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DEVELOPMENT OF A NEW CONSOLIDATION TEST PROCEDURE USING SEEPAGE FORCE

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ABSTRACT

A new style of consolidation test is proposed for the purpose of predicting consolidation constants of sediments to be formed in a dyked pond by hydraulically dredged clay materials. This new test is performed by applying the seepage force on a specimen prepared by sedimentation in a consolidometer. The principle that the seepage force is converted into consolidation stress is at first presented. It is next shown that all of the consolidation constants and the compression curve as well can be determined by performing following three measurements in the state of steady seepage flow realized after the completion of consolidation; (1) distributions of pore water pressures and of (2) water contents within the specimen, and (3) velocity of the flow passing through the specimen. With use of the test apparatus developed for this study, consolidation constants were determined in the wide range of stress from the very low stress of 0.01 kN/m^2 to 50 kN/m^2.

Key words: cohesive soil, consolidation test, seepage, test equipment, test procedure
IGC: D5

INTRODUCTION

Many of the land filling works in ports of Japan are in general carried out by hydraulically dredging bottom sediments and then pumping the dilute soil-water mixtures into dyked ponds located near the shore. When the bottom sediment is clayey, a landfill initially having a very high water content is unavoidably formed. In this case, a large settlement inevitably takes place owing to the self-weight of the landfill itself. Since the process is usually quite time-consuming, it strongly influences the plannings about soil stabilization, future utilization of the landfill, and so on. Therefore, it is very important to evaluate beforehand the settlement-time behaviors of a landfill.

This evaluation can be successfully executed by use of a consolidation theory which takes account of the self-weight of soils as well as a considerable change in consolidation constants caused by the increasing consolidation stress during the self-weight consolidation. As far as the author awares, such large-strain consolidation theories different from the classical linear theory have been proposed by Mikasa (1963) and Monte and Krizek (1976). As a matter of course, these theories are useless when no prior informations regarding consolidation constants are available.

When dilute slurries of dredged soil, here called "dredged fluid mud," is pumped into a pond, soil particles settle down generally forming flocs, their fabric being formed under complex restrictions such as physico-chemical interactions between the particles and the

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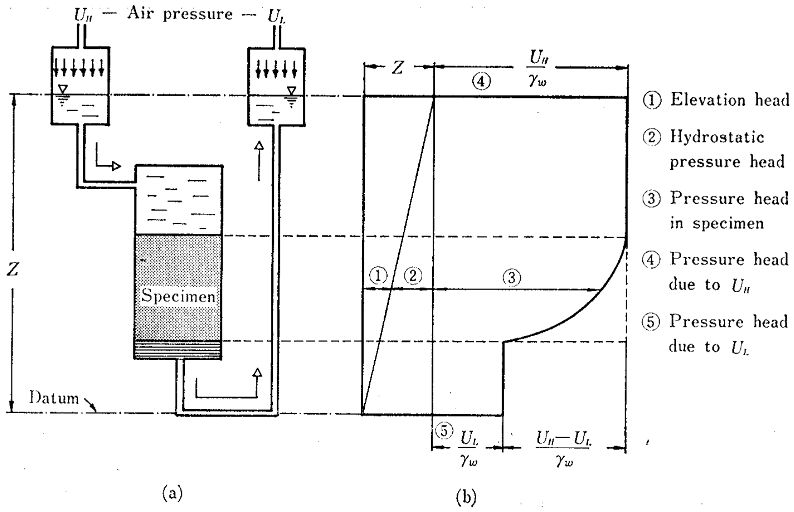


Fig. 1. Fundamental ideas of hydraulic consolidation: (a) water flow caused by a constant water head difference consolidates the specimen prepared from dredged fluid mud by sedimentation; (b) water head distribution throughout the system

surrounding water, solid concentration of the dredged fluid mud, and so on. These flocs settle on the bottom and compress the underlying flocs. Since the buoyant weight of flocs is very small, self-weight consolidation may start from the very low effective stress. Hence, the consolidation constants required for analyses of the self-weight consolidation process should be obtained (1) for the low effective stress and (2) by use of a test specimen prepared from dredged fluid mud. The conventional oedometer test evidently cannot satisfy these two requirements simultaneously. To surmount this difficulty, several trials have been carried out by Umehara and Zen (1975) and Keshian, Ladd and Olson (1977).

In this study, another method, here called "hydraulic consolidation test," is further proposed. It is based on the principle of consolidation by seepage force. When an artificial sediment prepared from dredged fluid mud is subjected to the forced water flow which is induced by a constant water head difference, every element in the specimen is consolidated by the seepage force acting on the element (Fig. 1(a)). After the consolidation is completed at every element, condition of steady seepage flow is realized. In this steady state, effective stress and hence water content as well vary from point to point in the specimen as shown in Fig. 1(b). By the measurements of pore water pressure distribution, water content distribution and velocity of the flow passing through the specimen, all the consolidation constants necessary to settlement analyses of a landfill can be obtained.

FUNDAMENTAL IDEAS OF HYDRAULIC CONSOLIDATION

Principle of Consolidation by Seepage Force

In the conventional oedometer tests, loading weights are applied on the surface of specimen step by step; namely, consolidation is caused by the surface force. In the hydraulic consolidation tests, on the other hand, every element in a specimen is consolidated by the seepage force acting on the element; namely, consolidation is caused by the body force.

