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Consolidation Behaviors of Compacted Soils Using a Large Scale Oedometer

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The paper describes on the results of investigation for consolidation behaviors of a compacted soil conducted by using a large scale oedometer (the size of the ring is 30 cm in diameter and 10 cm in height). As compared with the results obtained by the large scale oedometer and the standard one (6 cm in diameter, 2 cm in height), the auther found that both gave identical values in the volume compressibility of consolidation m_v, however, there were some considarable differences in the coefficient of consolidation c_v.

These phenomena can to some extent be explained quantitatively applying the dimensional analysis (Fig. 3, 4 and Eq.(2)).

Settlement versus time elapse relationship for a compacted soil is that the degree of immediate settlement is large. This is represented by two curves, which are due to the immediate settlement and the consolidation settlement, using a numerical calculation method (Fig. 5, 6). Besides, distribution of the pore pressure during the consolidaton test was measured by a small pressure meter which was placed at the center of the specimen.

As compared with the values calculated by Terzaghi's one dimensional theory of consolidaton, the primary consolidation settlement for a compacted soil was found to be the validity of Terzaghi's theory on the saturated soil (Fig. 7, 8, 9 and Eq.(11)).

Key words: compression, consolidation test, pore pressure, compaction, settlement

1. Introduction

The consolidation test is a model test in which a specimen of soil is subjected to pressure in order to predict the deformation that would occur to a stratum of soil under similar pressure in the field[1]. The success of the test depends on how well the model test represents the situation in nature. For the case of relatively hetrogeneous soils of compaction, it may be desirable to perform laboratory tests using the large scale sample[2]. This paper presents some examples of the behavior of the consolidation of the compacted soil by the large scale oedometer with specimens about 10 cm in height and 30 cm in diameter.

The purpose in this paper is to determine the difference of consolidation parameters measured in the large scale oedometer and the standard one (2 cm-height and 6 cm-diameter specimens) when used the fine-graded soil as the embankment materials, and to develope a fitting method of calculation for time-settlement relationships obtained in the large scale consolidation tests.

The second purpose of this study is described in which an attempt is made to verify the theoretical distribution of the pore pressure during one-dimensional consolidation tests in the compacted soil by measuring the pore pressure at each side of the ring across diameter during tests.

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2. Description of Apparatus and Testing Procedures

Fig. 1 is a schematic diagram of the large scale oedometer apparatus. The size of the ring is 30 cm in inside diameter, about 15 cm in height.

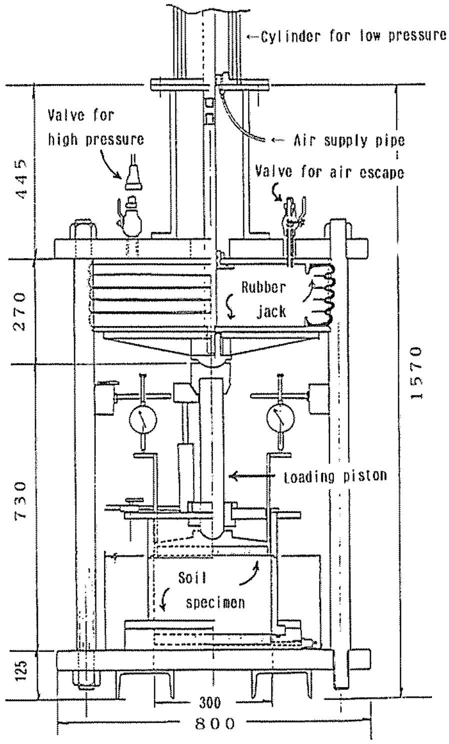

Fig. 1 Schematic diagram of large-scale oedometer (Dimensions in mm)

The consolidation pressures applied to the specimen during testing are provided by the rubber jack being filled with air through the air compresser. The capacity has been designed for the maximum pressure of 50 kgf/cm^2. The pressure can be maintained for any desired values using the pneumatic regulators. In order to counter balance self-weight of the loading piston and the upper cap, zero balance adjuster is mounted outside the ring. Throughout the test, the consolidation settlement of the specimen is measured with dialgages (1 unit = 1/10 mm). After testing has been achieved the specimen is pushed out the ring turning upside down.

