APPLICABILITY OF HYDRAULIC CONSOLIDATION TEST FOR VERY SOFT CLAYEY SOILS

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ABSTRACT

A consolidation test method using seepage force was previously proposed to obtain consolidation constants for very soft clayey soils subjected to very low stress. Its applicability to practice is herein examined.

The main results obtained are as follows: (1) Mikasa's large strain theory well interprets the process of seepage force consolidation, and a settlement-time curve obtained from the proposed test is in good agreement with the one calculated by using the theory and the constants obtained from that test; (2) Consolidation constants obtained from the proposed tests are in good agreement with the ones determined by the traditional oedometer tests; (3) Relationships between the constants and plasticity index are quite similar to the ones obtained from the constant rate of strain consolidation tests conducted by Umehara et al.

In summary, the effectiveness of the proposed test method is emphasized with several precautions in operating the test.

Key words: cohesive soil, consolidation test, seepage, slurry, test procedure (IGC : D5)

INTRODUCTION

Clayey landfill formed in water areas through the process of sedimentation is initially very soft. Its self-weight consolidation, therefore, starts from a very high void ratio and very low stress state, and it generally undergoes a large strain during consolidation. In order to predict the process of such large-strain consolidation, several theories have been proposed (Mikasa, 1963;

Gibson, England and Hussey, 1967; Monte and Krizek, 1976), and two consolidation test methods suitable for soft soils in a slurry state have been developed; one is a constant rate of strain consolidation test (Umehara and Zen, 1980, 1982), and another is a test method using seepage force, which has been called a "hydraulic consolidation test" (Imai, 1979).

In the hydraulic consolidation test, a specimen is formed through the process of

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sedimentation in a consolidometer. The specimen then undergoes high and low water pressures on the top and the bottom surfaces of the specimen, respectively. Therefore, the specimen is consolidated by the seepage force induced by a one-dimensional unsteady seepage flow. Soil at any height in the specimen, therefore, is subjected to increasing stress during consolidation. After the end of consolidation, however, the seepage flow becomes steady, and consolidation stress is constant at each height varying from zero at the top to some value at the bottom. For this steady state flow rate, distribution of pore water pressure and distribution of water content are measured; from these measurements all consolidation constants can be calculated from their definitions.

The hydraulic consolidation test (HCT) thus performed is different from the standard oedometer test (SOT) in several points. 1) Every specimen for an HCT is always in a normally consolidated state, while a specimen for an SOT is not necessarily so. 2) The relationships between consolidation constants and consolidation stress are obtained by step-by-step loadings in the SOT, but by one-time loading in the HCT. 3) In the SOT the coefficient of consolidation, c_v , is determined by fitting a theoretical consolidation process to a measured one, but in the HCT, by applying the definition of c_v to the final steady state of consolidation.

In order to confirm the applicability of the HCT in practice, therefore, examination must be made to see whether consolidation constants determined by the HCT are reasonable or not. For this purpose three types of examination have been carried out, and their results will be summarized in this paper. In the first examination, theoretical settlement-time curves calculated using large-strain theory are fitted to the measured curve in order to find the most reasonable value of c_v , which is then compared with the c_v -value determined in the HCT itself. In the second examination, consolidation constants obtained in the HCT and SOT are compared with each other. And in the final

examination, consolidation constants obtained in the HCT are compared with the constants which were obtained in the constant rate of strain consolidation tests performed by Umehara and Zen (1982).

HYDRAULIC CONSOLIDATION TEST METHOD

Apparatus

Fig. 1 is a schematic diagram of the HCT-apparatus which was initially fabricated by the first author (Imai, 1979).

In the consolidometer, a specimen is formed through the process of sedimentation from a dilute soil-water mixture. Constant water pressures acting at the top and bottom of the specimen are supplied by the air pressures acting in the head tanks. A constant water table in both head tanks is achieved with a winch. The water pressure at the bottom is applied as back pressure.

