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A PRACTICAL MODEL FOR SECONDARY COMPRESSION

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ABSTRACT

A practical method for estimating the settlement of soft grounds including secondary compression has been proposed in the former papers (1979, 1980). The theory can exclude the difficulty that the vertical strain at 100% primary consolidation increases with the thickness of clay layer, owing to the secondary compression involved in the process of excess pore-water pressure dissipation, that is, during primary consolidation.

The basic postulates, which were employed in the proposed equation in the former papers, are firstly reexamined. Among them, it is attempted to make clear the physical meaning of the following basic equations

$$\Delta e = f(\sigma_v, t) \qquad \text{(i)}$$

$$\Delta e = \Delta e_p + \Delta e_s \qquad \text{(ii)}$$

where σ_v; vertical pressure, t; elapsed time, Δe; change in void ratio, Δe_p, Δe_s; void ratio changes due to primary consolidation and secondary compression, respectively. Eq. (i) is common with primary consolidation and secondary compression. As a result of the formulation by means of the both relations (i) and (ii), it is confirmed that the method proposed herein is in good agreement with the new concept of settlement which Bjerrum (1973) had brought out in analysis of settlement of structures in Norway. In addition, numerical investigations on the influences of several factors on the settlement versus time relations are performed by incorporating the experimental finding by Mesri (1973) that the ratio of the coefficient of secondary compression to the compression index, $C_\alpha/C_c(=R)$ is kept constant depending on the soil type into the basic equation. In numerical investigations, special emphasis is laid on the influence of the thickness of clay layer on settlement versus time curves.

Computed time-settlement curves are compared with observations of settlement by Aboshi (1973) which are most reliable so far regarding the scale effect of one-dimensional consolidation of clay soil. This comparison suggests that the present theory will be valid especially for settlement prediction of soft grounds which exhibits predominant secondary compression.

Key words : consolidation, consolidation test, void ratio, time effect, settlement, soft ground, secondary compression, clay (**IGC** : D 5/E 2)

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INTRODUCTION

Secondary compression has been one of the most up-to-date topics in consolidation problems since the Terzaghi's theory was published. There have been several approaches to the solution for settlement analysis containing secondary compression. However, after many years of research into secondary compression, no reliable method has yet been available for calculating the magnitude and the rate of such settlement. For this reason, in case when estimates of secondary compression are required in practice, there are generally based on empirical procedures. Attempts to formulate time-settlement observations in the laboratory, and in the field into a practical relation were carried out by Buisman (1936), Koppejan (1948) and Zeeveart (1975). These researches fall into category of the experimental approach to secondary compression.

On the other hand, Taylor and Merchant (1940), Barden (1968) and Sekiguchi (1976) tried to explain secondary compression by numerical method with the consideration of rheological properties of cohesive soils. However, it should be emphasized that these theories have a special contrast in formulation of each theory. Namely, Barden did utilize rheological model while Sekiguchi did not employ such a model.

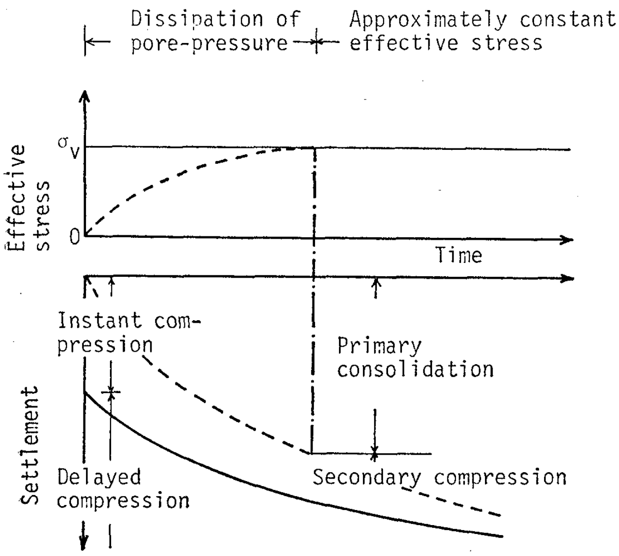

Fig. 1 Relationship between conventional and Bjerrum's concepts for settlement

Instead of the conventional concept of primary consolidation and secondary compression, a new concept that the settlement consists of instant and delayed compression was introduced by Bjerrum (1967). His conceptual sketch for the settlement computation is shown in Fig. 1.

Recent experimental researches have given some important and valid information. Aboshi (1973, 1981) run a series of large-scale oedometer tests in the laboratory and the field to examine the scale effect on one-dimensional consolidation, and concluded that the vertical strain corresponding to 100% primary consolidation increases with the initial thickness of the clay specimen. Besides this, he deduced that the increased vertical strain with the thicker specimen might be caused by secondary compression contained in the process of pore-water pressure dissipation. Mesri and Choi (1979) pointed out that when a clay with a vertical pressure higher than the overburden pressure was subjected to increment loads spanning the critical pressure, an abnormal pore-water pressure developed, which did not obey the conventional theory, and at the same time the predominant secondary compression was observed. Assuming that secondary compression is derived from dilatancy due to the principal stress difference exerted in the process of one-dimensional consolidation, Inada and Akaishi (1980) proposed a method for estimating secondary compression with consideration of the effect of dilatancy.

