A SIMPLIFIED METHOD OF ESTIMATING CONSTRUCTION PORE PRESSURES IN EARTH DAMS

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ABSTRACT

A simplified method of evaluating pore pressure behaviors of earth dams during construction was proposed in the present paper. The basic scheme of the method is similar to that presented by Eisenstein et al. (1976), but some simplifications were introduced in the part of consolidation analysis. The main advantages of the method are to estimate generated pore pressures from the results of conventional oedometer tests and to evaluate pore pressure dissipation by using the relationship between the degree of consolidation \( U \) and the time factor \( T \) in one-dimensional consolidation. Concerning the latter, it is recommended here that \( U-T \) relationships be composed from pore pressure measurements in the field, because it is not necessarily sufficient to represent such field consolidation conditions as anisotropic permeability and the difference in drainage systems through laboratory tests. In the comparisons between computed and observed results for three representative earth dams, it was noticed that the present analytical approach on the whole showed good correspondence with pore pressure behaviors in actual dams, offering sufficient accuracy for practical purposes.

Key words: case history, consolidation, dam, design, earthfill, finite element method, pore pressure (IGC: E.0/H.4)

INTRODUCTION

In the evaluation of stress-deformation behaviors and the stability of slopes of earth structures such as earth dams and levees constructed of cohesive soils, analyses have been made assuming that these embankments behave in unconsolidated and undrained conditions during construction. This assumption may be reasonable for comparatively small embankments in a sense of a safe-side design. In recent high earth dams, however, consolidation is expected to proceed to a considerable amount during a long-term construction period, so that its effect may no longer be negligible. From another point of view, it is noticed particularly in the case where highly compressible soils with high water content are used as construction materials that the rate of placement should be controlled in order to reduce generated pore pressures to a given extent and to

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expect higher soil strength due to consolidation during construction. Unconsolidated and undrained analyses are therefore no longer applicable in these situations, because set-up pore pressures and deformations developed during construction tend to be overestimated, resulting in the underestimation of the stability of embankments.

Analytical approaches to the estimation of pore pressure behaviors in earth dams have been proposed by Gibson (1958) and Sawada and Toriyama (1967) on the basis of the Terzaghi theory of one-dimensional consolidation. Recent development of finite element analysis, however, has made it feasible to evaluate both pore pressure and deformation behaviors simultaneously by combining an effective stress-deformation analysis for porous skeleton and a seepage analysis for pore fluid. Some consolidation problems on clay foundations have been solved by Sandhu and Wilson (1969), Yokoo et al. (1971) and Hwang et al. (1972), treating them as coupled problems by use of finite element formulations of the Biot theory of consolidation. Lewis et al. (1976) later refined these approaches by incorporating nonlinear material properties of soils.

Concerning consolidation behaviors of earth dams during construction, Cavounidis and Höeg (1977) have presented both linear and approximate nonlinear coupling approaches for fully saturated fills. Chang and Duncan (1977) and Nakagawa et al. (1981) have also developed some refinements for compacted partially saturated fills by incorporating compressibility of pore fluid. In view of practical applications, however, these comprehensive coupling approaches have some essential difficulties in their complicated analytical procedures and in determining material properties associated. In order to avoid these deficiencies, Eisenstein et al. (1976) have proposed a simplified method of consolidation analysis which evaluates generation and dissipation of pore pressures and resulting consolidation deformations in earth dams by repeating a finite element analysis of pore pressure dissipation and that of stress-deformation alternately at each time step and construction stage. It was later shown (Eisenstein and law, 1977) that this analytical procedure is not so inferior to rigorous coupling approaches, offering reasonable accuracy in practical applications.

The method proposed by Eisenstein et al. is considered to be one of practically useful methods of consolidation analysis, though it still requires some computational efforts in solving the differential equation of consolidation in every construction stage. In the present paper, a more practical and convenient method of estimating construction pore pressures is presented by introducing some additional simplifications to the consolidation process of calculation involved in the prescribed method. The main advantages of the method are to estimate generated pore pressures from the results of conventional oedometer tests and to evaluate pore pressure dissipation on the basis of the results from pore pressure measurements in the field.

**ANALYTICAL PROCEDURES**

*Finite Element Consolidation Analysis*

The computational scheme of the analysis presented by Eisenstein et al. (1976) for calculating consolidation deformations in earth dams is illustrated in Fig. 1(a). As is done in an ordinary finite element stress-deformation analysis, an embankment is idealized in several horizontal layers to simulate actual construction process. The procedures for each stage of construction are as follows:

1. A total stress analysis is performed for a new placement load to obtain total stress increments $\Delta \sigma$ and undrained (immediate) deformation increments $\Delta \epsilon_u$. Modulus of elasticity $E$ and Poisson's ratio $\nu$ used in the analysis are those with respect to total stresses.
2. Incremental pore pressures $\Delta \epsilon_{pu}$ generated due to the change in total maximum principal stresses $\Delta \sigma_1$ can be calculated for a given value of the pore pressure coefficient $B$. The value of $B$ can be changed depending
on the state of stresses and the degree of consolidation during construction process.