The seepage flow through the specimen is separated into two parts at the bottom of the specimen; the flow through the central area of the cross section and the flow through the circumferential area. After seepage force consolidation has completed, the side surface of the specimen and the inside surface of the consolidometer part and a small gap is formed between their surfaces. Therefore the circumferential flow then increases. But the flow disturbance due

Fig. 1. A schematic diagram of hydraulic consolidation test apparatus

Photo. 1. Hydraulic consolidation test apparatus with three consolidometers

to the gap formation gives no influence on the inner flow, which yields the correct value of flow rate in the specimen. The rate is measured by the double-tube flow meter.

After seepage force consolidation has completed, the values of pore water pressure in the specimen is measured. It is done by inserting into the specimen a hollow-piped needle having a small opening at its head and a pressure transducer at its tail. The distribution of pressure is obtained by penetrating the needle and changing the elevation of the head opening.

Immediately after the seepage force has been removed, a continuous soil column is sampled from the specimen by vertically and downward penetrating a thin wall sampler of 35 mm diameter. The sample is then pushed out from the sampler 2 mm by 2 mm to obtain slices of 2 mm thickness. Then water content value of each slice is measured to yield the distribution of void ratio in the specimen.

These measurement methods are precisely described in the previous paper (Imai, 1979).

Photo.1 shows the apparatus which has produced most of the data presented in this paper. It has three consolidometers. The water table in both head tanks can be automatically adjusted.

Sample Preparation

Most dredging and landfilling work in Japan is carried out in sea-water areas using the pump system. In this case, the very soft cohesive soil which is formed in the dyked pond generally has the following peculiarities: 1) The soil rarely includes coarse particles because particle-size sorting takes place during soil transportation in the pond. 2) The soil is formed from dilute soil-water mixtures. 3) The soil is usually an aggregate of flocs which have been formed in the mixture and then settled.

In order to simulate these peculiarities, the sand fraction is first taken off from the dredged soil. Then the remainder is mixed with sea water, and the water content of the mixture is adjusted to the desired value. The soil particles in the mixture poured into the consolidometer soon flocculate, and the many flocs thus formed settle in the aggregate. After the settling of all the flocs is confirmed, the sediment is used as a specimen.

Because both mixtures and specimens show high water content, ordinarily measured water content values are corrected to remove the effect of salinity as follows (Imai, Tsuruya and Yano, 1979):

$$w = \frac{1 + \beta}{1 - \beta w_m} w_m \quad (1)$$

Where w is the corrected water content, w_m is the measured water content, and β is the mass ratio of the salt to the pure water contained in the water phase.

Test Procedures

Before the start of consolidation, the flow routes of the apparatus are wholly filled with the sea water used for specimen preparation. When high and low water pressures are applied at the top and bottom of the specimen, respectively, consolidation by seepage force begins. As shown in Fig.2, the thickness of the specimen decreases with time, finally approaching a nearly constant value. The flow rate also shows a similar tendency.

After the end of consolidation three kinds of measurement are carried out within a

Fig. 2. A typical example showing the process of seepage force consolidation

short time; flow rate v , distribution of pore water pressure $u(z)$, and distribution of water content $w(z)$, where z is measured downward from the top of the specimen.

Calculation of Consolidation Constants

Fig. 3 shows the flow chart to calculate consolidation constants. First of all, the distribution of void ratio $e(z)$ is calculated from the $w(z)$ data, full saturation of the specimen being assumed. Then, based on the $u(z)$ data the distribution of consolidation stress $\sigma_v'(z)$ and of hydraulic gradient $i(z)$ are calculated.

When connecting $e(z)$ with $\sigma_v'(z)$, a relationship between e and σ_v' , i.e. a compression curve, is obtained. Therefore, the compression index $C_c(\sigma_v')$ and the coefficient

Fig. 3. Flow chart showing the determination method of consolidation constants

of volume compressibility $m_v(\sigma_v')$ are determined from their definitions; $C_c = -\Delta e / \Delta \log_{10} \sigma_v'$ and $m_v = -\Delta e / (1+e) \Delta \sigma_v'$.