The other practical method for predicting settlement of soft soils including secondary settlement has been proposed by the author (1978, 1980). In the present paper, basic postulates which have been contained in the proposed equation are examined, and it is attempted to make clear the physical meaning of the basic equation for a method of settlement computation. In addtion, by incorporating the experimental finding by Mesri and Godlewski (1977) into the method, the numerical examination

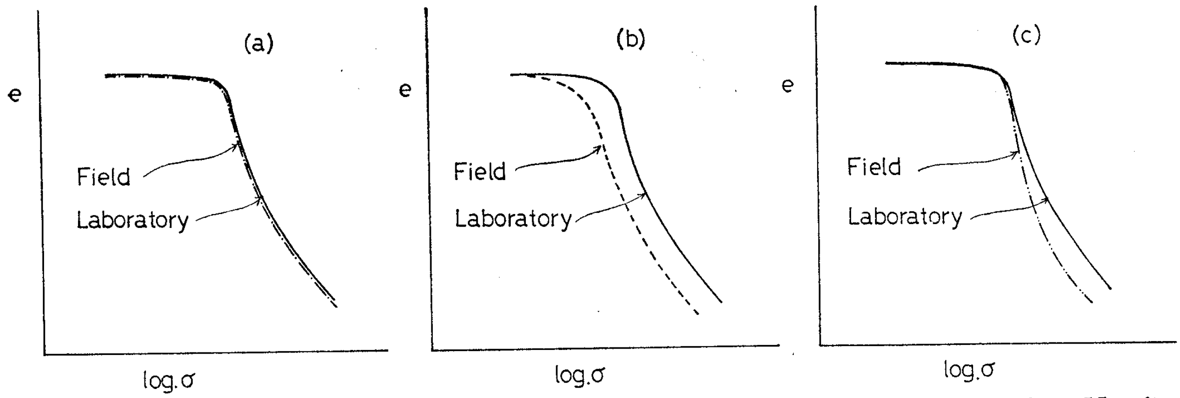

Fig. 2. Three possibilities regarding the scale effect in consolidation (After Mesri)

concerning the influences of several factors on time-settlement relation is performed. Computed time-settlement relations are compared with observations in the laboratory which were carried out by Aboshi (1973, 1981).

PRELIMINARY CONSIDERATION

Many attempts to establish the consolidation equation that takes account of the non-linear stress-strain relation for soils into consideration have been performed for secondary compression analysis. In fact, these attempts have been the main current of theoretical approaches to secondary compression after Taylor and Merchant (1940) released their theory. They started with the following fundamental expression

$$de=\left(\frac{\partial e}{\partial \bar{\sigma}}\right)_t d\bar{\sigma}+\left(\frac{\partial e}{\partial t}\right)_{\bar{\sigma}} dt \qquad (1)$$

This expression has been drawn from the assumption that the void ratio change during consolidation is a function of the effective stress, $\bar{\sigma}$, and the elapsed time, t, i.e., we have

$$\Delta e=f_1(\bar{\sigma}, t) \qquad (2)$$

This is different from the well-known Terzaghi's assumption

$$\Delta e=f_2(\bar{\sigma}) \qquad (3)$$

Meantime, remarkable development has recently been made in the constitutive equation for soils. Provided that the more realistic stress-strain relation would be combined with the consolidation equation, the powerful

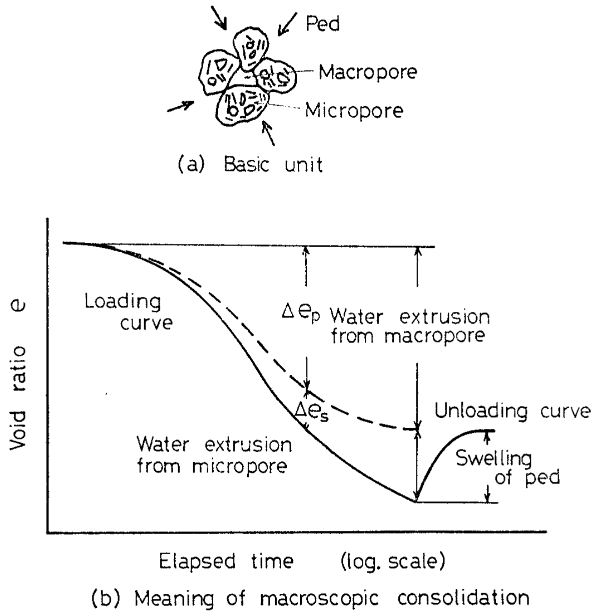

Fig. 3. Mechanism of behavior of peds and pores in consolidation of clay

method for secondary compression described in this paper will be proposed by following this type of procedure.

The scale effect in consolidation is taken as the most difficult and important problem to be considered in the construction of a theory. Fig. 2 shows three possiblities of e-log σ relations between the laboratory and the field. Each curve corresponds to the 100% primary consolidation. Fig. 2(a) corresponds to the Terzaghi's theory, Fig. 2(b) supports the experimental results by Aboshi which will be mentioned later, and Fig. 2(c) is the model lying between models in Fig. 2 (a) and Fig. 2 (b). Unfortunately, there has been no experimental data for the scale effect of consolidation up to the present time except Aboshi's experiences.

BASIC EQUATION FOR AN ANALYTICAL MODEL

The following postulates are employed for presentation of a method for accessing the one-dimensional consolidation of soft clay ground with the special reference to secondary compression.

i) The void ratio change due to both primary consolidation and secondary compression is expressed as a function of the effective stress and the elapsed time. This is same as the original postulate used by Taylor and Merchant (1941).

ii) The void ratio change during consolidation which consists of primary consolidation and secondary compression is related to drainages from the macro-pore and micro-pore, respectively, as schematically illustrated in Fig. 3 (Yong et al., 1975; Kamon, 1979).

iii) Secondary compression in the thin sample is not appeared in the process of dissipation of pore-water pressure.

iv) The coefficient of secondary compression, C_α, and the coefficient of consolidation, c_v, are independent of the load increment ratio (Mesri and Godlewski, 1977).

v) The time duration necessary for pore-water pressure dissipation increases in proportion to the square of the thickness of a clay layer followed by the Terzaghi's theory.