To see the mechanism controlling the consolidation by seepage force, it should be clarified here how the seepage force is converted into the effective consolidation stress. Fig. 2 shows a stress state of an element in specimen, through which seepage water steadily flows downward. Since the total stress difference between the elevation z and $z+dz$ results from only total weight of the element, the next relationship holds:

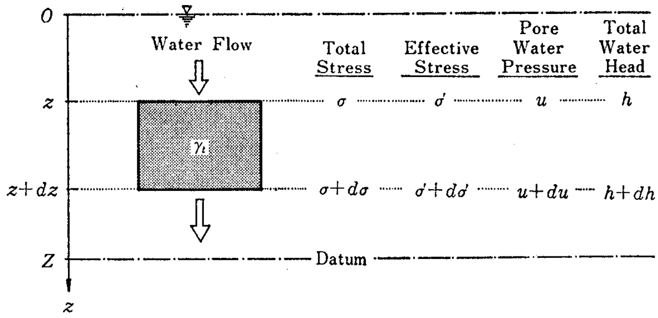


Fig. 2. Stress state of an element in specimen being subjected to the action of seepage force

$$d\sigma=\gamma_t dz \tag{1}$$

where γ_t is unit weight of the element. By applying the principle of effective stress to Eq. (1) the change in effective stress, $d\sigma'$, is given as follows:

$$d\sigma'=\gamma_t dz-du \tag{2}$$

where du is the change in total water pressure in which hydrostatic pressure is included. Because the change in total water head is, on the other hand, given by

$$dh=\frac{1}{\gamma_w}(du-\gamma_w dz) \tag{3}$$

the next fundamental equation is obtained by combining Eq. (2) with Eq. (3):

$$\frac{d\sigma'}{dz}=-\gamma_w\frac{dh}{dz}+\gamma' \tag{4}$$

where γ_w and γ' are the unit weight of water and the buoyant unit weight of the element, $\gamma_t-\gamma_w$, respectively.

In the soil mechanics, seepage force is expressed by the force per unit soil volume, j, which is defined by the total water head gradient, i, times the unit weight of water:

$$j=\gamma_w i \tag{5}$$

where i is given as follows according to its definition:

$$i=-\frac{dh}{dz} \tag{6}$$

Hence, Eq. (4) results in

$$\frac{d\sigma'}{dz}=j+\gamma' \tag{7}$$

This Eq. (7) means that the gradient of effective consolidation stress is caused by the action of seepage force and by the buoyant weight of soil. In other words, seepage force and/or buoyant weight of soil is converted into the effective consolidation stress according to Eq. (7).

Fundamental Procedures of Consolidation Constants Determination

When a steady state of seepage flow is realized after the completion of consolidation, distributions of j and γ' no longer change. By measuring z downward from the top surface of specimen in this steady state, the solution of Eq. (4) or Eq. (7) is obtained followingly:

$$\sigma'(z)=\gamma_w\{h_0-h(z)\}+\int_0^z \gamma'(\eta)d\eta \tag{8}$$

where h_0 is the total water head at the top surface of specimen. Since the value of $h(z)$

is naturally smaller than h_0 as shown in Fig. 1 (b), and the value of γ' is positive, σ' increases according to the increase in z. The value of σ' is of course zero at $z=0$, because no effective consolidation stress must act at the top surface. Since water content of normally consolidated soils decreases as effective stress increases, the water content in the specimen varies from place to place decreasing its value with the increase in z. Hence, when the water content distribution and the pore water pressure distribution in the specimen are measured, compression curve and therefore compression index as well can be determined in the whole range of consolidation stresses realized in specimen.

The coefficient of volume compressibility can further be determined by use of the compression curve. When a soil mass having a void ratio of e_0 is consolidated by a stress increment, $\Delta\sigma'$, and a consequent decrease in the void ratio of Δe takes place, the strain increment induced is expressed by

$$\Delta\varepsilon=\frac{\Delta e}{1+e_0} \tag{9}$$

Therefore, the coefficient of volume compressibility is determined by the following equation:

$$m_v=\frac{1}{1+e_0}\frac{\Delta e}{\Delta\sigma'} \tag{10}$$

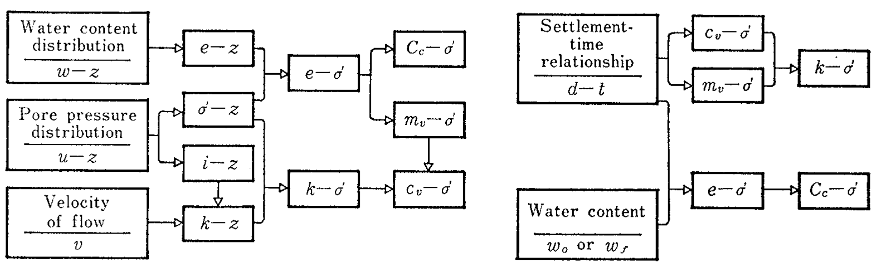
To determine the coefficient of consolidation, permeability coefficient should be further obtained. When velocity of flow, v, is measured in the steady state after the consolidation, the coefficient of consolidation can be determined after Darcy's law:

$$k=\frac{v}{i} \tag{11}$$

Since v is constant throughout the specimen and the total water head gradient, i, is calculated from the distribution of pore water pressures, value of k can be obtained as a function of effective consolidation stress. Coefficient of consolidation is, therefore, determined by combining Eq. (10) with Eq. (11); it is calculated as follows according to its definition:

$$c_v=\frac{k}{\gamma_w m_v} \tag{12}$$

A flow chart showing the above stated procedures is drawn in Fig. 3, where another chart for the conventional oedometer test is also presented for comparison. In the case of conventional test, c_v is determined by directly fitting the consolidation theory to the primary consolidation process, i.e., unsteady process, and k is calculated from the com-



(a) Hydraulic consolidation test (b) Conventional oedometer test

Fig. 3. Fundamental flows to determine the consolidation constants: (a) in the case of hydraulic consolidation test; and (b) the conventional oedometer test

bination of m_v and c_v. In the hydraulic consolidation test, on the other hand, m_v and k are determined for the steady state, and c_v is calculated by combining them; in this case there is no need to employ any consolidation theory.

