strain γ relationship are shown in Fig. 3 (a), (b), (c), (d), for example. It is clear from these figures that linear relationships exist between r and γ also in the range of the magnitude of shear strain greater than the value corresponding to $\gamma_{oct}=1.5\%$ ($\gamma\geqq3.2\%$). And the parameter k related to the susceptibility to clay disturbance, corresponding to the gradient of r versus γ relationship, is different for each sample. Parameter k is greater for undisturbed samples having greater clay sensitivity, and for remolded samples having equal sensitivity the value of plasticity index is smaller, parameter k is greater. So parameter k is assumed to be represented by Eq. (5) as func-

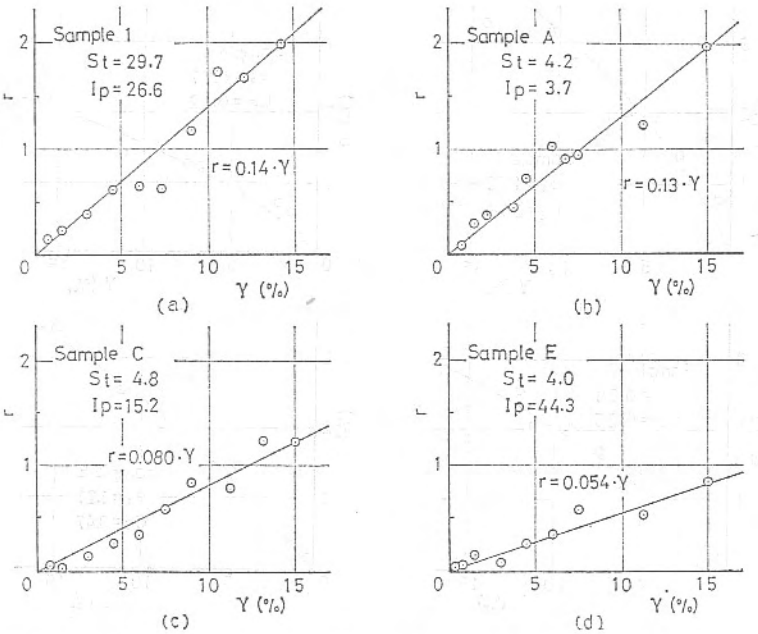

Fig. 3 Relationships between the increment of disturbance ratio r and the magnitude of shear strain γ during undrained shear stress loading.

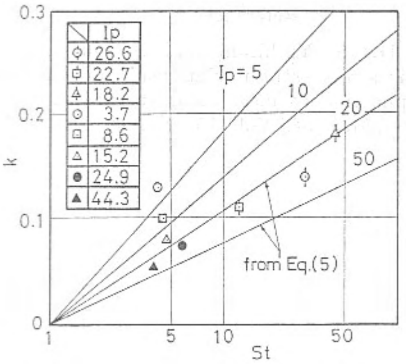

Fig. 4 Relationship between parameter k and clay sensitivity S_t in function of I_p.

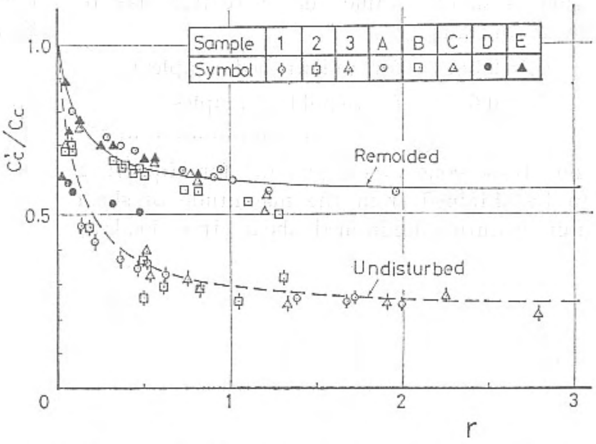

Fig. 5 Relationship between compression index ratio C_c'/C_c and the increment of disturbance ratio r.

tion of clay sensitivity (S_t) and the value of plasticity index (I_p).

$$k=0.33\cdot I_p^{-0.37}\cdot\log S_t \quad \cdots\cdots(5)$$

In Fig. 4 parameter k obtained from these test results are plotted on semi-logarithmic chart k versus S_t in function of I_p.

Compression index of disturbed clay in the reconsolidation region C_c' is also investigated according to test results. Figure 5 shows compression index ratio C_c'/C_c (C_c; compression index at undisturbed state) versus the increment of disturbance ratio r. C_c' decreases compared with C_c and its way of decrease is different for undisturbed samples and for remolded ones. But it is assumed to be convenient to use constant C_c' value for undisturbed samples and for remolded ones, as C_c' value decreases rapidly in the

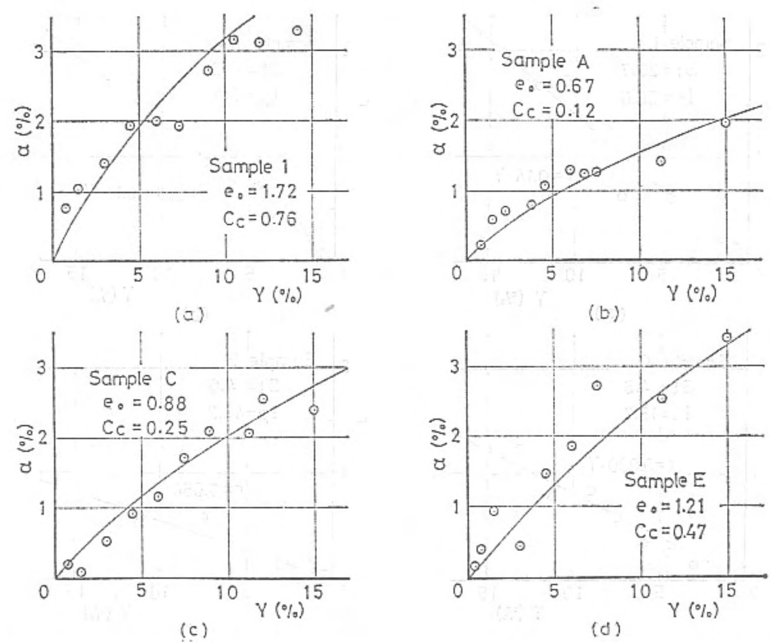

Fig. 6 Relationships between the volume decrease α due to clay disturbance and the magnitude of shear strain γ during undrained shear stress loading.

region of small r value and thereafter may be almost constant.

$$\left.\begin{array}{ll} C_c'=0.3C_c & \text{for undisturbed samples} \\ C_c'=0.6C_c & \text{for remolded samples} \end{array}\right\} \quad \cdots\cdots(6)$$

Using these constants k and C_c' for Eq. (4). α can be obtained from the magnitude of shear strain γ during undrained shear stress loading. For example α versus γ relationships are shown in Fig. 6 (a), (b), (c), (d).

Reference

1) Ohta, H., S. Yoshitani and S. Hata: Anisotropic stress—strain relationship of clay and its application to finite element analysis, Soils and Foundations, Vol. 15, No. 4, pp. 61–79, 1975.