The specimens are formed with vibrating compactor in the consolidation ring in five lift of approximately equal thickness. Water contents and density of the tested soil are depended upon the results of compaction test. In addition, a pore pressure transducer is buried in the middle of the specimen. The pressuring cacity is 10 kgf/cm^2 and the diameter of the pressuring plane is 6 mm. The results of measuring the pore pressure are recorded automatically by a digital strain meter.

Testing procedure is based on the method of the standard consolidation test through-out.

3. Soil Description

Physical properties of the soil used for this study are shown in Table 1.

The soil was prepared by sieving through a No. 4 sieve (Maximum grain size is 4.76 mm). The relationships between dry density and water content are given in Fig. 2.

Specification for compaction test is that volume of mold is 2208 cm^3, height of rammer drop 45 cm, numbers of layers 5, weight of rammer 4.5 kgf, number of blows per layer 5 and compaction energy 25.3 kgf cm/cm^3 [3]. Main purpose adopted this specification is to prepare for the study of the soil with gravell-sand mixture, in future.

Each samples assigned a name based on a compaction curve. D100 sample means that the ratio of a given dry density to the maximum dry density is 1. Three different types of the compaction soil, that is, D100, D90, and D80 are used in the test. Degree of saturation of D100, D90 and D80 sample is 81.9%, 91.6% and 93%, respectively. Dry density ρ_d versus water content w of each sample is shown in Fig. 2.

Table 1 Physical properties of soil used

Specific gravity G_s	2.671	Less than 5 μ clay fraction	12.1%
Liquid limit W_L	31.6%	Uniformity coefficient U_c	48.2
Plastic limit W_p	25.9%	Optimum water content W_{opt}	14.0%
Plasticity index I_p	5.7%		
2~4.76 mm gravel fraction	3.0%	Max. dry density ρ_{dmax}	1.828 gf/cm^2
0.074~2 mm sand fraction	52.7%	Japanese unified soil classifica-tion	SM
5~74 μ silt fraction	32.2%		

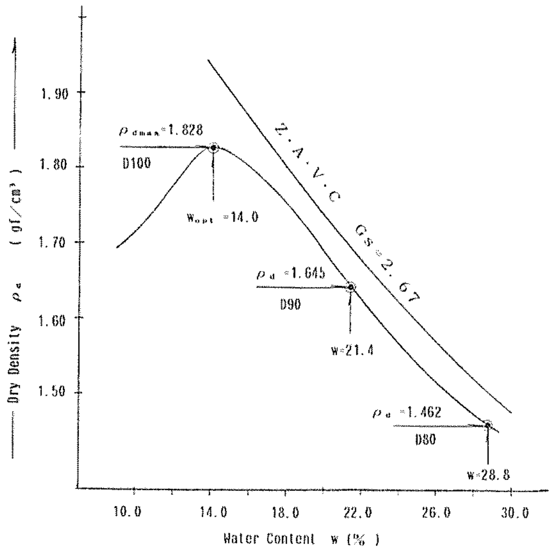

Fig. 2 Compaction curve of tested soil and description of the experimental points

4. Experimental Results and Discussion

4-1. *Comparison of experimental results by the standard consolidation test and the large scale one*

Coefficient of consolidation c_v versus average consolisation pressure $\bar{p}$ relationships for the standard and the large scale consolidation test are presented in Fig. 3, and coefficient of volume compressibility m_v versus $\bar{p}$ relationships in Fig. 4.

In the c_v-$\bar{p}$ relationships it can be seen that the c_v values of the large scale test are very different from that of the standard test, and the former is considerably larger than the latter. With increasing degree of compaction, the pattern of the values of c_v becomes increasingly large for the large scale test, especially it seems to be remarkable in the lower consolidation pressure range.