Here arises an important question; i.e., "Is there any assurance that c_v calculated from the steady state naturally accords with c_v measured during the unsteady consolidation process?" Answer to this question cannot be given theoretically but given only when examining experimentally whether the permeability coefficient "measured" in the steady state accords or not with that "calculated" from the unsteady consolidation process. Such examinations carried out in the past are summarized by Mesri and Olson (1971). The results show that "measured" permeability coefficients are near or slightly higher than "calculated" values. Hence, the value of c_v determined by the hydraulic consolidation test is expected to be near or slightly higher than the value from conventional test. Strict comparisons between the two values will be discussed in the other paper and is not mentioned in this paper.

CONSTITUTION OF TEST APPARATUS

Outline of Test Apparatus

Fig. 4 shows an outline of the test apparatus developed for the present study. This apparatus is designed to allow the following operations and measurements: (1) preparing a test specimen in consolidometer; (2) applying a constant water head difference on the specimen; (3) measuring the water volume which passed through the specimen; and (4) measuring a distribution of pore water pressures in the specimen. To satisfy these requirements, the test apparatus consists of four systems: (1) system for soil sedimentation and consolidation, i.e., consolidometer; (2) system applying a constant water head difference, i.e., air compressor, air regulators, pressure gauges, head tanks and winches; (3) system measuring flow velocity, i.e., double-tube flow meters; (4) system measuring pore pressures, i.e., pickups and strain indicator.

Water in the high pressure head tank flows passing through the specimen in consolidometer, and then the flow is separated into two at the bottom of specimen; one is the flow from the central area of cross section and the other is from the circumferential area en-

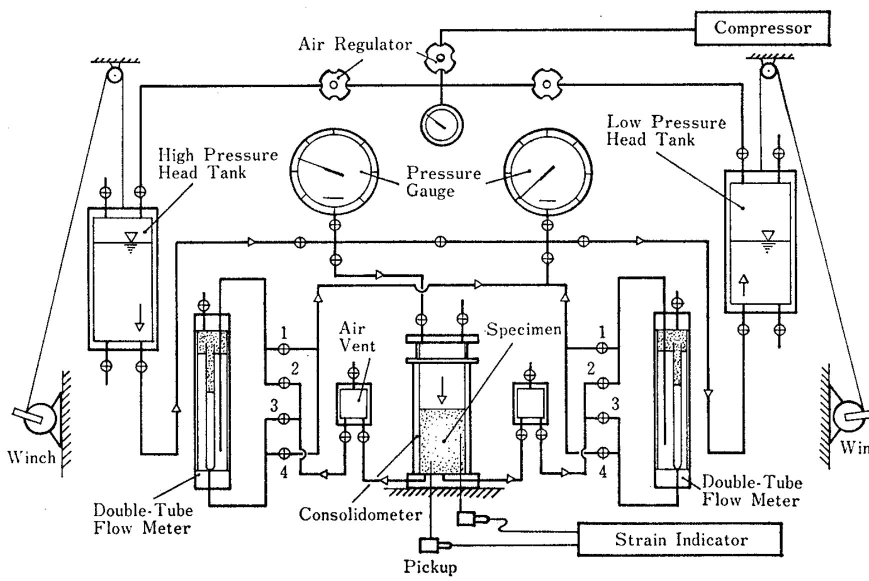


Fig. 4. An outline of the test apparatus used in the present study

circling the central area. These two flows join again after having passed through each double-tube flow meter. The joined flow lastly runs into the low pressure head tank. Suspended positions of the both head tanks are always adjusted by winches to nail the free surface in tanks to the fixed elevation predetermined.

Details of Consolidometer

The consolidometer used in this study has dimensions of 12 cm in diameter and 40 cm height by inside measurements. Four tension rods assemble it from the following parts; (1) cap plate, (2) acrylic cylinder, (3) bottom plate and (4) intermediate plate, which ties the cylinder to the bottom plate when no cap plate is used. Details of the bottom plate is shown in Fig. 5. As the water flow in specimen may not be uniform over the cross section, that is, a disturbed flow is apt to occur near the wall of cylinder, the porous stone interposed between the specimen and the bottom plate should be divided into two pieces; one of them is circular and incorporated at the central portion of the bottom plate to catch the undisturbed flow passing through the central area of specimen, and another daughnut-like one is at the surrounding portion to catch the disturbed circumferential flow. Water communication between these two porous stones is perfectly cut off by a dividing wall burried in bottom plate at the boundary between the central and the circumferential porous stone.

Pickup for Pore Pressure Measurements

Pore water pressure in the specimen are measured by the pickup shown in Fig. 6 in detail. It consists of a lead pipe and a chamber in which an electric pressure transducer is incorporated. The pickup is made of brass to ensure its stiffness, because insufficient stiffness causes dull responsibility of the system (Whitman, Richardson and Healy, 1961; Northey and Thomas, 1965).

At the end point of lead pipe two little holes are drilled laterally. Pore water pressure caught at this hole is transmitted through the lead pipe to the pressure transducer, at which it is transfered into electric voltage. Six pickups were used in this study.

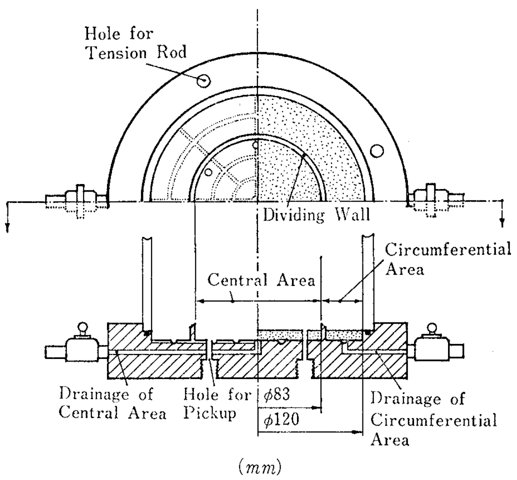


Fig. 5. Details of bottom plate of consolidometer; porous stone is divided into two by a dividing wall to catch undisturbed steady flow passing through the central area of specimen