The procedure of derivation of the governing equation for secondary compression analysis is as follows:

From the postulate (i), we assume again

$$\Delta e=f(\bar{\sigma}_v, t) \tag{4}$$

where $\bar{\sigma}_v$; the vertical effective stress, t; the elapsed time. Differentiation of Eq. (4) yields to

$$d(\Delta e)=\left(\frac{\partial(\Delta e)}{\partial\bar{\sigma}_v}\right)_t\cdot d\bar{\sigma}_v+\left(\frac{\partial(\Delta e)}{\partial t}\right)_{\bar{\sigma}_v}\cdot dt \tag{5}$$

Then, since, according to the postulate (ii), both primary consolidation and secondary compression contribute to the total void ratio change, we can assume

$$\Delta e=\Delta e_p+\Delta e_s \tag{6}$$

Differentiating Eq. (6), we get

$$\left(\frac{\partial(\Delta e)}{\partial t}\right)_{\bar{\sigma}_v}=\left(\frac{\partial(\Delta e_p)}{\partial t}\right)_{\bar{\sigma}_v}+\left(\frac{\partial(\Delta e_s)}{\partial t}\right)_{\bar{\sigma}_v} \tag{7}$$

$$\left(\frac{\partial(\Delta e)}{\partial\bar{\sigma}_v}\right)_t=\left(\frac{\partial(\Delta e_p)}{\partial\bar{\sigma}_v}\right)_t+\left(\frac{\partial(\Delta e_s)}{\partial\bar{\sigma}_v}\right)_t \tag{8}$$

In Eq. (7), the void ratio change due to primary consolidation, Δe_p, is governed by the Terzaghi's theory, so we obtain

$$\left(\frac{\partial(\Delta e_p)}{\partial t}\right)_{\bar{\sigma}_v}=0 \tag{9}$$

Considering Eq. (7) and Eq. (9), we have

$$\left(\frac{\partial(\Delta e)}{\partial t}\right)_{\bar{\sigma}_v}=\left(\frac{\partial(\Delta e_s)}{\partial t}\right)_{\bar{\sigma}_v} \tag{10}$$

Substitution of Eq. (7) and Eq. (10) into Eq. (5) leads to

$$d(\Delta e)=\left\{\left(\frac{\partial(\Delta e_p)}{\partial\bar{\sigma}_v}\right)_t+\left(\frac{\partial(\Delta e_s)}{\partial\bar{\sigma}_v}\right)_t\right\}\cdot d\bar{\sigma}_v+\left(\frac{\partial(\Delta e_s)}{\partial t}\right)_{\bar{\sigma}_v}\cdot dt \tag{11}$$

Then, we can transform Eq. (11) into

$$d(\Delta e)=\left(\frac{\partial(\Delta e_p)}{\partial\bar{\sigma}_v}\right)_t\cdot d\bar{\sigma}_v+g'(\bar{\sigma}_v, t) \tag{12}$$

where

$$g'(\bar{\sigma}_v, t)=\left(\frac{\partial(\Delta e_s)}{\partial\bar{\sigma}_v}\right)_t\cdot d\bar{\sigma}_v+\left(\frac{\partial(\Delta e_s)}{\partial t}\right)_{\bar{\sigma}_v}\cdot dt \tag{13}$$

The upper equation apparently equals $d(\Delta e_s)$, so we obtain

$$g'(\bar{\sigma}_v, t)\equiv d(\Delta e_s) \tag{14}$$

Hence, going back to Eq. (3) and using Eq. (14), Eq. (3) is rewritten into

$$\Delta e=\Delta e_p+\Delta e_s=\Delta e_p+\int g'(\bar{\sigma}_v, t) \tag{15}$$

where

$$g'(\bar{\sigma}_v, t)=\left(\frac{\partial(\Delta e_s)}{\partial\bar{\sigma}_v}\right)_t\cdot d\bar{\sigma}_v+\left(\frac{\partial(\Delta e_s)}{\partial t}\right)_{\bar{\sigma}_v}\cdot dt \tag{16}$$

For application of the basic equation to field problems, the concrete expression of each term in Eq. (16) should be determined.

Fig. 4 schematically shows the relation between the summation of the void ratio

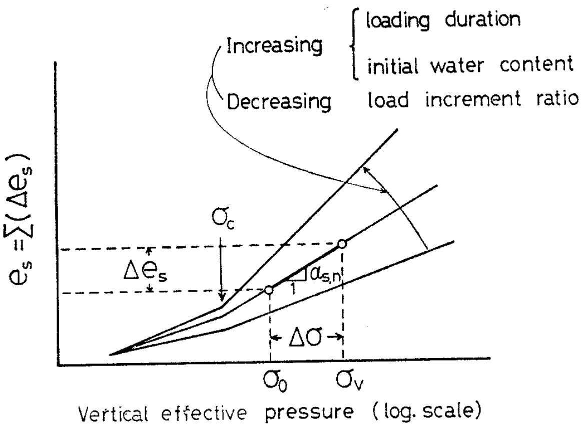

Fig. 4. Relation between the void ratio change due to secondary compression and the effective stress

change and the vertical effective stress in such a thin sample with H^* in height where secondary compression is completely separated from primary consolidation. Considering the linearity of e_s-log $\bar{\sigma}_v$ relation in Fig. 4, the void ratio change Δe_s due to secondary compression taking place during the change of effective stress $\bar{\sigma}_0$ to $\bar{\sigma}_v$ is given by

$$\Delta e_s=\alpha_{s,0}\log\frac{\bar{\sigma}_c}{\bar{\sigma}_0}+\alpha_{s,n}\log\frac{\bar{\sigma}_v}{\bar{\sigma}_c} \qquad (17\,\mathrm{a})$$

(In the case of spanning the critical pressure)

$$\Delta e_s=\alpha_{s,n}\log\frac{\bar{\sigma}_v}{\bar{\sigma}_0} \qquad (17\,\mathrm{b})$$

(In the case of being beyond the critical pressure)

where $\alpha_{s,0}$, $\alpha_{s,n}$ are the slopes of the linear portion before and after the critical pressure. Among them, $\alpha_{s,n}$ is regarded to be much increased with the higher initial water content and the longer loading duration. In the case of neglecting the secondary compression of over-consolidated region, differentiation of Eq. (17 b) gives

$$\frac{\partial(\Delta e_s)}{\partial\bar{\sigma}_v}=0.434\,\alpha_{s,n}\frac{1}{\bar{\sigma}_v} \qquad (18)$$