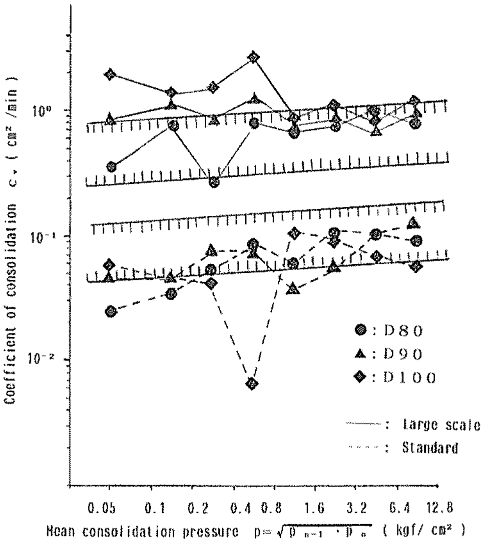

Fig. 3 Relations between the mean consolidation load and the coefficient of consolidation

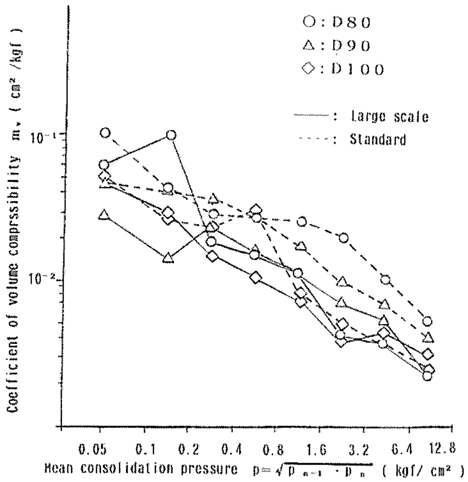

Fig. 4 Relations between the mean consolidation load and the coefficient of volume compressibility

The m_v-$\bar{p}$ rerationships indicate that values of m_v decrease gradually as the $\bar{p}$ increase. However, there is no significant difference between the standard test and the large scale test.

Owing to qualitative explanation for these tendencies the auther examined the difference of the c_v and m_v values in both tests using the similarity rule.

L and T denote length and time, and subscripts s and l indicate the standard test and the large scale test, respectively.

Then, the ratio of size between these two is as follows.

$$\eta=\frac{L_s}{L_l}=\frac{6}{30}=\frac{1}{5}$$

In the permeability of the soil k is assumed to be independant of the size of test apparatus, then:

$$\frac{k_s}{k_l}=\frac{L_s}{T_s}\cdot\frac{T_l}{L_l}=1 \qquad \therefore\ \frac{T_s}{T_l}=\eta \qquad \cdots(1)$$

Let c_v^s and c_v^l be the coefficient of consolidation by the standard and the large scale oedometer ring, respectively, a relation is given as:

$$\frac{c_v^s}{c_v^l}=\frac{L_s^2}{T_s}\cdot\frac{T_l}{L_l^2}=\eta^2\cdot\frac{1}{\eta}=\eta \qquad \therefore\ c_v^l=\frac{c_v^s}{\eta} \qquad \cdots(2)$$

It should be noted that the value of the c_v^l shows $\frac{1}{\eta}$ times, that is, 5 times as large as the value of the c_v^s. The result of this calculation agrees with increasing of the c_v value with increasing the height of the specimens presented by other investigations[4),5)], and it corresponds roughly with the experimental results as seen in Fig. 3. In general, however, the relation between the height of the specimen and the c_v value can not be represented quantitatively on a satisfactory theoretical basis . Fig. 3 also shows the relation between the experimental values and the results of calculation according to Eq. (2) for each test, showing the scalle effects of the specimen clearly.

On the other hand, the coefficient of volume compressibility m_v is defined:

$$m_v=\frac{d\varepsilon}{dp'}$$

where $d\varepsilon$ is change in strain and dp' is change in effective consolidation pressure.