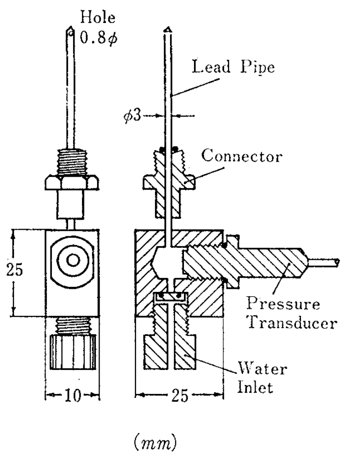


Fig. 6. Details of pickup for the measurement of pore water pressures; lead pipe is penetrated into the specimen through the hole in bottom plate

Double-tube Flow Meter

A water volume which had passed through the central and circumferential area of specimen is measured by each double-tube flow meter. Double-tube system was employed to avoid the fracture of burrette due to the back pressure applied throughout the system of apparatus. A burrette of 25 cm^3 is set in an outer glass cylinder as shown in Fig. 4. A change in elevation of the interface between the coloured oil and the water in burrette gives the water volume flowed. When valves 1 and 3 are closed releasing 2 and 4, the interface moves downward; the reversal valve operation results in the upward movement. When no measurement is necessary, valves 1 and 2 or 3 and 4 should be closed releasing 3 and 4 or 1 and 2, respectively; in this case no flow through flow meter takes place.

TEST PREPARATIONS

Preparations of Test Specimen

In hydraulic consolidation tests, test specimens are prepared in consolidometer. First of all, therefore, fluid mud having an expected water content must be prepared by mixing the sea water with the muddy sediment sampled from dredging site. When the fluid mud is poured into the consolidometer, gravitational sedimentation takes place. The process of sedimentation is in general divided into the following three stages: (1) soil flocs or fabrics are formed in the initial stage; (2) they fall through the water in the second stage to form a sediment at the bottom; and (3) the sediment undergoes self-weight consolidation in the last stage (Keshian, Ladd and Olson, 1977; Yano, Imai and Tsuruya, 1977; Yano and Imai, 1977).

When fluid mud with high water content is dealt with, all of the above three stages occur. When water content of fluid mud is low, on the other hand, no second stage appears. Because the consolidation test should be carried out after the fabric formation had been completed, seepage force must be applied on the specimen being in the process of self-weight consolidation, i.e., in the last stage of sedimentation. When working with the fluid mud having a very high water content, a considerable degree of particle segregation is apt to occur during the initial and second stages of sedimentation. The particle segregation is especially unavoidable when coarser materials are contained in the fluid mud. To avoid this troublesome problem, multi-stage sedimentation method is effective; in this case a uniform specimen can be obtained (Bjerrum and Rosenqvist, 1956). When dealing with fluid mud of considerably low water content, on the other hand, this particle segregation is out of problems.

In the present study, fluid mud of 480-percent water content was prepared by use of the bottom sediment ($w_L=98\%$, $w_P=32\%$) and the sea water both sampled from Tokyo Bay. This fluid mud was poured into the consolidometer as to produce a specimen of about 5-cm thickness after the consolidation. Examples of test results obtained for the mud will be shown in the later sections.

Application of Seepage Force

After the preparation of test specimen, a space above the sediment in consolidometer is filled with the same water as used in the fluid mud preparation, and covered with cap plate. Prior to the application of seepage force, free surfaces in two head tanks should be adjusted to the same elevation, at which the zero-point of two pressure gauges should be determined. A back pressure is next applied throughout the system. The necessity of back pressure results from the following two reasons: (1) the perfect linear response of pressure transducer is not necessarily secured in the low pressure near zero; (2) a high degree of saturation ensures an accuracy of the pore pressure value measured. The back pressure is applied by increasing the air pressures in both head tanks to the same predetermined level, U_L, which is checked by each pressure gauge. In this study, the back pressure of 50 kN/m² was employed. When the air pressure in high pressure head tank is further increased to another higher level, U_H, and the water pressure thus induced in the tank is led to the water above specimen in consolidometer, consolidation by seepage force starts under the action of the pressure difference, $\Delta U_a = U_H - U_L$.

MEASUREMENT PROCEDURES

Measurements of Flow Velocity

The velocity of flow should be measured frequently during the process of consolidation. It is determined by reading the change in interface locations in burrette on the cubic-centimeters scale. When the reading is repeated at every time interval, Δt, during $2n\Delta t$ minutes, and those readings are denoted by $r_1, r_2, \cdots, r_i, \cdots, r_{2n}$, the velocity of flow is calculated by

$$v = \frac{1}{An\Delta t}\sum_{i=1}^{n}|r_{2i-1} - r_{2i}| \tag{13}$$

where A is the cross sectional area; in this study, the value of A was 54.1 cm² for the central area and 59.0 cm² for the circumferential area. As a matter of course, velocity of flow decreases as consolidation proceeds. An example of this trend is shown in Fig. 7, where the reduction of specimen thickness with time is also shown.

In this example, the unsteady consolidation process comes to an end at about 1300 minutes and thereafter little change in specimen thickness and in flow velocity of central area as well can be appreciated; that is, a steady state is attained. But the trend of flow velocity of circumferential area is quite different from that of central area. When the unsteady consolidation process is completed, it abruptly increases and then decreases gradually. This extraordinary behavior may be caused by a change in specimen shape; the shape of top surface becomes concave due to the side friction, a gap is then formed between the specimen and the wall. From these results, it is concluded that the velocity of flow for the calculation of permeability coefficient should be determined by use of the undisturbed steady flow passing through the central area.