Owing to the postulate (iv), secondary compression exerted in the normally consolidated region is given by

$$\partial(\Delta e_s)=C_\alpha\cdot\partial(\log t) \qquad (19)$$

or

$$\partial(\Delta e_s)/\partial t=0.434\,C_\alpha(1/t) \qquad (20)$$

It is revealed from Eqs. (18) and (20) that the following relation is obtained as the total differential equation

$$\frac{\partial}{\partial t}\left(\frac{\partial(\Delta e_s)}{\partial\bar{\sigma}_v}\right)_t=\frac{\partial}{\partial\bar{\sigma}_v}\left(\frac{\partial(\Delta e_s)}{\partial t}\right)_{\bar{\sigma}_v}=0 \qquad (21)$$

If this condition is satisfied, Eq. (16) can be easily integrated from $\bar{\sigma}_i$ to $\bar{\sigma}_f$ as the effective stress and from t_0 to t_f as the elapsed time. Then, we have

$$\Delta e_s=\alpha_{s,n}\log\frac{\bar{\sigma}_f}{\bar{\sigma}_i}+C_\alpha\log\frac{t_f}{t_0} \qquad (22)$$

This is the complete expression for secondary compression.

On the other hand, primary consolidation is, as well known, denoted by

$$\Delta e_p=C_c\log\frac{\bar{\sigma}_f}{\bar{\sigma}_i} \qquad (23)$$

By superposing Eq. (22) and Eq. (23), we have

$$\Delta e=\Delta e_p+\Delta e_s$$
$$=(C_c+\alpha_{s,n})\log\frac{\bar{\sigma}_f}{\bar{\sigma}_i}+C_\alpha\log\frac{t_f}{t_0} \qquad (24)$$

A SIMPLIFIED EQUATION FOR ESTIMATING SECONDARY COMPRESSION

For engineering practice, it is essential to determine a coefficient $\alpha_{s,n}$ relating the void ratio change due to secondary compression, Δe_s, to the logarithmic effective stress, which is seen in Fig. 4. Referring to Fig. 4, the amount of secondary compression, Δe_s, exerted in the thick sample is represented by

$$\Delta e_s=\alpha_{s,n}\log\frac{\bar{\sigma}_f}{\bar{\sigma}_i} \qquad (25)$$

In the meantime, Fig. 5 illustrates the void ratio versus the log time relations for the thin and thick samples under the same load increment, $\Delta\sigma_v$. Furthermore, it is apparent that secondary compression in the thin sample (Curve I) from $t_0{}^*$ to t_0 is given by

$$\Delta e_s=C_\alpha\log\frac{t_0}{t_0{}^*} \qquad (26)$$

Since this quantity of secondary compression, Δe_s, is believed to be included in the process of pore-water pressure dissipation in the

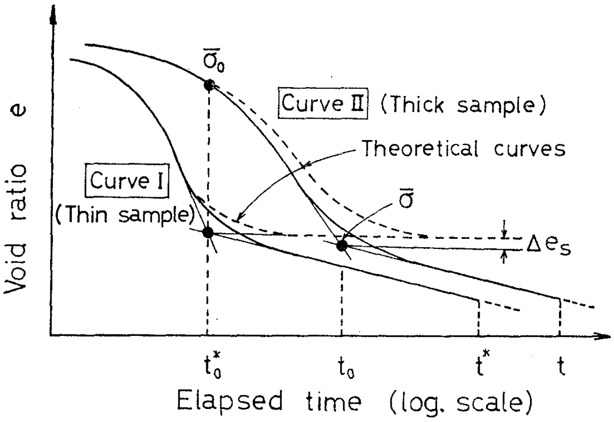

Fig. 5. Schematic diagram for the difference in vertical strain-log time relation between thin sample (curve I) and thick one (curve II)

thick sample, Eq. (26) should be equal to Eq. (25). Hence, we have

$$\alpha_{s,n}\log\frac{\bar{\sigma}_v}{\bar{\sigma}_0}=C_\alpha\log\frac{t_0}{t_0{}^*} \tag{27}$$

Therefore,

$$\alpha_{s,n}=C_\alpha\log\frac{t_0}{t_0{}^*}\Big/\log\frac{\bar{\sigma}_v}{\bar{\sigma}_0} \tag{28}$$

By replacing $C_\alpha/\log(\bar{\sigma}_v/\bar{\sigma}_0)$ as follows

$$\bar{C}_\alpha=C_\alpha/\log\frac{\bar{\sigma}_v}{\bar{\sigma}_0} \tag{29}$$

Eq. (28) is reduced to

$$\alpha_{s,n}=\bar{C}_\alpha\log\frac{t_0}{t_0{}^*} \tag{30}$$

In the meantime, according to the postulate (vi), we obtain

$$\frac{t_0}{t_0{}^*}=\left(\frac{H}{H^*}\right)^2 \tag{31}$$

where H^*: the initial height of the thin sample. Substitution of Eq. (30) and Eq. (31) into Eq. (24) gives

$$\Delta e=\left\{C_c+\bar{C}_\alpha\cdot\log\left(\frac{H}{H^*}\right)^2\right\}\log\frac{\bar{\sigma}_f}{\bar{\sigma}_i}+C_\alpha\cdot\log\frac{t_f}{t_0} \tag{32}$$

The effective vertical stress in Eq. (32) is difined by

$$\bar{\sigma}_f=\bar{\sigma}_i+\Delta\bar{\sigma}_v=\bar{\sigma}_i+\Delta\sigma_v\cdot U_\sigma \tag{33}$$

where U_σ is the degree of consolidation regarding the effective stress. By introducing

$$R=\frac{\bar{C}_\alpha}{C_c} \tag{34}$$

which was defined by Mesri and Godlewski (1977), Eq. (32) is rewritten into

$$\Delta e=C_c\left\{1+R\log\left(\frac{H}{H^*}\right)^2\right\}\log\left(1+\frac{\Delta\sigma_v}{\bar{\sigma}_i}U_\sigma\right)+C_\alpha\log\frac{t_f}{t_0} \tag{35}$$