$d\varepsilon$ is dimensionless and $dp'=1$ is common to both tests because of adopting the same ratio of load increment. Therefore, the values of m_v are independant of the effect of size of the ring used. While they are somewhat scatterd they are generally consistent with the results of the experiments summarized in Fig. 4.

As explained later in this paper, the values of the c_v used here were determined by fitting method of calculation for the time-consolidation settlement relationships.

4-2. *The laboratory time-consolidation settlement relationships*

(1) *A calculation method of the coefficient of consolidation c_v*

Graphical procedures have been used to obtain the c_v values from the oedometer tests, such as, $\sqrt{t}$ method[6] and curve rule method[7]. In the case of the compaction soil, however, these methods are sometimes useless because of increase in the immediate settlement and the scatter of overall data.

The auther has proposed a fitting method of calculation as an alternative method. The outline will be described below. Detail is shown in reference[8].

Average degree of consolidation U in one-dimensional consolidation is given as[9]:

$$U=1-\sum_{m=0}^{\infty}\frac{2}{M^2}\exp(-M^2\cdot T_v) \qquad \cdots(3)$$

where,

$$M=\frac{1}{2}(2m+1)\pi,\ m=0,\ 1,\ 2,\ \cdots,$$

$T_v=c_v\cdot t/H^2$: *time factor*,
c_v : *coefficient of consolidation*,
H : *one-half of height of the specimen.*

Putting $c_v/H^2=\lambda$, the first approximation λ is derived from a equation including the terms up to the third order of the infinite series of Eq. (3).

$$d=A-B\left(a^{\lambda t}+\frac{1}{9}b^{\lambda t}+\frac{1}{25}c^{\lambda t}\right) \qquad \cdots(4)$$

where A and B is const. and

$$a=\exp\left\{-\left(\frac{\pi}{2}\right)^2\right\},\ b=\exp\left\{-\left(\frac{3\pi}{2}\right)^2\right\},\ c=\exp\left\{-\left(\frac{5\pi}{2}\right)^2\right\}$$

Now, finding time t in terms of arithmetic sequence of numbers to common difference h, then,

$$t=t_0+(\nu-1)\cdot h, \quad \nu=1,2,3,\cdots$$

where, t_0 is a initial value of t

Let d_ν and $d_{\nu+1}$ denote the consolidaton settlement for $\nu=\nu$, $\nu+1$, using the relation:

$$d_{\nu+1}-d_\nu=y_{\nu+\frac{1}{2}}$$

and denoting:

$$a^{\lambda h}=z_1, \quad b^{\lambda h}=z_2, \quad c^{\lambda h}=z_3 \qquad \cdots(5)$$

An equation is obtained from four equations in respect of $y_{\nu+\frac{1}{2}}$, $y_{\nu+\frac{3}{2}}$, $y_{\nu+\frac{5}{2}}$ and $y_{\nu+\frac{7}{2}}$,

$$z^3+\alpha\cdot z^2+\beta\cdot z+\gamma=0 \qquad \cdots(6)$$

where,

$$-(z_1+z_2+z_3)=\alpha$$
$$z_1\cdot z_2+z_2\cdot z_3+z_3\cdot z_1=\beta$$
$$-z_1\cdot z_2\cdot z_3=\gamma$$

Therefore,

$$y_{\nu+\frac{1}{2}}\cdot\gamma+y_{\nu+\frac{3}{2}}\cdot\beta+y_{\nu+\frac{5}{2}}\cdot\alpha+y_{\nu+\frac{7}{2}}=0 \qquad \cdots(7)$$

is derived.