Measurements of Pore Pressure Distribution

All of the six pickups No. 1 to No. 6 are connected to the bottom plate preceding the

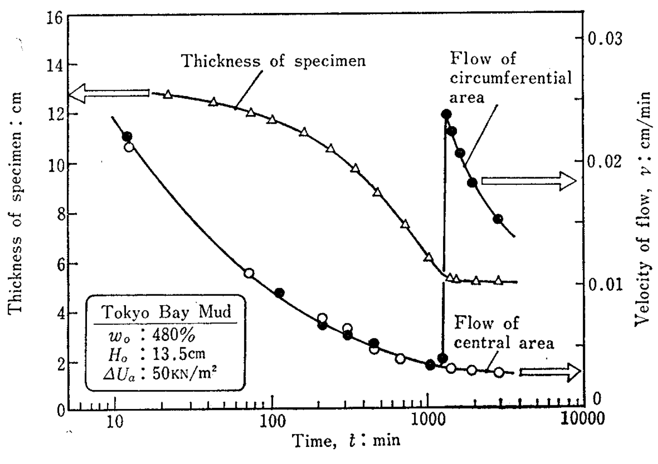


Fig. 7. Changes in flow velocities and specimen thickness with progress of the consolidation by seepage force

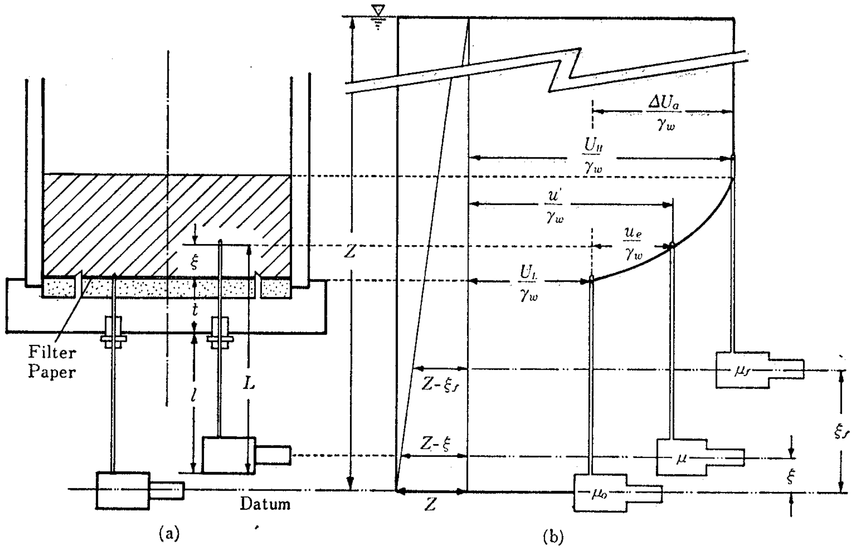


Fig. 8. Basic diagrams for the calculation of pore pressure distribution: (a) relationships among *L, t, l* and ξ; (b) relationships between the pore water pressure at hole of lead pipe and pressure acting at the transducer

specimen formation, the side little holes of every lead pipe being located just at the position of filter paper covering the porous stone (Fig. 8(a)). After having confirmed the completion of consolidation, every pickup is penetrated into the specimen upward and fixed to the bottom plate as is shown in Fig. 8(a). As a result of disturbance due to the penetration of lead pipe, the pore pressure measured shows an incorrect value during the initial several to score minutes. But an output value of the transducer, being indicated by a strain indicator by micro-voltage, gradually settles with recovery of the disturbance and lastly converges into a fixed value. This fixed output value corresponds to the correct pore water pressure at the hole of lead pipe. Elevation of the hole, ξ, is measured upward from the bottom of specimen and is calculated by the following equation:

$$\xi=(L-t)-l \tag{14}$$

where L, t and l are beforehand known, ξ is correctly determined by measuring l.

The procedure of pore water pressure measurement are divided into the following three steps:

(1) Output value of each transducer, μ_0, is read before penetrating the pickup. That is, the value of μ_0 corresponds to the pore water pressure at $\xi=0$; i.e., the value of back pressure.

(2) After penetrated every pickup a little distance, the position of hole, ξ, should be determined by use of Eq. (14), and the output value, μ, is read correspondingly to that elevation. The same procedures are repeated by penetrating every pickup inch by inch untill the little hole reaches to the top surface of specimen.

(3) Lastly, the little hole should be located in the water above specimen, and the output value, μ_f, and the location, ξ_f, are determined. The value of μ_f corresponds to the high pressure, U_H.

Determination of Pore Pressure Distribution

From the data obtained above, pore pressure distribution in the specimen can be determined. Fig. 8(b) gives the relationship among water head distribution, location of the hole of lead pipe and location of the transducer. As can be realized by the figure, output value, μ_0, at the position $\xi=0$, results from the following two contributions; i.e., hydrostatic water pressure, $\gamma_w Z$, and applied low pressure, U_L. Since the output of a transducer linearly increases with the total water pressure acting on the diaphragm of transducer, the next relationship holds:

$$\mu_0=\alpha_1(U_L+\gamma_w Z)+\alpha_2 \tag{15}$$

where α_1 is a linearity constant and α_2 is a constant. From the same discussions as above, μ_f is expressed by

$$\mu_f=\alpha_1\{U_H+\gamma_w(Z-\xi_f)\}+\alpha_2 \tag{16}$$

By eliminating α_2 from Eqs. (15) and (16), the linearity constant, α_1, is determined for every transducer as follows:

$$\alpha_1=\frac{\mu_f-\mu_0}{\Delta U_a-\gamma_w\xi_f} \tag{17}$$

In this study, value of α_1 ranged from 7.15 to 8.33 μV/kN/m². When the pore water pressure excluding the hydrostatic pressure at the elevation, ξ, is denoted by u', it is determined by

$$u'=U_L+\frac{\mu-\mu_0}{\alpha_1}+\gamma_w\xi \tag{18}$$

Since the pressure, U_L, is the back pressure and does not contribute to the consolidation, the pore water pressure effective to consolidation should be

$$u_e=\frac{\mu-\mu_0}{\alpha_1}+\gamma_w\xi \tag{19}$$

Hence, the water head effective to consolidation is given as follows:

$$h_e=\frac{u_e}{\gamma_w} \tag{20}$$

Fig. 9 shows an example of the pore pressure distribution obtained through the process stated above by using the transducers of 200 kN/m² capacity. The plotted points show little scatter although different six pickups were used. Because the minimum reading accuracy of the indicator was 1 μV, pore pressure values were determined with an accuracy of about 0.2 kN/m².

