Estimation of m_v and c_v Values for the Design of Sand Drains

Estimation des Valeurs m_v et c_v pour le Calcul des Drains de Sable

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Summary

The method of estimating consolidation coefficients for the design of vertical sand drains is discussed. The effects of secondary compression, of side friction, and of remoulding on consolidation characteristics are considered. By comparing the values of the coefficients obtained from laboratory consolidation tests with the values from observations in the field, the authors found a convenient method of estimating these coefficients for design purposes.

Sommaire

Cette communication traite des méthodes d'évaluation des coefficients de consolidation pour le calcul des puits de sable verticaux. On examine l'influence du remaniement, de la compression secondaire et du frottement latéral sur les caractéristiques de consolidation. Les auteurs ont trouvé une méthode convenable pour estimer les coefficients en comparant les valeurs obtenues par des essais de consolidation en laboratoire avec celles obtenues sur le chantier.

Introduction

In the design of sand drains, estimates of the coefficient of consolidation c_v and the modulus of volume change m_v are essential for the prediction of consolidation rate and total settlement of the ground.

It is quite difficult to establish the procedure of determining the appropriate values of these coefficients by laboratory tests, because problems of friction between the consolidation box wall and the soil specimen, secondary compression, remoulding effects of soil, and complicated stress distribution in the ground are still obscure.

Since it seems to be very difficult to estimate the values of these coefficients from laboratory tests, it has been recommended that observations in test sections in the field should be used. The authors have tried to find and analyse the relationship between the values obtained from tests in the laboratory and those from observations in the field.

Secondary Compression

Y. ISHII (1952) found by his mathematical study on secondary compression that the magnitude of value of m_v varies with the duration of the consolidation process, and that secondary compression which appears in laboratory tests with the usual 24 hours duration must be included in the primary compression for consolidation phenomena in the field of more than a few months duration. He suggested that the magnitude of m_v used in the design must include the secondary compression which might appear in a period corresponding to the duration of consolidation phenomena in the field in addition to the m_v value of primary compression.

The ratio of the secondary compression to the primary obtained by 70 days loading tests are shown in Table 1. According to this table, about 50 per cent of total secondary compression during 70 days appears in the first 24 hours and 75 to 80 per cent in the first week. As is well known, the relation between secondary compression and time becomes a straight line on semi-log scale coordinates. The total secondary compressions of 70 days are 35 to 100 per cent of primary compressions.

Sometimes the settlement during 24 hours has been a base of computation of m_v value. The ratios of the secondary compression during 24 hours to the primary found in the authors' laboratory are summarized in Table 2. These ratios are about 60 to 80 per cent and, therefore, conventional m_v values from 24 hours loading are much larger than those by primary compression. The ratios for undisturbed samples have significantly larger values for loads smaller than pre-consolidation than the ratios for remoulded and re-consolidated samples.

Table 1

Ratio of the secondary compression to the primary obtained by 70 days' loading consolidation tests

Rapport de la compression secondaire à la compression primaire obtenue par les essais de consolidation de charge pendant 70 jours

Normal stress kg/cm²		0·151	0·378	3·02	6·04
Secondary compression %	1 day	34·2 (33·8)	27·0 (54·0)	20·5 (48·0)	16·2 (46·3)
	1 week	62·0 (61·3)	40·5 (81·0)	32·0 (74·9)	27·0 (77·1)
	2 weeks	67·0 (66·3)	45·0 (90·0)	35·6 (83·4)	31·0 (88·6)
	10 weeks	101·0	50·0	42·7	36·0
Rate of secondary compression	mm/min	$8{\cdot}05\times10^{-7}$	$1{\cdot}02\times10^{-6}$	$7{\cdot}94\times10^{-6}$	$7{\cdot}74\times10^{-6}$
	e/min	$1{\cdot}19\times10^{-7}$	$2{\cdot}08\times10^{-7}$	$1{\cdot}49\times10^{-7}$	$1{\cdot}20\times10^{-6}$

Sample—undisturbed Yokkaichi clay, clay content 68%, natural void ratio 1·9
( )—% of 10 weeks secondary compression

Table 2

Ratio of the secondary compression to the primary obtained by 24 hours' consolidation tests