The degree of consolidation, U_σ, in Eq. (35) is completed by

$$U_\sigma=1-\frac{\int_0^H\left\{1-\left(\frac{\bar{\sigma}_i}{\bar{\sigma}_f}\right)^B\right\}dz}{\int_0^H\left\{1-\left(\frac{\bar{\sigma}_i}{\bar{\sigma}_f}\right)\right\}dz} \tag{36}$$

where

$$B=\sum_{M=0}^{\infty}\frac{2}{M}\sin\left(M\frac{z}{H}\right)\exp(-M^2T_v) \tag{37}$$

$(M=(2m+1)/2,\ m;$ positive integer)

Eq. (36) has been obtained from arrangement of the Davis and Raymond's theory (1965).

From examination of the physical meaning of secondary compression, we can conclude that Eq. (35) resultantly converts the Bjerrum's concept of settlement into the mathematical formulation, that is to say,

i) Instant compression:

$$\varepsilon_i=\frac{C_c}{1+e_0}\log\left(1+\frac{\Delta\sigma_v}{\bar{\sigma}_i}U_\sigma\right) \tag{38}$$

ii) Delayed compression:

$$\varepsilon_d=\frac{\bar{C}_\alpha}{1+e_0}\log\left(\frac{H}{H^*}\right)^2\log\left(1+\frac{\Delta\sigma_v}{\bar{\sigma}_i}U_\sigma\right)+\frac{C_\alpha}{1+e_0}\log\frac{t_f}{t_0} \tag{39}$$

In putting Eq. (35) into practical computation of settlement, it should be kept in mind that we must calculate primary consolidation during pore-water dissipation process and secondary compression during the constant effective stress independently and then superimpose each other as the approximate procedure. That is to say,

In case of $t\leqq t_0$:

$$\Delta e=\left\{C_c+\bar{C}_\alpha\log\left(\frac{H}{H^*}\right)^2\right\}\log\left(1+\frac{\Delta\sigma_v}{\bar{\sigma}_v}U_\sigma\right) \tag{40}$$

In case of $t>t_0$:

$$\Delta e=\left\{C_c+\bar{C}_\alpha\log\left(\frac{H}{H^*}\right)^2\right\}\log\left(1+\frac{\Delta\sigma_v}{\bar{\sigma}_v}V_\sigma\right)$$

$$+\bar{C}_\alpha \log \frac{t_f}{t_0} \tag{41}$$

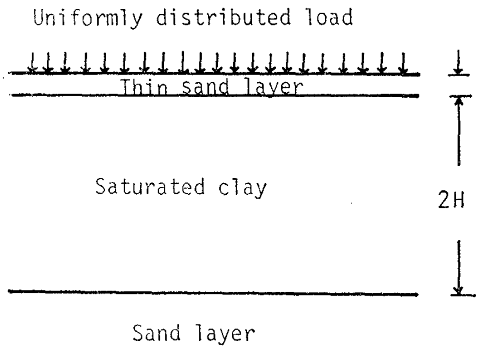

Fig. 6. Imaginary ground for calculation of consolidation

EXAMPLE OF CALCULATION

The influence of several factors on settlement-time curves of the imaginary ground is numerically examined using the method proposed for estimating secondary compression. Calculated settlement-time curves are compared with observed curves in a series of consolidation tests concerning the scale effect of clay carried out by Aboshi, which are most reliable so far.

Example 1: An imaginary ground as shown in Fig. 6 whose upper and lower boundaries are sandwiched by permeable layers is assumed for settlement calculation. Material constants are the height in clay layer H, the compression index C_c, the coefficient of

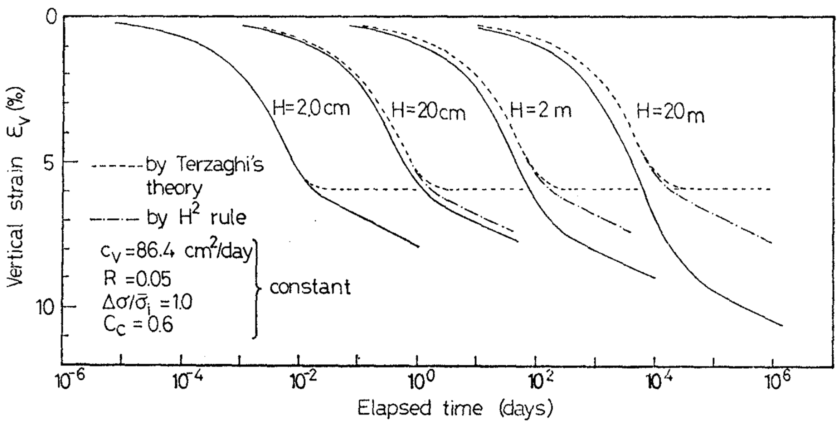

Fig. 7. Vertical strain versus elapsed time relations in consolidation of an imaginary soft ground accompanied by change of thickness of soil layer

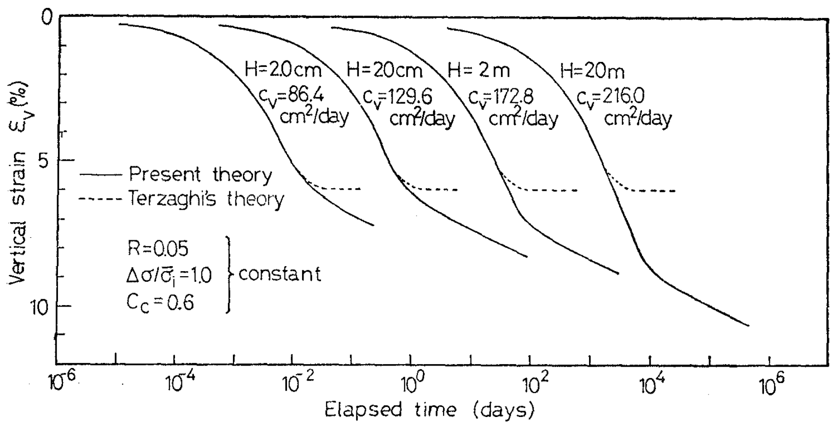

Fig. 8. Vertical strain versus elapsed time relations in consolidation of an imaginary ground in the case of changing c_v-value with thickness of soil layer

secondary compression C_α, and the coefficient of consolidation c_v. The stress condition is represented by the load increment ratio, $\Delta\sigma/\sigma_i$.