Forming a normal equation of Eq.(7), values of α, β and γ are determined. By the solution and Eq.(6), the first approximation of λ is obtained from Eq.(5). In order that the λ values are suitable, the condition have to be satisfied.

$$z>0 \ : \ ln(z)\leqq 0$$

In order to obtain the more accurate value of λ, one makes the value change in the vicinity of the first approximation value using the equation as follow and finds out a unique λ value with iterative calculation where the total summation of squares of the residual difference becomes to a minimum in the least square method.

$$d=A-B\cdot\Phi(\lambda\cdot t) \qquad \cdots(8)$$

where,

$$\Phi(\lambda t)=\sum_{m=0}^{\infty}\frac{1}{M^2}\exp(-M^2\cdot\lambda\cdot t)$$

A and B are const.

so that,

$t=0 \ : \ d_0=A-\pi^2 \cdot B/8$

$t=\infty \ : \ d_{100}=A$

$c_v=\lambda \cdot H^2$

A great advantage of this tequnique is that artifitial errors can be reduced to compared with the conventional graphic method, such as, the curve rule method. The auther believes that this method is valid for not only determining the estimation of data of the laboratory test but also analyzing the observed time-settlement relationships in the field.

(2) *Time-consolidation settlement of the compacted soil*

Settlement S of the compacted soil is genellaly presented by the sum of three components, that is, the immediate settlement (or instantaneous compression) S_i, the consolidation settlement S_c with pore water and air drainage, and the elapsed creep deformation after drainage, so called, the secondary compression S_s[10].

$$S=S_i+S_c+S_s$$

So that time-settlement relations in the compacted soil indicate complicated shaped curves consisting of three components in which the portion of S_i is predominated.

It can be considered to follow Terzaghi's theory for the portion of S_c since the fitting method of numerical calculation priscribed above is applied well.

The portion of S_i is considered as elastic deformation with a little delay of time (about 1 minute in this experiment). To simplify the analysis of experimental data an attempt was made to adopt the fitting method of calculation for the portion of the immediate settlement S_i. Examples of the result are shown in Fig. 5 (in the case of p=0.2, 0.4 kgf/cm²), and Fig. 6 (p=0.8, 1.6 kgf/cm²) for D80 sample.

From these illustrations, we can recognize that the above method is capable of fitting well to not only the portion of the consolidation settlement S_c but also the potion of the immediate settlement S_i. However, it is

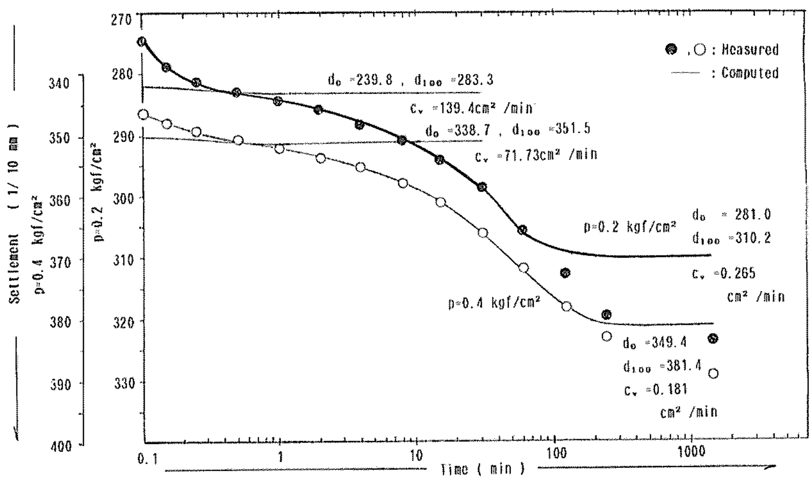

Fig. 5 Comparison of measered and computed relationship for settlement versus time (D80 specimen, in the case of p=0.2, 0.4 kgf/cm²)