Because the value of v is the same everywhere in the specimen, the distribution of the permeability coefficient $k(z)$ is calculated from the $i(z)$ data using Darcy's law. When connecting $k(z)$ with $\sigma_v'(z)$, a relationship between k and σ_v' is obtained. Therefore, the coefficient of consolidation $c_v(\sigma_v')$ is determined from the definition; $c_v = k / \gamma_w m_v$, where γ_w is the density of water.

Detailed calculation methods are described in the previous paper (Imai, 1979).

THEORETICAL EXAMINATION OF HYDRAULIC CONSOLIDATION TEST

Consolidation Theory Used

In order to theoretically compute the process of hydraulic consolidation, a consolidation theory taking into account large-strain effects should be used. In the present analysis Mikasa's theory, which considers the effect of a thickness change (1963), is adopted. The basic equation is:

$$\frac{\partial \zeta}{\partial t} = c_v \zeta^2 \frac{\partial^2 \zeta}{\partial z_0^2} \quad (2)$$

where z_0 is the original coordinate measured downward from the top of the specimen, ζ is the consolidation ratio expressed by $(1+e_0)/(1+e)$, in which e_0 is the initial void ratio of the specimen and e is the void ratio at z_0 and time t . Eq. (2) assumes that the value of c_v is constant throughout consolidation.

Eq. (2) is analytically unsolvable. Therefore, it is replaced by a finite difference equation as follows:

$$\begin{aligned} \zeta(z_0, t + \Delta t) = & \frac{n^2 \Delta T}{4} \zeta^2(z_0, t) \\ & \times \{ \zeta(z_0 + \Delta z, t) - 2\zeta(z_0, t) \\ & + \zeta(z_0 - \Delta z, t) \} + \zeta(z_0, t) \end{aligned} \quad (3)$$

where $\Delta z_0 = H_0/n$, $c_v \Delta t / (H_0/2)^2 = \Delta T$, and H_0 is the initial thickness of the specimen. In the present calculation $n=10$ and $\Delta T=0.001$ were adopted.

Initial Conditions and Boundary Conditions

The initial conditions required to solve Eq. (3) are the values of e_0 and H_0 ; the value of e_0 is assumed to be the same everywhere in the specimen. In the present study, a duplicate specimen was made preceding the real test and its water content distribution was measured at the predetermined start time of consolidation. Then the averaged value of the distributed void ratios was taken as the e_0 -value. Of course $\zeta(z_0, 0)=1$.

As boundary conditions, the values of the void ratio at the top and bottom of the specimen are required. Because no consolidation stress acts at the top surface throughout consolidation, the void ratio at the top surface is constant at the initial value e_0 , which should be the same as the value determined above. The consolidation stress acting at the bottom is the sum of the submerged weight of the specimen per unit area and the water-pressure difference ΔP between the top and bottom. Because this stress is also constant throughout consolidation, the void ratio at the bottom, e_f is also a constant value, determined from the real test after the end of consolidation.

Calculation of Settlement-Time Relation

Because the distribution of ζ -values at time t , $\zeta(z_0, t)$ is calculated from Eq. (3), the values of $e(z_0, t)$ are obtained by the relationship $e=(1+e_0)/\zeta-1$. Using the $e(z_0, t)$ data thus obtained, the settlement of the top surface, $S(t)$ is calculated by the following equation:

$$S(t) = \int_0^{H_0} \frac{e_0 - e(z_0, t)}{1 + e_0} dz_0 \quad (4)$$

Consolidation Tests Performed

In order to fit a computed settlement-time relation to the measured one, a consolidation test must be performed and data required for the calculation, H_0 , e_0 and e_f , must be obtained. Table 1 lists the four tests performed for this purpose. The material used was Sendai bay mud(A).

Case I and II specimens were formed from a mixture having a water content of

Table 1. Calculation conditions used for curve fitting calculations

Case	Water content of mixture w (%)	Pressure difference applied ΔP (kN/m ²)	Specimen initial height H_0 (cm)	Initial void ratio e_0	Bottom void ratio e_f
I	1000	50	16.6	18	2.55
II	1000	40	12.9	18	2.65
III	700	30	19.0	14.7	2.85
IV	700	20	14.5	15	2.95

1000 percent. The soil particles in the mixture soon flocculated and the flocs thus formed settled in the aggregate with a constant rate of settling. When all the flocs had settled, the sediment thus formed started to undergo self-weight consolidation and the settling rate abruptly dropped down (Imai, 1980, 1981). When this abrupt drop of the settling rate was confirmed, seepage force was applied on the sediment and seepage force-consolidation was initiated.