The first investigation is concerned with the influence of the height of the clay layer on the settlement-time curve. Keeping other parameter C_c, R $(=C_\alpha/C_c)$, c_v and $\Delta\sigma/\sigma_i$ constant, the height of clay layer is varied with 2.0, 20, 200 and 2 000 cm, respectively. Calculated results are plotted in Fig. 7. It is remarkably clear that the vertical strain at the 100% primary consolidation tends to be somewhat increased with the increase of the thickness of clay layer. For instance, the difference of 100% primary consolidation between the thickness of 2.0 m and 20 m differes by no more than 2% in strain. However, this will be a significant difference of settlement, especially in thick clay layers.

It is frequently pointed out that the c_v-value observed in the field has a tendency to be larger than in the laboratory. This fact was also verified by observations of Aboshi. Calculated curves in the case of allowing the c_v-value to change with the thickness of layer are shown in Fig. 8. The c_v-values are common with the curves which were computed by the present theory (full line) and the Terzaghi's one (dashed line). Time-settlement relations due to the pure primary consolidation are also plotted in Fig. 7 and Fig. 8. The outstanding difference between the calculations by the conventional and the proposed methods is recognized in a family of curves in both figures.

From an another point of view, the influence of the coefficient of consolidation on the settlement-time curves is then examined. Consider a clay layer with a height of 20 m. Other parameters are assumed to remain constant. They are of the same values as the ones adopted in Fig. 7 and Fig. 8 and only the c_v-value is taken to be variable, depending on the height of clay layer, from 43.2 cm^2/d to 12 960 cm^2/d. The calculated settlement is plotted against the elapsed time as a parameter of c_v in Fig. 9. We can understand from Fig. 9 that there might be a large amount of the difference in settlement if we failed to select the c_v-value. Suppose, for instance, c_v is of the constant value, 43.2 cm^2/d, throughout the different height of the specimen. The vertical strain at 100% primary consolidation, ε_{100}, is more delayed by 1% than that from curves at the same elapsed time when c_v is assumed to be thirty times as 43.2 cm^2/d. This inclination suggests that we should develop the method for modification of c_v oedometer tests in order to correspond with the height of the clay layer at the field.

Example 2: A family of settlement-time

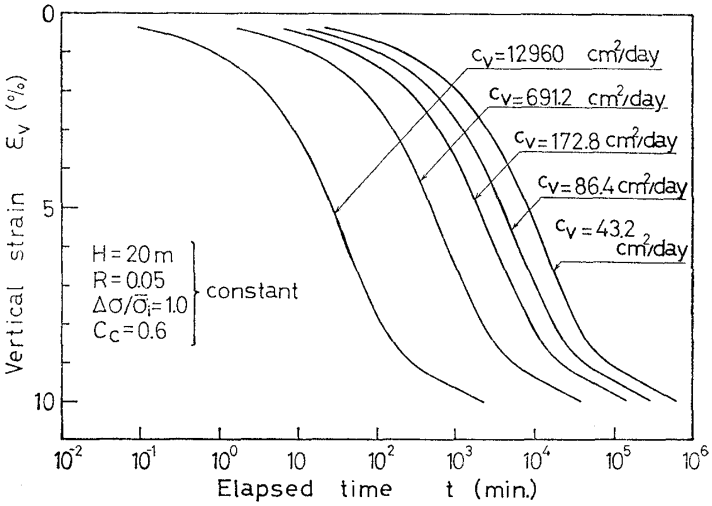

Fig. 9. Influence of the coefficient of consolidation on vertical strain versus elapsed time relations

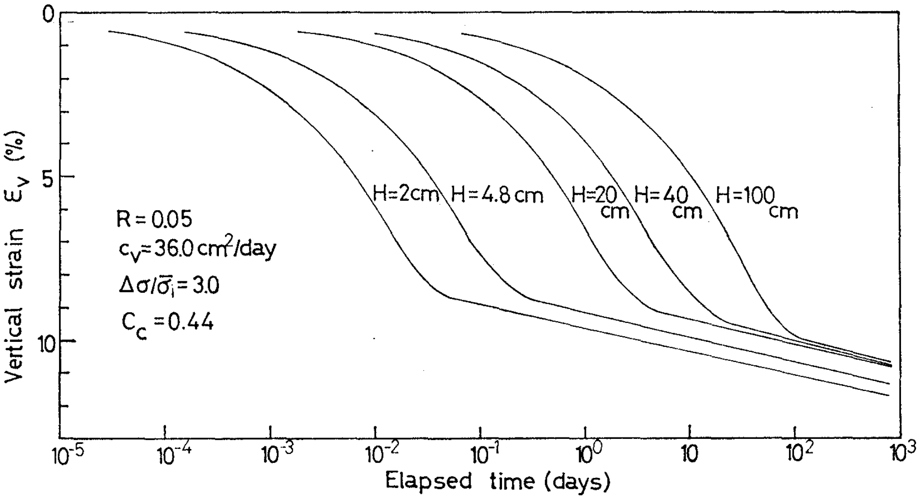

Fig. 10. Calculated vertical strain-consolidation time curves for comparison with observations by Aboshi