Rapport de la compression secondaire à la compression primaire obtenue par les essais de consolidation pendant 24 heures

Sample	*Toyosu*	*Yokkaichi*	*Shimizu*	*Nagasaki*	*Shiogama*
Ratio %	60–108	30–70	56–71	36–70	56–80
Mean %	80	60	60	60	60

The effect of side friction on secondary compression obtained from the consolidation test also seriously affects the analysis of consolidation phenomena in the laboratory. During consolidation, the neutral stress must be transmitted to intergranular stress as well as to frictional resistance of the side wall. Both the strain produced by the increase of effective stress and that by the increase of frictional resistance are complicated and are closely related to each other. Moreover, creep of frictional strain might have an important influence on compression creep as well as on effective stress. Because of these complicated phenomena, it is doubtful if the so-called secondary compression obtained in laboratory tests is real consolidation creep.

Effect of Side Friction

TAYLOR (1942) found the values of 0·1 to 0·3 as coefficients of side friction f, in his tests on Boston Blue Clay.

TSCHEBOTARIOFF (1953) carried out the measurement of K_0 values on sand in a box of diameter 12 in., height 18 in., and found that the K_0 value at the depth of 3 in. was about five times its value at a depth of 16·5 in. showing the large effect of side friction.

M. Aoyama has computed the side friction coefficients

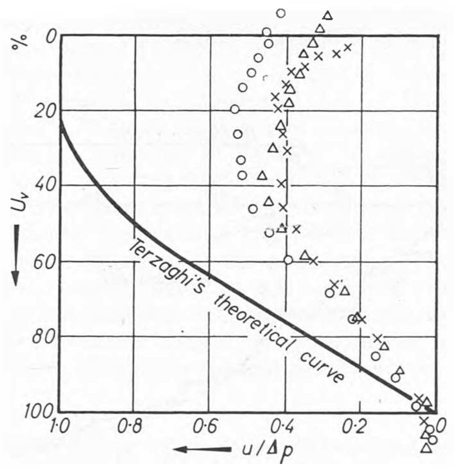


Fig. 1 Relation between U_v and $u/\Delta p$ on Amagasaki clay (L–6–2). U_v, degree of consolidation; u, pore water pressure; Δp, increment of consolidation pressure

Relation entre U_v et $u/\Delta p$ pour l'argile d'Amagasaki (L–6–2). U_v, degré de consolidation; u, pression interstitielle; Δp, augmentation de la pression de consolidation

from the difference of effective stresses resulting from the different side friction in fixed and floating ring types consolidometers. Assuming that the mean effective stress within the samples must be equal for the same void ratio, the coefficients of side friction f were computed from the following equation:

$$f = \frac{R}{2H} \log_e \left(\frac{P_{fix}}{2P_{flt} - P_{fix}}\right)$$

where, R is the radius of sample, H the height of sample and P_{fix}, P_{flt}, the actual load for the same void ratio. The f-values obtained from undisturbed Yokkaichi clay (clay content 46 per cent, void ratio 1·92, and LL 98 per cent) are shown in Table 3.

Table 3
Results of Aoyama's tests
Résultats des essais d'Aoyama

P_{fix} kg/cm²	0·378	0·755	1·51	3·02	6·04	12·1	15·1
P_{flt} kg/cm²	0·260	0·480	1·07	2·09	4·12	8·2	10·2
f	1·63	2·22	1·61	1·93	2·33	2·5	2·6

These values are much larger than those obtained by Taylor's tests. At present these large differences are doubtful because of the difficulty of measuring the void ratio exactly. However, the differences are enough to show the importance of side friction.

ARAI (1955) computed the side friction coefficient from the pore water pressure measured in consolidation tests. Fig. 1 shows the measured pore water pressure. These have considerably smaller values than the increments of applied load. It seems to be due to the friction between soil and ring wall. Assuming that the side friction is mobilized from the beginning of consolidation, side friction coefficient f is computed from the following equation:

$$= \frac{R}{4H'} \log_e \left(\frac{P + \Delta P}{U + P.e^{-(4f'H/R)}}\right)$$

Where, U is excess pore water pressure, P, the consolidation pressure, ΔP, the increment of P, f' the side friction coefficient before consolidation, and H' the sample height after consolidation.