At this start time of consolidation, a void ratio distribution in the sediment obtained by the above stated measuring method was 18.5 at the top surface and 17.5 at the

Fig. 4. Comparisons between computed and measured consolidation curves (Sendai bay mud A)

bottom. Therefore, the initial void ratio, $e_0=18$ was used for theoretical calculation.

Because the mixture's water content for Case III and IV was lower than for Case I and II, a far longer flocculation time, about one day, was required and the formation of flocs was not clearly observed. To obtain a completely flocculated specimen, 2 days were needed until the start of seepage force consolidation. At this start time, both the specimens showed a fairly uniform distribution of void ratio; 14.7 for Case III and 15 for Case IV. These values were used in theoretical calculation as the initial value.

The measured settlement-time relationships are plotted by the round circles on Fig. 4. Because Sendai bay mud shows high plasticity (see Table 3), 2 to 3 days were required for the completion of consolidation.

Theoretical Curve of Best Fit

Theoretical settling-time relationships were computed for each test case, several c_v -values being assumed. Among them the curve which best fitted the measured results was found. The unbroken lines in Fig. 4 show the computed curves thus selected, which are in good agreement with the measured results. This agreement shows that the process of seepage force consolidation can be well interpreted by Mikasa's theory incorporating large-strain effect.

Comparison of c_v -Values

In order to confirm the reasonableness of the HCT, it should be confirmed that the c_v -value used in the calculation of the best fit curve coincides with the c_v^* -value obtained from the HCT itself. In Fig. 4, both values are compared for each case. Because the c_v -value is dependent on the value of σ_v' , the measured c_v^* -values at $\sigma_v'=\Delta P/2$ were given in Fig. 4.

Although the coincidence is not complete, the measured values and the values of best fit are close to each other. It may be concluded, therefore, that the HCT method is theoretically reasonable.

Fig. 5. Comparison of the final distribution of void ratio

Comparison of the Final State

Fig. 5 shows void ratio distributions in the specimen; the ordinate is the current depth z normalized by the current thickness of the specimen at time T . Both the computed lines and the measured points were obtained for Case IV in Table 1. In this case, the consolidation was nearly completed in time $T=0.53$.

The measured distribution of e , obtained after completion of consolidation, shows a similar tendency to the theoretical curve for $T=0.5$. Similar results were obtained for the other cases; in this respect, the HCT method may be considered a reasonable one.

The measured void ratio, however, is larger than the computed one; this discrepancy is the same as for the final difference between the measured and computed settlement. Fig. 5 shows that the discrepancy is larger for smaller values of z/H . This means that the smaller the acting stress, the larger the discrepancy. Although the real cause was not clarified, it may be that the decrease in void ratio due to secondary compression during consolidation is larger for smaller stresses under conditions of very low stress.

HCT-RESULTS COMPARED WITH SOT-RESULTS

Specimen Preparation for the SOT

In order to compare the consolidation constants obtained by the HCT with those obtained by the SOT, specimens for the SOT must be prepared to satisfy the following conditions: 1) The specimen must be in the normally consolidated state. 2) The specimen must be formed under the same conditions as in the HCT from the view point of material, water content of mixture, sedimentation type, and so on.

To satisfy these requirements, a sedimentation tube having the same inside diameter as the oedometer, 60 mm, was used. A mixture having a predetermined water content was poured into the tube. After settling had completed, a piston was inserted in the tube. Then the sediment was consolidated with double drainage by loading a weight of 5 kN/m² on the piston. After the end of preconsolidation, the consolidated soft specimen was pushed into the oedometer, which had been connected to the sedimentation tube.

In order to make a specimen with a thickness of about 2 cm, an adequate amount of mixture was poured into the tube.