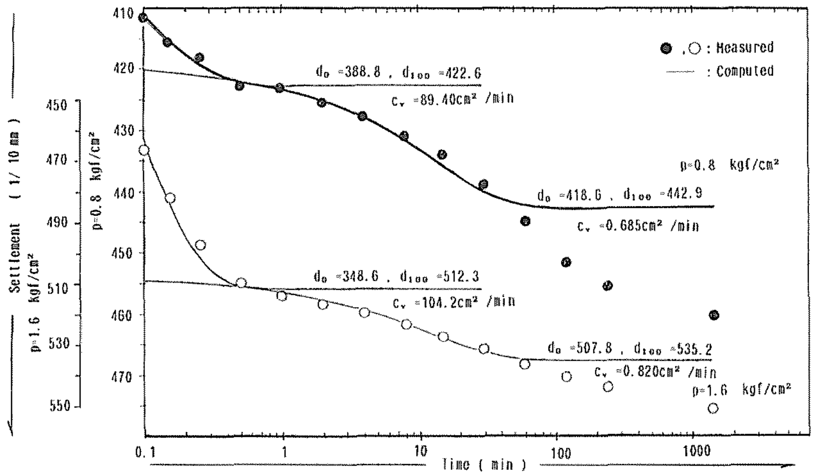

Fig. 6 Comparison of measured and computed relationship for settlement versus time (D80 specimen, in the case of p=0.8, 1.6 kgf/cm^2)

difficult to evaluate the mechanical significance of the c_v values obtained here. The problem needs to be applied differently, using procedures outside the scope of this study.

If each portion of S_i and S_c can be approximated by two curves, the degree of consolidation is written as follows:

$$\left.\begin{aligned} 0 \leqq t \leqq t_0 \quad &: \quad U_i = 1 - \exp\left(-M^2 \frac{c_{v1}}{H^2} t\right) \\ t \geqq t_0 \quad &: \quad U_c = 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} \cdot \exp\left\{-M^2 \frac{c_{v1}}{H^2} t_0 - M^2 \frac{c_v}{H^2}(t - t_0)\right\} \end{aligned}\right\} \cdots (9)$$

where, t_0 is time-dependent boundary between S_i and S_c,

U_i is the degree of consolidation of S_i, and c_{v1} its coefficient of consolidation,

U_c is the degree of consolidation of S_c.

From Fig.5 and Fig. 6, it can be seen that the range of the S_i portion becomes larger with the increase of consolidation pressure and corresponding to it the value of c_{v1} also becomes larger.

(3) *Distribution of the pore pressure in the specimen*

In measuring the pore prssure in the specimen during the consolidation test, the technique which places the pressure meter at the bottom of the consolidation cell, has been used in ealier studies. However, the rigidity of the pressure plane has great influence on the values of the observed pressure[11),12)]. In this study a small pressure meter mentioned above was installed at the center of the specimen, and inner pore pressure was measured. The auther recognizes that such treatment is a main advantage using the large scale oedometer.

Under these circumstances it seems reasonable to correct the measured value as there exists the time-lag between loading of the consolidation pressure and generation of the pore pressure owing to the heigh of the specimens (the greatest length of drainage is about 5 cm).

An equation of distribution of the pore pressure by one dimensional consolidation theory is given:

$$u=p\sum_{m=0}^{\infty}\frac{2}{M}\exp(-M^2\cdot\lambda\cdot t)\cdot\sin\left(\frac{M}{H}z\right) \qquad \cdots(10)$$

where, p : consolidation pressure, z: coordinate in vertical direction

t_H denotes the time up to the maximum pore pressure after loading. Then, a next equation is assumed:

$$p=\alpha\cdot\tau$$

where τ is time-dependent coordinate from 0 to t_H

α is const.

Putting z=H in Eq.(10) and using a relation $dp=\alpha\cdot d\tau$[13], then

$$u=\sum_{m=0}^{\infty}\frac{2}{M^2}(-1)^m\int_0^{t_H}\frac{dp}{d\tau}\exp\left\{-M^2\cdot\lambda\cdot(t-\tau)\right\}d\tau$$
$$=\frac{2\alpha}{\lambda}\sum_{m=0}^{\infty}\frac{(-1)^m}{M^3}\left[\exp\left\{-M^2\cdot\lambda\cdot(t-t_H)\right\}-\exp(-M^2\cdot\lambda\cdot t)\right] \qquad \cdots(11)$$

is obtained.

Fig. 7, Fig. 8 and Fig. 9 indicate the measured and calculated values of the pore pressure in contrast with the time-consolidation settlement relationships for D80 sample.