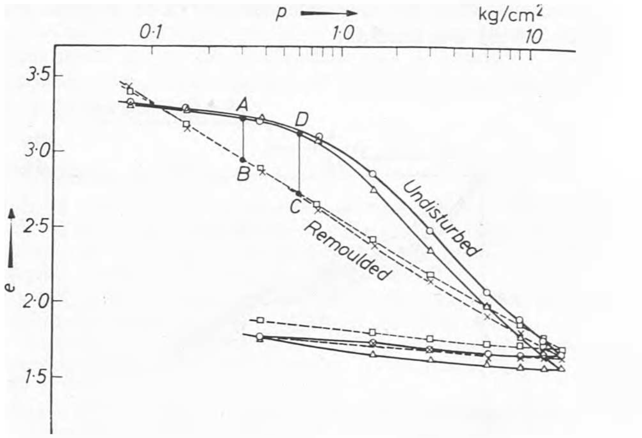


Fig. 2 e-log p curves of undisturbed and remoulded Takashima clay (8–2139)

Courbes e-log p des argiles intactes et remaniées de Takashima (8–2139)

Table 4 shows the f values on Yokkaichi and Amagasaki clays. These values are somewhat larger than the values obtained by Taylor's tests.

Table 4
f-values obtained by Arai's tests
Valeurs f obtenues par des essais d'Arai

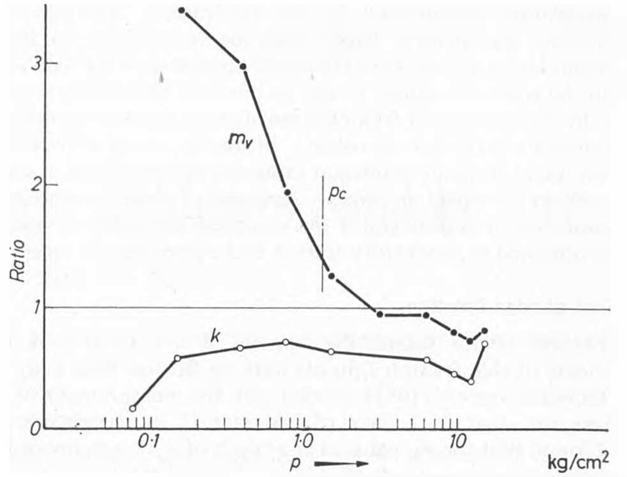
Consolidation pressure kg/cm²	0·51	1·02	2·04	4·08	8·16
Sample: Amagasaki L–6–2	0·589	0·701	1·01		
Amagasaki L–6–3	0·215	0·319			
Amagasaki L–6–	0·097	0·139	0·216	0·351	
Yokkaichi remoulded 4–2–1	0·153	0·354	0·604	0·961	1·39
Yokkaichi remoulded 4–2–2	0·133	0·320	0·470	0·722	1·13

Considering these test data, it seems to be necessary to consider about 0·5 as value of the coefficient of side friction. When f is equal to 0·5, values of m_v and k increase by 30 per cent and effective consolidation pressures decrease by 23 per cent for the sample in the fixed ring, and 15 per cent and 13 per cent respectively for the sample in the floating ring.

Remoulding Effect

It is very difficult to estimate to what extent soils will be remoulded by sand drain driving, but it is valuable to know the difference between consolidation of undisturbed and completely remoulded samples. The void ratio of a remoulded sample is always smaller than that of an undisturbed sample unless the latter is remoulded by large consolidation stress. Therefore, in estimating settlements in a sand-drain project, a certain amount must be allowed for the remoulding of soils.