Calculation of Consolidation Constants for the SOT

Consolidation pressures of 5, 10, 20, 40 kN/m², and so on, were applied step by step on the specimens having a preconsolidation pressure of 5 kN/m². The specimen undergoing an initial pressure of 5 kN/m², therefore, was in an overconsolidated state; the constants obtained for that loading step were omitted. The constants obtained for the other steps were considered to be those for the middle stress values between two successive consolidation pressures.

Loading duration was 24 hours for each step. Compression curves, c_c -values and m_v -values were obtained from the settlement data measured at the end of 24 hours consolidation. The values of c_v were determined

Table 2. Test cases performed to compare consolidation constants

Material used	Plasticity index I_p	HCT		SOT
		Water content of mixture w (%)	Pressure difference applied ΔP (kN/m ²)	Water content of mixture w (%)
Sendai bay mud (B)	92	2260	50	2120
		510	50	500
Tokyo bay mud	65	1000	50	1030
		480	50	550
Kaolin	15	1050	50	1030
		440	50	450

by the $\sqrt{t}$ -method.

Test Cases Performed

The test cases performed are listed in Table 2. Three kinds of material with different plasticity were used. Specimens were prepared from mixtures with high water content, about 1000 or 2000 percent, and from mixtures with low water content, about 500 percent.

Fig. 6. Comparisons of compression curves

Comparison of Compression Curves

Fig. 6 shows the compression curves obtained from the HCT compared with the results of the SOT. The results for the specimens prepared from the thinner mixtures and the thicker mixtures, respectively, are shown in Figs. 6 (a) and (b). The ordinate scale for Sendai bay mud is given on the right-hand side.

Because the stress ranges covered by the HCT and SOT are different from each other, a doubled stress range is narrow and limited to 7-50 kN/m². In this limited stress range, however, the compression curves of the HCT and SOT are in good agreement, and both curves smoothly connect with each other. This can be taken as evidence supporting the usability of the HCT.

Comparison of m_v -values

Because m_v -values are basically obtained from a compression curve, it is natural that the m_v - σ_v' relationship obtained from the HCT should smoothly connect with that from the SOT. This natural expectation can be confirmed by the results shown in Fig. 7.

Fig. 7. Comparisons of the coefficient of volume compressibility

Fig. 8. Comparisons of the coefficient of consolidation

Comparison of c_v -values

The method of determining c_v -values is quite different between the HCT and SOT. But if c_v -values obtained from the HCT were in agreement with the ones from the SOT, the method of the HCT can be used for the testing purpose.

The results shown in Fig. 8 confirm satisfactory agreement of the c_v -values from the HCT with those from the SOT. From this result, including the comparison results regarding compression curves and m_v -values, it may be said that the hydraulic consolidation test produces data of nearly the same quality as the SOT.

CHARACTERISTICS OF CONSOLIDATION CONSTANTS UNDER VERY LOW STRESS

Materials Used

In order to determine the characteristics of consolidation constants under very low stresses and to compare them with data obtained by other investigators, both the HCT and SOT were performed for many materials. These were, except for kaolin, sampled from

Table 3. Physical properties of the materials used

Sampling Site	Symbol	Specific gravity G_s	Liquid limit w_L (%)	Plastic limit w_p (%)	Plasticity index I_p	Finer than 5 μ (%)
Befu port	☒	2.522	150	56	94	57
Hakata port	●	2.690	78	29	49	41
Hatsukaichi port	●	2.609	139	40	99	54
Hiroshima bay	☒	2.601	123	43	80	52
Off Kanda (A)	☒	2.640	100	29	71	44
(B)	☒	2.591	104	29	75	45
(C)	☒	2.596	103	20	74	43
Kaolin	△	2.673	45	30	15	68
Kawanoishi port	▽	2.818	47	24	23	32
Nagoya port (A)	⊖	2.301	89	31	58	22
(B)	⊖	2.211	131	40	91	37
Nakagusuku port	☒	2.715	69	26	43	66
Osaka bay	△	2.712	102	31	71	72
Otaru port (A)	☐	2.676	100	36	64	51
(B)	☐	2.665	95	35	60	56
Sendai bay (A)	○	2.599	119	34	85	54
(B)	○	2.582	124	32	92	62
Tokyo bay	⊕	2.661	96	35	61	59

Fig. 9. Relationship between C_c and plasticity index

many dredging and landfilling sites in Japan. Their physical properties and symbols used in Fig. 9 to Fig. 13 are listed in Table 3.