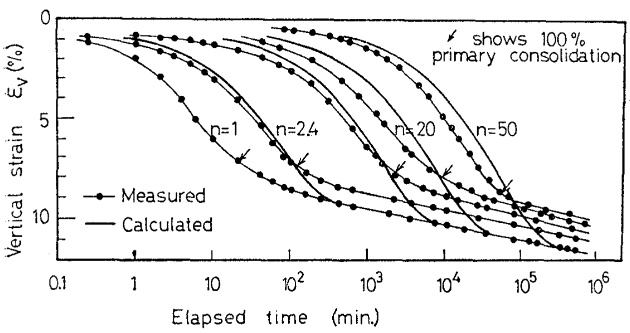

Fig. 11. Scale effect in consolidation of silty clay (Aboshi : 1973)

Table 1. Index properties of a silty clay used for experiments by Aboshi (1973)

Clay (%)	Silt (%)	Sand (%)	L. L. (%)	P. I.	G_s	w_n (%)	q_u (kPa)
27	68	5	100.0	42.0	2.65	80.0	150

curves in Fig. 10 are assessed by the proposed method to compare with the series of experiments by Aboshi on the scale effect of one-dimensional consolidation as shown in Fig. 11. Index properties given at the reference are tabulated in Table 1. Other parameters were deduced by those indices in order to be consistent with the average values of the typical marine clay in Japan. Among them, it should be noted that c_v is kept constant because no reliable method to modify the c_v-value in oedometer tests in accordance with the specimen height is available.

Comparison of Fig. 11 with Fig. 10 shows that strain versus log time plots have the same tendency each other. The vertical strain at 100% primary consolidation increases with the specimen height as well as in the Example 1. Linear portions in each

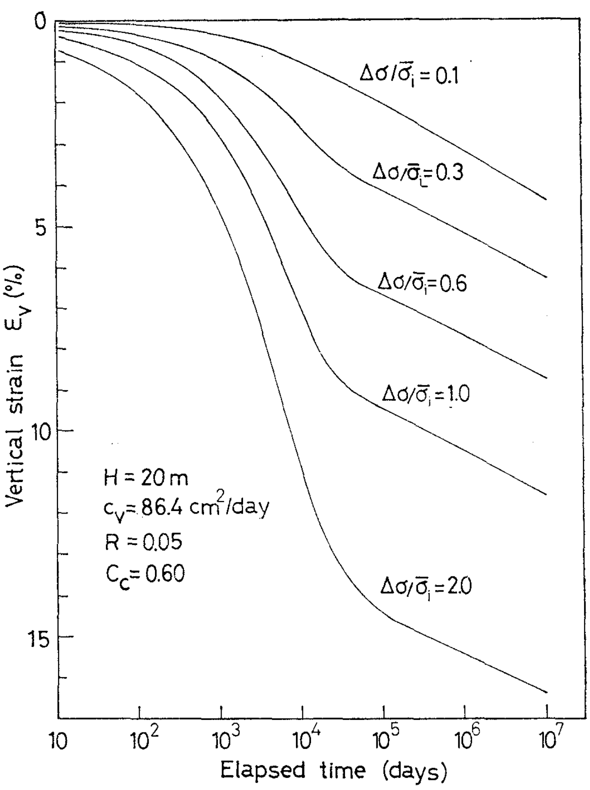

Fig. 12. Influence of the load increment ratio on vertical strain versus elapsed time relations in consolidation of an imaginary ground

Table 2. Parameters used for computations (I)

H(cm)	e_0	$C_c/(1+e_0)$	$\Delta\sigma_v/\bar{\sigma}_i$	$\bar{C}_\alpha/(1+e_0)$
2	2.42	25.15	5.56	0.0125
20	2.40	25.29	5.05	0.0125
200	2.17	27.14	2.82	0.0125
2 000	1.54	29.10	0.518	0.0125

Table 3. Parameters used for computations (II)

H(cm)	e_0	$C_c(1+e_0)$	$\Delta\sigma_v/\bar{\sigma}_i$	$\bar{C}_\alpha/(1+e_0)$
2	2.42	25.15	1.0	0.0125
20	2.40	25.29	1.0	0.0125
200	2.17	27.14	1.0	0.0125
2 000	1.54	29.10	1.0	0.0125

curve of Fig. 10, which are prescribed secondary compression, do not coincide on the single line as indicated by the isotache method of Suklje (1957). These tendencies are of the good agreement with the experimental observations by Aboshi.

Example 3: In general, settlement-time curves in the one-dimensional consolidation are significantly affected by the load increment ratio. Barden (1965) and Leonards et al. (1961) provided the representative test results of the different patterns of settlement-time curves depending on the load increment ratio. Calculations with c_v, C_c, R and H which are kept constant for variations of $\Delta\sigma/\sigma_i$ are performed to obtain Fig. 12. The inflection point in every curve in Fig. 12 with the small load increment ratio is not clearly recognized. This tendency was experimentally confirmed by Leonards et al. (1961) and Barden (1965) as described before.

The same imaginary ground as shown in Fig. 6 which consists of the homogeneous soft soil is used again. The index properties and the mechanical constants necessary for settlement computation are listed in Table 2. The computed settlement-time curves in connection with the following two cases are demonstrated.