The consolidation pressure is not equal to the maximum pore pressure because of the unsaturation of soils tested. To simplify computation the ratio of the maximum pore pressure by measurement to the consolidation pressure is contained in the α value of Eq.(11).

In the case of p=1.6 kgf/cm^2 in Fig. 7, poor correlation was obtained for the measured pore pressure values versus the values computed from the Eq.(11). From this reasoning, the auther presented the distribution of

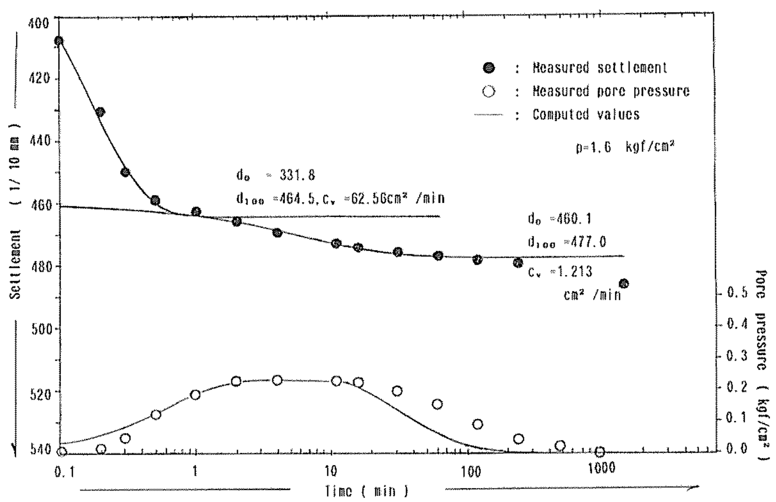

Fig. 7 Comparison between the settlement and the development of pore pressure (in the case of p=1.6 kgf/cm^2, 1 kgf/cm^2=98.1 KN/m^2)

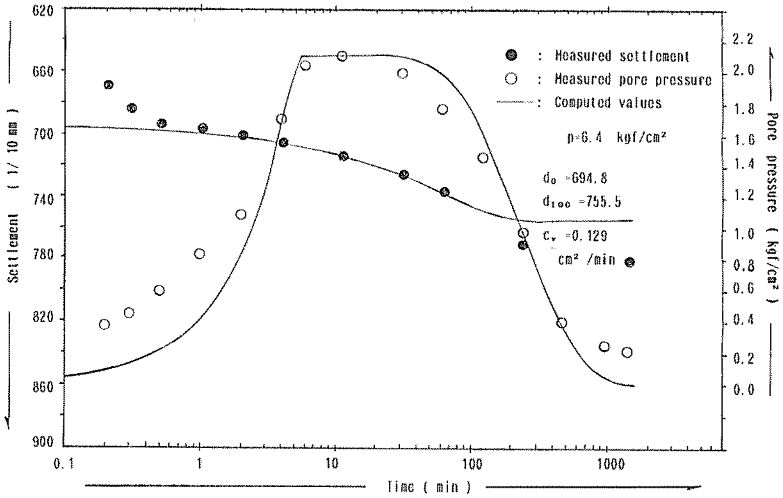

Fig. 8 Comparison between the settlement and the development of pore pressure (in the case of p=6.4 kgf/cm^2, 1 kgf/cm^2=98.1 KN/m^2)