Compression Index

As shown in Fig. 6, compression curves under very low stresses are in general not straight but concave. The compression index C_c for any compression curve, therefore, is not a constant, but decreases with increasing stress. In this paper, however, the averaged C_c -value for the stress range 1–100 kN/m² was calculated for each material, and the value was plotted against its plasticity index. Fig. 9 shows the results thus obtained, including similar data obtained by Umehara and Zen (1982) in the same stress range.

The broken lines in Fig. 9 are the upper and lower limits of data obtained. Although the discrepancy between the two lines is fairly large, the well known general tendency that the more plastic soil has the higher compression index can be confirmed. Furthermore, good agreement between the results from the HCT and from the constant rate of strain consolidation test can be seen.

Fig. 10. Relationship between m_v and consolidation stress

Coefficient of Volume Compressibility

Fig. 10 shows the relationship between m_v and σ_v' . Because the data obtained fell in a narrow band, only the area occupied

Fig. 11. Relationship between m_v and plasticity index

Fig. 12. Relationship between c_v and plasticity index

by the data is enclosed by unbroken lines. As far as the data shown in this figure is concerned, the relationship between m_v and σ_v' may be crudely represented by the equation $m_v = 0.15 / \sigma_v'$, which is represented by the broken line in Fig. 10.

The values of m_v under the same σ_v' -value are replotted in Fig. 11, plasticity index being taken as the abscissa. While the data presently obtained are nearly independent of plasticity index, the SOT-data for many marine cohesive soils obtained by Ogawa and Matsumoto (1978) show a strong dependency on plasticity index under the range below 60.

Based on these findings, it can be said that, for soils having I_p -values larger than 60, the relationship between m_v and σ_v' is uniquely determined irrespective of plasticity index, and may be crudely represented by the equation $m_v = 0.15 / \sigma_v'$.

Coefficient of Consolidation

As shown in Fig. 8, the value of c_v is considerably affected by the plasticity index and stress level, too.

Fig. 12 shows the relationship between c_v and I_p under stresses of 1 and 10 kN/m². While the relationship appears to be independent of I_p for I_p -values larger than 60, a soil having a lower plasticity shows a larger c_v -value for I_p -values smaller than 60.

Fig. 13. Relationship between c_v -ratio and plasticity index

Fig. 13 shows the relationship between c_v -ratio and I_p . The c_v -ratio is the c_v -value at $\sigma_v' = 10 \text{ kN/m}^2$ divided by that at $\sigma_v' = 1 \text{ kN/m}^2$, the larger the c_v -ratio, the larger the stress dependency of c_v . The results shown in Fig. 13 indicate that the stress dependency of c_v is more remarkable in soil having a lower plasticity, and that the variation of c_v with stress becomes smaller in soil having a higher plasticity index.

From the results shown in Fig. 9 to Fig. 13, it can be concluded that the results from the HCT and from other investigators are in good agreement, and therefore that the hydraulic consolidation test can be effectively used in practice.

PRECAUTIONS IN TEST OPERATION

In order to obtain good data from the HCT, several precautions should be taken especially in sample preparation, test operation, and so on. These precautions will be presented based on the experience obtained to date.

Problems of Particle-Size Sorting

As stated before, a specimen for an HCT should be formed from a mixture prepared after taking off the sand fraction from dredged soil. If the sand fraction is not removed or when mixture with a very high water content is used, particle-size sorting occurs during settling and a sand rich part

Fig. 14. Effect of particle-size sorting on compression curve

is formed near the specimen bottom.

In this case, the compression curve obtained may be influenced by sorting as shown in Fig. 14. Void ratio values for the sand rich part, where σ_v' is larger than 12 kN/m^2 in Fig. 14, are abnormally lower than those expected from the $e-\sigma_v'$ relation for the upper part.