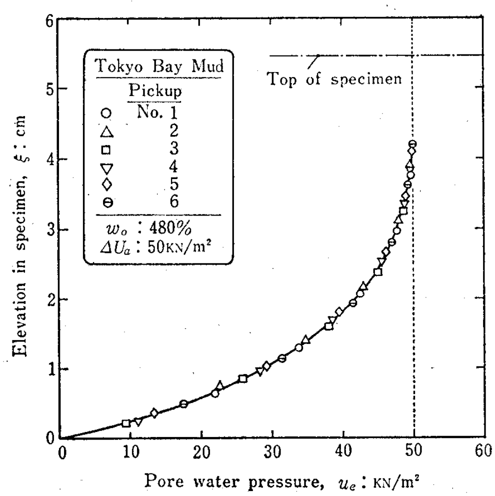


Fig. 9. An example of the pore pressure distribution in specimen

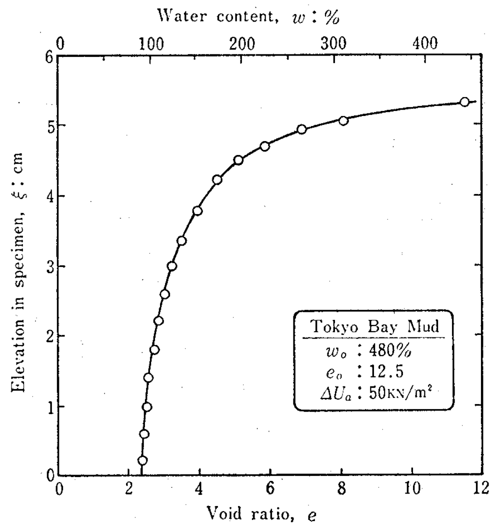


Fig. 10. An example of the void ratio distribution in specimen

Measurements of Water Content Distribution

After the pore pressure measurements, the high pressure, U_H, is dropped to the low pressure, U_L, then pressures in both head tanks are released simultaneously. As a matter of course, on the removal of seepage force a consequent volume expansion unavoidably takes place. But, at present, we have no means to know the water content in the specimen which is undergoing the seepage force. Hence, the water content cannot but be measured by using the specimen which had experienced some degree of volume expansion.

Sampling from the specimen can easily be performed by penetrating a sampling tube vertically into the specimen; in this study a brass tube of 50 mm in inside diameter, 150 mm length and 1 mm thickness was used. After pulled up the tube out of the specimen, a pushing piston is inserted into the tube from its bottom. By pushing this piston upward relatively to the tube inch by inch a thin layer of the sampled column of soil appears out of the tube from its top end. When these slices of soil are cut with a predetermined thickness and their water contents are measured, water content distribution is obtained.

Since water content is usually very high near the top surface of specimen where the very low effecitive stress acted on, the water contents measured by oven drying procedures should be corrected taking account of the effect of salinity (Imai, 1978). When salt content, β, is introduced, the correct water content, w, is calculated from the measured value, w_m, as follows:

$$w=\frac{1+\beta}{1-\beta w_m}w_m \tag{21}$$

Salt content, β, should be determined by drying a sample of water which had passed through the specimen; it is defined by the weight ratio of salt crystal obtained after drying to fresh water evaporated through drying. The correct void ratio should, therefore, be calculated by use of the corrected water content as follows:

$$e=\frac{G_s}{G_w}w \tag{22}$$

Where G_s and G_w are the specific gravity of soil particles and water, respectively. Fig. 10 shows an example of the void ratio distribution determined by the processes stated above for the Tokyo Bay mud.

DETERMINATION OF EFFECTIVE STRESS DISTRIBUTION

Prior to the determination of consolidation constants, a distribution of effective consolidation stresses must be determined on the basis of Eq. (8). When the effective stress due to seepage force and buoyant weight of soil is denoted by σ_j' and σ_r', respectively, the effective consolidation stress, σ', is given by their summation. Since the value of σ_j' is given by the first term in the right hand side of Eq. (8), and the total water head difference, $h_0-h(z)$, is equal to $\{\varDelta U_a-u_e(z)\}/\gamma_w$ as can be realized from Fig. 8(b), the distribution σ_j' can be determined by the following equation, because the pore water pressure distribution was already determined by Eq. (19):

$$\sigma_j'=\varDelta U_a-u_e \tag{23}$$

The applied pressure difference, $\varDelta U_a$, plays a role of the total stress in the consolidation by seepage force.

The effective stress due to buoyant weight of soil, σ_r', on the other hand, can be determined from the void ratio distribution in specimen. In the measurements of water content distribution, a sampled specimen was cut into slices from the top surface inch by inch. Let us denote the thickness and the void ratio of the i-th slice $\varDelta z_i$ and e_i, respectively, then the buoyant weight of that slice is given by

$$W_i=\frac{G_s-G_w}{1+e_i}\gamma_w\Delta z_i \tag{24}$$

Distribution of σ_r' can be determined by accumulating the value of W_i from the top to the bottom.

The effective consolidation stress, σ', acting at an elevation, ξ, should be given by summing up the values of σ_j' and σ_r' acting at that elevation. The measurement accuracy of σ_j' is, however, not the same order as that of σ_r'; being about $0.2\,\mathrm{kN/m^2}$ in σ_j' and $0.001\,\mathrm{kN/m^2}$ in σ_r'. Hence, for the stress level below $0.2\,\mathrm{kN/m^2}$ the summation itself has no meaning and consequently the distribution of effective consolidation stress coincides with that of σ_r' in the stress range lower than $0.2\,\mathrm{kN/m^2}$. Fig. 11 shows an example of the obtained effective consolidation stress distribution, in which a discontinuity caused by the above stated imbalance in measurement accuracies appears. Because the effective consolidation stress should varies continuously from place to place in the specimen, it is reasonable that the correct distribution should be such a smoothly continued line as plotted in Fig. 11 by the dotted line. In this case, the effective stress due to seepage force, σ_j', should be re-determined as the value which is obtained by subtracting σ_r' from the corrected value of σ'.