Example 4: The vertical strain versus the log time relation in the case of keeping C_α constant, and $C_c/(1+e_0)$ and H changeable is given in Fig. 13. It is assumed herein

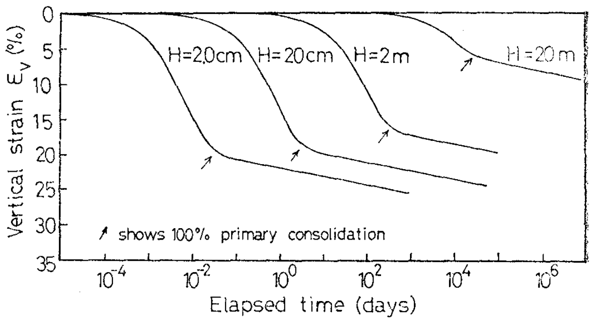

Fig. 13. Calculated vertical strain versus elapsed time relations in the case of keeping C_α constant, and $C_c/(1+e_0)$ and H variable

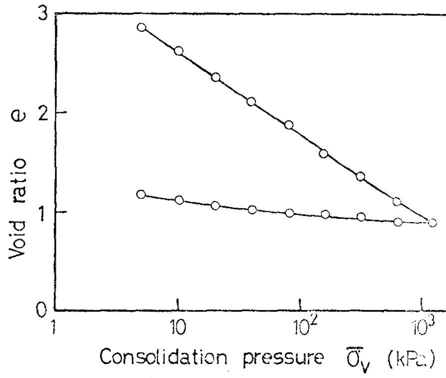

Fig. 14. Representative e-log σ_v relation of saturated clay constituting the imaginary soft ground

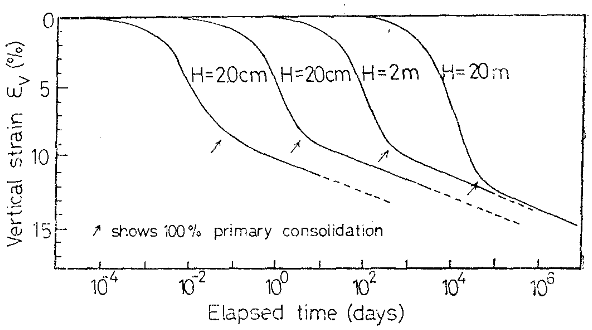

Fig. 15. Calculated vertical strain versus elapsed time relations in the case of keeping C_α, $C_c/(1+e_0)$ and H variable and keeping C_α and $\Delta\sigma/\bar{\sigma}_i$ constant

that the consolidation of an imaginary ground in Fig. 6 follows a single e-log σ_v relation as shown in Fig. 14. Thus, each initial void ratio corresponding to the vertical stress equivalent to the self-weight in each height of soil layer is read off from Fig. 14, and the value of $C_c/(1+e_0)$ at each layer is determined. Fig. 13 shows the decreasing tendency of the vertical strain at 100% primary consolidation accompanied by the increase in thickness of clay layer. This is caused by the drastic decrease in the load increment ratio with the depth of subsoil in the imaginary ground.

Example 5: The results computed by the modified equation are plotted in Fig. 15. In this case, both $C_c/(1+e_0)$ and $\alpha_{s.n}$ are also changed in accordance with the increased thickness of clay layer. The condition for computation is summarized in Table 3. Fig. 15 indicates that we have a little increase in the amount of vertical strain corresponding to the thickness of soil layer. It is understood that, considering the characteristics of Fig. 14, this behavior will be caused by the increase of secondary compression contained in the process of pore-water pressure dissipation. This tendency also bears a good resemblance to the settlement behavior presented by Aboshi, as well as in Example 2. Eventually, strain-time performance in Fig. 15 suggests the availability of the present theory to settlement prediction containing secondary compression.

CONCLUSION

A practical model for analyzing the one-dimensional settlement of saturated cohesive soils is presented. The proposed method is characterized by the feature of attempting to explain the time-settlement performance by taking the solution of the non-linear one-dimensional consolidation into the stress-strain relation.

Investigation of the influencing factors on the settlement-time curves for imaginary ground was carried out. Besides, the availability of the proposed method was confirmed by comparison with laboratory settlement records obtained by Aboshi. Following conclusions have been drawn from these considerations.

(1) The practical model proposed herein can explain the behavior that the vertical strain at 100% primary consolidation increases somewhat with the increase of the thickness of soil layer. In other words, calculated strain-time curves have the good similarity with patterns of the settlement-time curves performed by Aboshi which is concerned with the scale effect of one-dimensional consolidation.

(2) The present method can explain the experimental fact that the shapes of the settlement-time relations are varied by the load increment ratio.

(3) The one-dimensional stress-strain-time relation is conceptually equivalent to the definition of settlement of Drammen Clay in Norway proposed by Bjerrum which labels the instant compression and the delayed compression, respectively.

NOTATION

$B=\sum_{M=0}^{\infty}\frac{2}{M}\sin\left(M\frac{z}{H}\right)\exp(-M^2T_v)$

C_c=compression index

C_α=coefficient of secondary compression

$\bar{C}_\alpha$=coefficient of secondary compression where load increment ratio is 10

c_v=coefficient of consolidation

e=void ratio

e_0=initial void ratio

Δe=void ratio increment

Δe_p=void ratio increment due to primary consolidation

Δe_s=void ratio increment due to secondary compression

H=height of soil layer

H^*=height in thin soil layer

$M=(2m+1)/2$

m=positive intergral figure

$R=C_\alpha/C_c$

t=time

t_0=time at end of primary consolidation

t_f=time at end of secondary compression

t_0^*=time at end of primary consolidation for the thin sample

T_v=time factor

U_σ=degree of consolidation regarding the effective stress

z=distance from the surface of soil layer

$\alpha_{s,n}$=coefficient relating secondary compression to effective stress in the region of normally-consolidation

$\alpha_{s,0}$=coefficient relating secondary compression to effective stress in the region of over-consolidation

ε_i=vertical strain due to instant compression

e_d=vertical strain due to delayed compression

ε_v=vertical strain

ε_{100}=strain at end of primary consolidation

$\Delta\sigma$=stress increment

$\Delta\sigma_v$=vertical stress increment

$\Delta\bar{\sigma}_v$=vertical effective stress increment

$\bar{\sigma}_v$=effective stress, vertical effective stress

σ_c=critical pressure

$\bar{\sigma}_i$=initial effective stress

$\bar{\sigma}_0$=effective stress in the region of over-consolidation

$\bar{\sigma}_f$=final effective stress

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