The upper part of a specimen includes very few sand particles; it is very uniform from the viewpoint of gradation as long as the water content of the mixture is not too high (Imai, 1981). This part of the specimen, therefore, is considered representative of the sediment which is formed without sand particles in an actual landfill. Therefore, consolidation constants obtained for the sand rich part of a specimen should be set aside, and those obtained for the upper uniform part may be used.

Water Content of Mixture

When the water content of the mixture is not too high and not too low, the soil particles in the mixture flocculate into discrete flocs which settle in the aggregate; this is called "zone settling", when the water

Fig. 15. Effects of water content of mixture on consolidation constants

content is low, on the other hand, discrete flocs observable with the naked eye are not formed; the mixture consolidates under its own weight after the end of flocculation; this is called "consolidation settling" (Imai, 1980). This difference in flocculation degree may necessarily cause a difference in the fabric of the sediment formed and therefore in the characteristics of the consolidation constants of the sediment.

Fig.15(a) shows the effect of the water content of the mixture on the compression curves. The mixture with $w=2000$ and 1000 percent showed typical zone settling, and the 500 percent mixture showed consolidation settling. The specimens formed from the former mixtures show similar compression characteristic to each other, but show a higher void ratio and a higher compressibility than does the sediment formed from the latter mixture. This result shows the considerable effect of settling type or water content of the mixture on the compression characteristics of sediment. Therefore, the water content should be carefully selected in accordance with conditions expected in the actual landfilling site under consideration.

Fig.15(b) shows the effect of the water

Fig. 16. Effects of water pressure difference on consolidation constants

content of the mixture on c_v -values. This effect is also marked as with the compression characteristics.

Water Pressure Difference Applied

Fig.16 shows the effects of water pressure difference ΔP on the consolidation constants. The result of the test for $\Delta P=50 \text{ kN/m}^2$ and that for $\Delta P=40 \text{ kN/m}^2$ are in very good agreement. That is, the magnitude of the water pressure difference has little influence on the result as long as a water pressure difference of about 50 kN/m^2 is applied to a specimen.

When too high or too low a pressure difference is applied, however, good results can not always be obtained. This may be caused by large imbalances in the accuracy of measurement.

Measuring Time

In the hydraulic consolidation test, measurement is carried out after the end of seepage force consolidation. But no measur-

Fig. 17. Effects of measuring time on consolidation constants

ing time has been so far laid down in this paper.

Fig. 17(a) shows the consolidation process for Tokyo bay mud. The first measurement was carried out at $t=1300$ minutes when seepage force consolidation was just completed, and the second one at $t=2830$ minutes. Up to this time only a small amount of secondary compression had taken place.

As shown in Figs. 17(b) and (c), however, the consolidation constants obtained at both measuring times are in very good agreement. This agreement must result from the small amount of secondary compression which occurred during the two measuring periods. Therefore, it may be concluded that measurement may be performed whenever consolidation has completed and as long as the amount of secondary compression occurring thereafter is small.

CONCLUSIONS

The applicability of the hydraulic consolidation test in practice was examined from various standpoints, and it was confirmed by the following result obtained:

(1) The process of seepage force consolidation can be well explained by Mikasa's theory incorporating large-strain effects. The theoretical c_v -value best explaining the measured process and the c_v -value measured in the HCT itself are in good agreement.

(2) The values of the consolidation constants obtained in the HCT and those obtained by the SOT are in good agreement for a stress range of 7-50 kN/m².

(3) The relationships between consolidation constants and plasticity index are about the same for the results of the HCT and the results of the constant rate of strain consolidation test performed by Umehara and Zen.

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NOTATION

- e_0 =initial void ratio of a specimen
- e_f =void ratio at the specimen bottom
- H_0 =initial thickness of the specimen
- n =mesh number
- ΔP =water pressure difference applied to the specimen
- S =settlement of the specimen
- T =time factor
- v =rate of seepage flow
- w, w_m =correct water content, measured water content
- z, z_0 =current depth from the specimen top, original coordinate of Z
- β =salt content of water
- ζ =consolidation ratio, $(1+e_0)/(1+e)$
- σ_v' =effective vertical stress in the specimen

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