The ratios of m_v and k values of remoulded samples to those of undisturbed samples are shown in Fig. 3, which indicates that the ratios of k values of remoulded to those of undisturbed samples are always smaller than unity under any consolidation stress. However, the ratios of m_v values are far larger than unity for consolidation pressures smaller than the pre-consolidation load and become smaller for pressures larger than pre-consolidation load. If soils are remoulded completely under a certain pressure, the void ratio might decrease from A to B as shown in Fig. 2, and move to C by increased load. In the field, the void ratio will have intermediate value between D and C after driving and loading.



p_c: pre-consolidation pressure

Fig. 3 Ratio of m_v and k values of remoulded to undisturbed Shimizu clay (3–1065)

Rapport des valeurs m_v et k des argiles remaniées aux argiles intactes de Shimizu (3–1065)

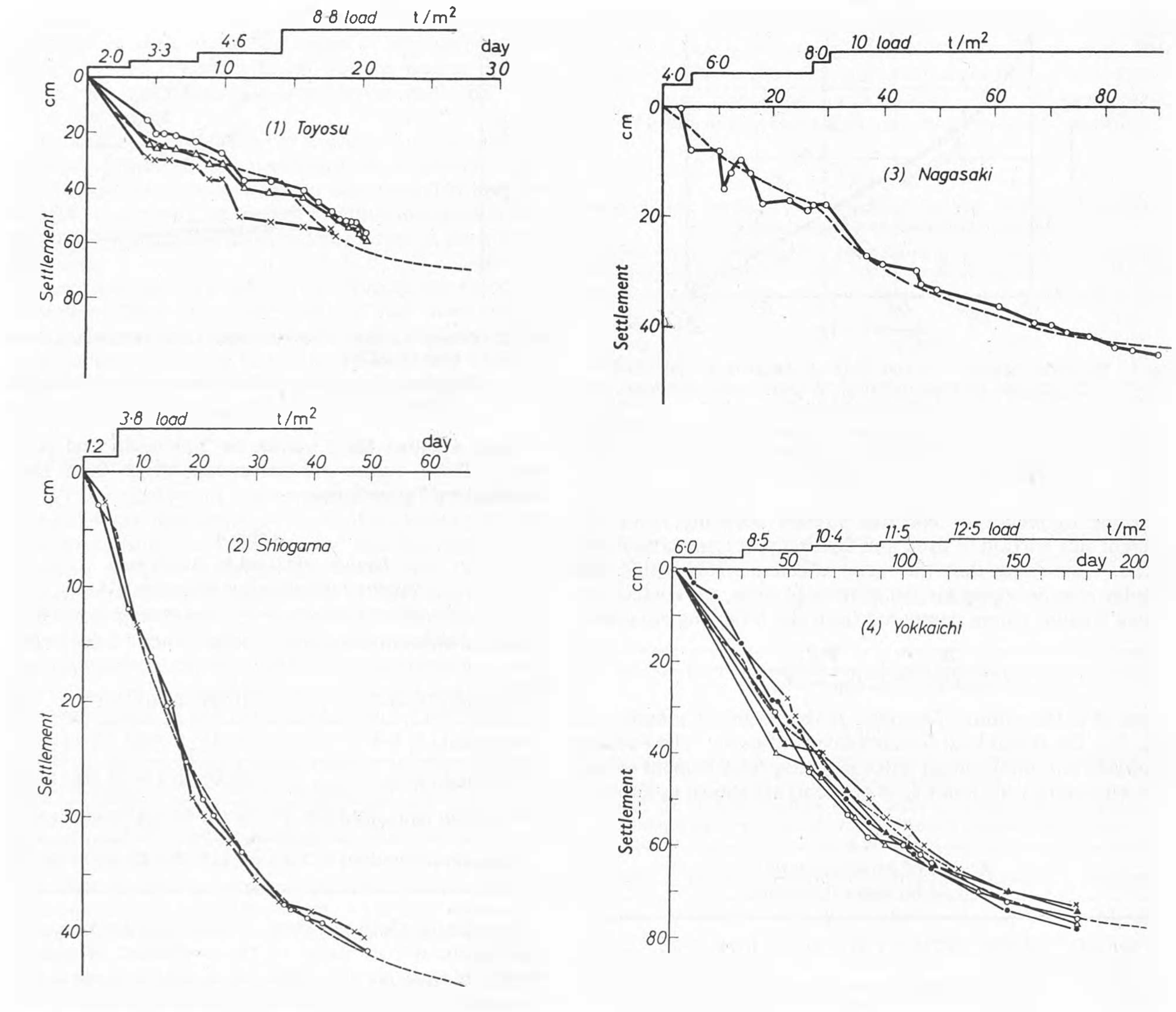


Fig. 4 Settlement curves

Courbes de tassement

Values of m_v and c_v Obtained by Observations in the Field and Comparison of these Values with those Obtained by Laboratory Tests

A summary of four sand drain projects is given in Table 5.