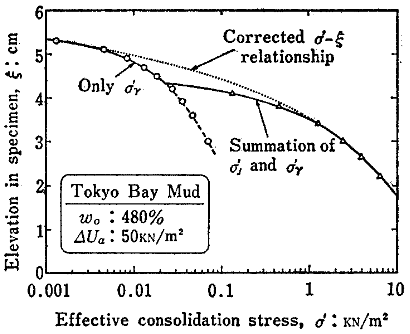


Fig. 11. Discontinuity on $\sigma'-\xi$ line caused by the imbalance of measurement accuracies and the corrected line

DETAILS OF CONSOLIDATION CONSTANTS DETERMINATION

Preparations

Preceding the determination of consolidation constants, distributions of $\log_{10}\sigma'$, $\log_{10}\sigma_j'$ and e in test specimen should be drawn as shown in Figs. 12 (a) and (b). For the next step, a series of effective stresses σ_1', σ_2', $\cdots$, σ_i', $\cdots$, σ_n', should be selected and plotted on the $\log_{10}\sigma'$-axis keeping a relationship, $\sigma'_{i+1}=2\sigma_i'$. Then determine the elevation, ξ_i, corresponding to σ_i' on the $\log_{10}\sigma'-\xi$ line, and find $(\sigma_j')_i$ acting at that elevation on the $\log_{10}\sigma_j'-\xi$ line. The value of e_i corresponding to σ_i' should lastly be found on the $e-\xi$ line as is shown in Fig. 12(b). The n-sets of the values thus obtained, $(\sigma_i', (\sigma_j')_i, \xi_i, e_i: i=1, 2, \cdots, n)$, are used to determine the consolidation constants.

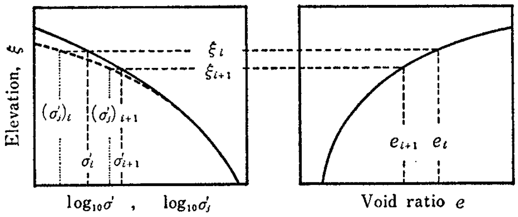
Compression Curve, Compression Index and Coefficient of Volume Compressibility

Compression curve can be determined by plotting the value, e_i, against the value of $\log_{10}\sigma_i'$. Fig. 13 shows an example of the compression curve obtained for the Tokyo Bay mud. Since compression index, C_c, is defined by the inclination of compression curve, it can be calculated by

$$C_c=\frac{e_{i+1}-e_i}{\log_{10}\sigma'_{i+1}-\log_{10}\sigma_i'} \tag{25}$$

Because the compression curve shows no linearity especially in the low effective stress range, the value of C_c changes according to the stress level. It should, therefore, be mentioned here that the value calculated by Eq. (25) fits only to the stress increment from σ_i' to σ'_{i+1}.

When the value of e_0 in Eq. (10) is replaced by e_i, $\Delta\sigma'$ is $\sigma'_{i+1}-\sigma_i'$ and Δe is $-(e_{i+1}-e_i)$. Coefficient of volume compressibility can, therefore, be calculated by the following



(a) $\sigma'-\xi$ (b) $e-\xi$

Fig. 12. Relationships among eight quantities necessary to calculate the consolidation constants: (a) relationships among σ'_i, ξ_i and $(\sigma_j')_i$; and (b) relationships between e_i and ξ_i

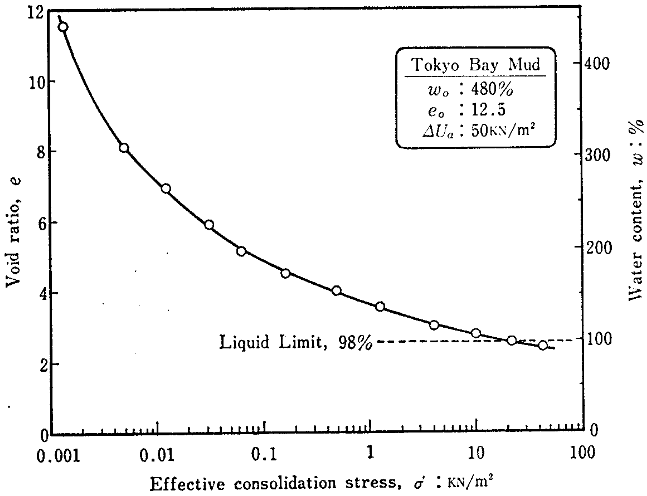


Fig. 13. An example of the compression curve determined by the hydraulic consolidation test

equation:

$$m_v=-\frac{e_{i+1}-e_i}{(1+e_i)(\sigma'_{i+1}-\sigma_i')} \tag{26}$$

This value of course fits to the stress level of $\sqrt{\sigma_i'\cdot\sigma'_{i+1}}$.

Permeability Coefficient and coefficient of Consolidation

The water head effective to the consolidation can be expressed as follows by combining Eq. (20) with Eq. (23):

$$h_e=\frac{1}{\gamma_w}(\varDelta U_a-\sigma_j') \tag{27}$$

As the distribution of σ_j' is drawn in Fig. 12(a), distribution of h_e can also be determined. Hence, averaged total head gradient between the elevation ξ_{i+1} to ξ_i is expressed by

$$i=\frac{(\sigma_j')_{i+1}-(\sigma_j')_i}{\gamma_w(\xi_{i+1}-\xi_i)} \tag{28}$$

Because the velocity of flow passing through the central area, v, is constant throughout the specimen, coefficient of permeability for the stress level of $\sqrt{\sigma_i'\cdot\sigma'_{i+1}}$ can be determined followingly:

$$k=-\gamma_w v\frac{\xi_{i+1}-\xi_i}{(\sigma_j')_{i+1}-(\sigma_j')_i} \tag{29}$$

As coefficient of consolidation is defined by Eq. (12), it can be expressed as follows by combining Eq. (26) with Eq. (29):

$$c_v=v(\xi_{i+1}-\xi_i)\frac{1+e_i}{e_{i+1}-e_i}\frac{\sigma'_{i+1}-\sigma_i'}{(\sigma_j')_{i+1}-(\sigma_j')_i} \tag{30}$$

This value should of course fit to the stress level, $\sqrt{\sigma'_i\cdot\sigma'_{i+1}}$.