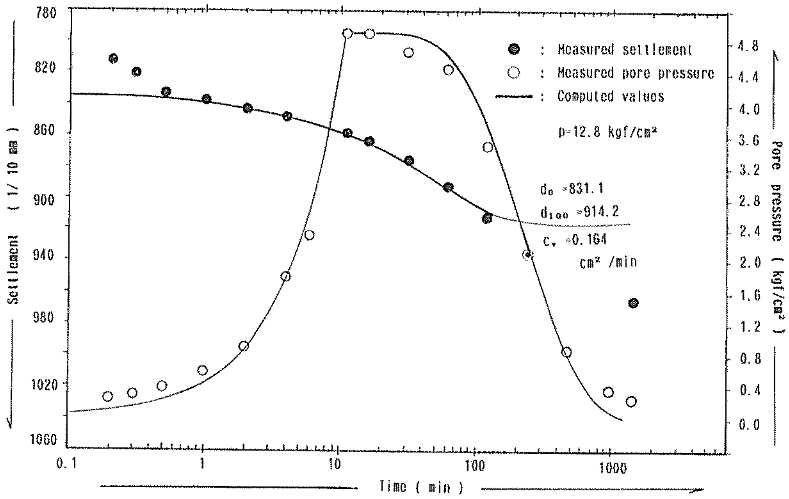

Fig. 9 Comparison between the settlement and the development of pore pressure (in the case of p=12.8 kgf/cm^2, 1 kgf/cm^2=98.1 KN/m^2)

the pore pressure as a consecutive curve for two parts consisting of the coefficient of consolidation c_{v1} in the portion of S_i and the c_v in that of S_c. They are indicated in this figure with the solid line. Even for such treatment, it can be seen that the calculated curve does not strictly correspond to the measured values, and the case p which is less than 1.6 kgf/cm^2 was the same pattern as this.

Contrastively, in the case of p=6.4 kgf/cm^2 in Fig. 8 and p=12.8 kgf/cm^2 in Fig. 9, the calculated values correspond fairly well with the measured values. These results show that the calculated pore pressure curves using the c_v value in the portion of S_c are capable of estimating the measured values successfully except for the portion of the immediate settlement S_i and the secondary compression S_s.

Thus, it can be seen that the portion of consolidation settlement S_c in the compacted soil also follows one dimensional theory of saturation to some extent.

A further improvement is possible in the teqnique in measuring the pore pressure of the compacted soil, especialy in the region of the low consolidation pressure.

5. Conclutions

1. For each of samples there was no significant difference between the values of volume compressibility of consolidation m_v obtained from the standard test and the large scale one. Whereas, results showed that the values of coefficient of consolidation c_v are greater by the large scale test than by the standard test.
2. Dimensional analysis was made to take account of differences of both tests. The results were derived that the values of c_v by the large scale test were 5 times larger than the other and the values of m_v were identical to both tests, independant of the scale effect of the oedometer ring. Compared to testing results, the value of c_v was found to be well related to the size of the ring of the oedometer.
3. It is characteristic that there exists the large portion of the immediate settlement in the time-sttlement relationships of the compacted soil.
4. For the portion of the consolidation settlement, the curve fitting method by numerical calculaltion is effective. Adopting this method to the portion of the immediate settlement routinely, it was seen to be able to fit successfully to the measuring values.
5. The calculated values from the pore pressure distribution equation with time dependent delay of the consolidation pressure were largely consistent with the measured values obtained by the small pore pressure meter which was buried at the center of the specimen. From this reasoning, it was noted that the portion of the consolidation settlement also followed the one dimensional consolidation theory of the saturated soil to some extent.

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大型圧密試験機による締固め土の圧密性状について

井　上　宗　治

本論文は大型圧密試験機（リングの大きさ　直径 30 cm，高さ 10 cm）を用いて締固め土の圧密特性を調べたものである。

大型試験機と標準試験機の試験値を比較した結果，体積圧縮係数は両者の間に大きな差異は認められなかったが圧密係数については大型試験機の方が標準試験機よりもかなり大きな値が得られた。そして，このような傾向が次元解析によって定量的に説明できることを示した。また，締固め土の圧密沈下量～時間関係において大型試験ではとくに即時沈下部分が大きな割合を占めることが特徴的である。数値計算法を用いてこの圧密沈下量～時間関係を即時沈下と圧密沈下の二つの曲線で表示したところ，計算値は比較的良好に実験値を近似し得ることが明らかとなった。

一方，供試体中に埋設した小型間隙圧計によって圧密中の間隙圧発生値を測定し，測定間隙圧値をテルツァギーの一次元圧密理論で計算される値と比較した結果，締固め土でも圧密沈下部分は飽和土理論に従うとみなしてもよいことを示した。