Table 5

Summary of four sand drain projects

Sommaire de quatre sable-drains projets

Site	*Thickness of layer* m	*Diameter of sand drain* cm	*Spacing of sand drain* m	t_{80} day	$qu/2$ *before drain* kg/cm²	$qu/2$ *after drain* kg/cm²	*Applied load* t/m²
Toyosu	10	40	1·8	20	0·1	0·3	8·8
Shiogama	3–6	45	1·75	30	0·05	0·15	3·8
Nagasaki	4·5–6	40	2·0	54	0·2	0·35	10·0
Yokkaichi	12	45	1·7	100	0·2	0·4	12·5

In Fig. 4, the full lines represent settlement observed practically in the field, and broken lines represent computed lines following to BARRON's method (1948) by finding m_v and c_v values by trial and error method. The c_v', m_v' and k' values obtained from the observations are shown in Table 6. Both m_v values computed from 24 hours loading, and c_v values from primary compression are indicated in Table 6. Coefficients of permeability were computed from the above m_v value (24 hours loading) and coefficient of consolidation (primary compression).

The important observations which follow from these comparisons are as follows: (1) the magnitude of consolidation coefficient c_v is nearly equal to that computed from primary compression obtained from laboratory tests, and (2) magnitude of modulus of volume change m_v approximates to the value computed from 24 hours settlement in laboratory.

Conclusions

Although very large values of m_v are indicated by consideration on secondary compression, remoulding effect, side friction and so on, the 24 hour settlement in the laboratory test gives close value of settlement in the field. The fact that the c_v value computed from primary compression coincides closely with the values observed in the field shows that permeability k greatly increases, presumably because of the large permeability of the soil in horizontal direction.

Table 6

List of consolidation coefficients obtained by laboratory tests and by observation in the fields

Liste des coefficients de consolidation obtenues par observations dans les places et les essais dans le laboratoire

Site		m_v cm²/kg	c_v cm²/min	k cm/min	m_v' cm²/kg	c_v' cm²/min	k' cm/min
Toyosu		$6{\cdot}0 \times 10^{-2}$	$3{\cdot}0 \times 10^{-1}$	$1{\cdot}8 \times 10^{-5}$	$7{\cdot}0 \times 10^{-2}$	$3{\cdot}0 \times 10^{-1}$	$2{\cdot}1 \times 10^{-5}$
	r	1·8	1·0	1·8	2·1	1·0	2·1
Shiogama		$5{\cdot}0 \times 10^{-1}$	$1{\cdot}8 \times 10^{-1}$	$9{\cdot}0 \times 10^{-4}$	$3{\cdot}8 \times 10^{-1}$	$1{\cdot}8 \times 10^{-1}$	$6{\cdot}8 \times 10^{-4}$
	r	1·6	1·0	1·6	1·2	1·0	1·2
Nagasaki		$8{\cdot}0 \times 10^{-2}$	$1{\cdot}2 \times 10^{-1}$	$9{\cdot}6 \times 10^{-5}$	$8{\cdot}0 \times 10^{-2}$	$1{\cdot}2 \times 10^{-1}$	$9{\cdot}6 \times 10^{-5}$
	r	1·6	1·0	1·6	1·6	1·0	1·6
Yokkaichi		$7{\cdot}0 \times 10^{-2}$	$4{\cdot}0 \times 10^{-2}$	$2{\cdot}8 \times 10^{-6}$	$6{\cdot}6 \times 10^{-2}$	$4{\cdot}0 \times 10^{-2}$	$2{\cdot}6 \times 10^{-6}$
	r	1·6	1·0	1·6	1·5	1·0	1·5

m_v, value obtained by 24 hours loading; c_v^2, value obtained by primary compression; k, computed from values of m_v and c_v; m_v', c_v', k', obtained by analysis of settlement curves observed; r, ratio of coefficients to values determined by primary compression

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