In Fig. 14, the values of m_v, k and c_v thus obtained for the Tokyo Bay mud are plotted against the effective consolidation stress of $\sqrt{\sigma_i'\cdot\sigma'_{i+1}}$ on log-log scale. As shown in Figs. 13 and 14, compression curve and consolidation censtants are determined in the stress range from 0.01 kN/m² to 50 kN/m². This stress range is sufficient for usual analyses of self-weight consolidation. Consolidation constants for the higher stress level than 50 kN/m² can of course be obtained by applying the higher pressure difference than 50 kN/m² on

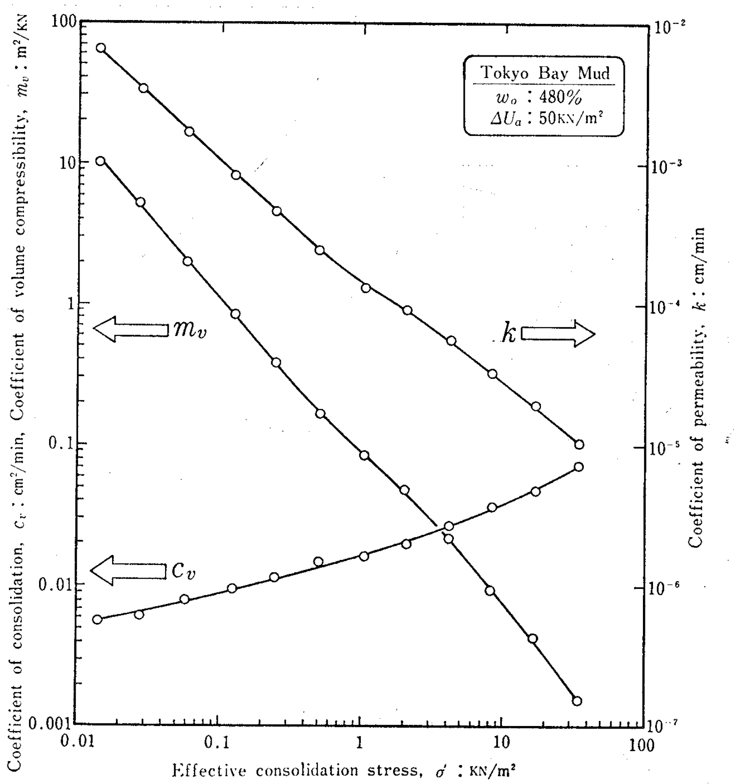


Fig. 14. Examples of the consolidation constants determined by the hydraulic consolidation test

the specimen, because the maximum consolidation stress acting in the specimen accords with the pressure difference applied.

A COMMENT ON UTILIZATION OF TEST RESULTS

Settlement of a landfill can be predicted by use of the self-weight consolidation theory such as Mikasa's one. In this case, consolidation constants in the basic equation should be expressed as functions of void ratio. Of course, they are obtained by the test presently proposed. Numerical integration of that equation, however, cannot be carried out without further two informations. One is boundary conditions; i.e., drained conditions, which can be decided without much troubles. The other one is initial conditions, probably the most perplexing problem in the settlement analyses of landfills. The best one now recommendable as initial conditions may be the layer thickness and vertical distributions of void ratio in the landfill at the completion of filling works, because the settlement after the filling works is of great concern to engineers. A detailed treatment of this initial condition in the computational technics is examined by Takada (1979).

CONCLUSIONS

In the present study, a new method of consolidation test different from the conventional one was proposed. It is based on the principle of consolidation by seepage force. Its distinctive features can be summarized as follows:

(1) Artificial sediments prepared in a consolidometer is used as a test specimen.
(2) Seepage force consolidates the specimen.
(3) All of the consolidation constants are determined on the basis of measurements

carried out in the state of steady seepage flow after the consolidation.

(4) There is no need to employ any consolidation theory in determining the consolidation constants.

With use of the test apparatus developed in this study, consolidation constants were determined in the wide range of consolidation stress higher than 0.01 kN/m^2. It may be concluded that this consolidation test is effectively used to obtain the consolidation constants and compression curves for analyses of the self-weight consolidation process of landfills.

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NOTATION

A = cross sectional area of specimen
C_c = compression index
c_v = coefficient of consolidation
e = void ratio
G_s, G_w = specific gravities of soil particles and water
h, h_e = water heads; total and effective to consolidation
i = total water head gradient
j = seepage force per unit soil volume
k = coefficient of permeability
L = length of pickup
m_v = coefficient of volume compressibility
r = reading of interface elevation in burrette
t = time
U_H, U_L = air pressure in high and low pressure head tank
u, u', u_e = pore water pressures; total, excluding hydrostatic pressure and effective to consolidation
v = velocity of flow passing through specimen
W = weight of sliced specimen per unit area
w_m, w = measured and corrected water content
Z = maximum elevation head of the system
z = space coordinate measured downward
α_1, α_2 = calibration constants of pressure transducer
β = salt content of water
γ_t, γ_w = unit weight of soil and water
γ' = buoyant unit weight of soil
ΔU_a = pressure difference applied on specimen
ε = vertical strain
μ_0, μ, μ_f = outputs of pressure transducer
ξ = elevation from bottom of specimen

σ' = effective consolidation stress
σ_J', σ_s' = effective stress due to seepage force and buoyant weight of soil

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