(3) Residual pore pressures $\Delta u_t$ which correspond to $\Delta u_0$ are subsequently determined at each time interval $\Delta t$ from a consolidation analysis.

(4) Assuming that total stresses are constant during consolidation, effective stress increments $\Delta \sigma_i'$ are represented at any time by $\Delta \sigma_i' = \Delta \sigma - \Delta u_t$.

(5) Converting effective stress changes into equivalent nodal forces, an effective stress analysis is performed to obtain consolidation deformations $\Delta v_i$. Elastic moduli with respect to effective stresses, $E'$ and $\nu'$, are used in this stage. Total deformations in the embankment are given at any time by the sum of the immediate and consolidation deformations, $\Delta v_i$ and $\Delta v'_i$.

In the above computations, material properties to be known are the unit weight of soil $\gamma_t$, the pore pressure coefficient $B$, the coefficient of consolidation $c_v$ and elastic moduli with respect to both total and effective stresses, $(E, \nu)$ and $(E', \nu')$. Considering one-dimensional and laterally confined compression states which are approximately assumed within the central core of a dam, all of these parameters may reasonably be determined from oedometer tests with pore pressure measurements.

In the present study, though the basic scheme of the analysis is almost the same as mentioned above, following simplifications are attempted to introduce in the part of consolidation analysis, as shown in Fig.1(b). In the stage (2), the relationship between generated pore pressures $u$ and total stresses applied $\sigma$ is composed from the results of conventional oedometer tests (without pore pressure measurements). The value of the pore pressure coefficient $B$ is given by an inclination of the $u-\sigma$ curve as a function of $\sigma$. In the stage (3), instead of solving the differential equation of consolidation, the use of the relationship between the degree of consolidation $U$ and the time factor $T$ in one-dimensional consolidation is proposed to calculate pore pressure dissipation. The $U-T$ relationship to be used in the analysis may be defined from the results of oedometer tests. It is, however, not necessarily sufficient to reproduce field consolidation conditions in laboratory tests, because anisotropic permeability is considered remarkable in the field according to the placement conditions and drainage systems may differ considerably for different dam sections. It is therefore recommended in the present analysis that $U-T$ relationships should be composed from pore pressure measurements in the field.

Detailed descriptions on these improvements are given in the following sections.
Fig. 2. u-σ relationship

\[ u = \frac{p_0 \delta}{(v_a + H_e v_w) - \delta} \]  

\( v_a \) and \( v_w \), respectively, are free air and free water contents in pore per unit volume of soil (\%), \( \delta \) is a compressive strain of soil, \( p_0 \) is the atmospheric pressure and \( H_e \) is the Henry’s coefficient of solubility (usually \( H_e = 0.02 \)). With compacting conditions (dry density \( \gamma_d \) and water content \( \omega \)) of the dam given, the values of \( v_a \) and \( v_w \) are known and \( u \) becomes a function of \( \delta \) in Eq. (1). If an effective stress \( \sigma' \) versus compressive strain \( \delta \) relationship is provided as shown in Fig. 2(a) from the results of oedometer tests on specimens compacted in the same conditions as in the actual dam, generated pore pressures \( u \) may be determined by Eq. (1) through the value of \( \delta \) for every corresponding value of \( \sigma' \). The pore pressure \( u \) versus total stress \( \sigma \) relationship is then obtained by considering \( \sigma = \sigma' + u \), as shown in Fig. 2(b). In the region where \( \delta > v_a \), the value of \( v_a \) should be zero (full saturation) and \( u-\sigma \) curve should be parallel to the line inclining at 45° to both axes (100% generation line). The value of \( B \) is defined as an inclination of the curve.

Based on the procedures described above, the \( B \) values can be determined as a function of \( \sigma \) from conventional oedometer tests without pore pressure measurements. It has been confirmed that Eq. (1) can well be applied to soils having high degree of saturation of more than 85 percent. The use of Eq. (1) to define the \( u-\sigma \) relationship is therefore considered practically useful for highly saturated earth dams compacted on the wet side of the optimum like those in Japan.

In the finite element consolidation analysis, the process of pore pressure generation and dissipation is repeated in steps according to the idealized construction stages, as shown by solid lines in Fig. 3. Actual pore pressure behaviors, however, should be represented by a smoothly curved line (dashed line) under a constant rate of placement. During these stages of the analysis, the value of \( B \) may reasonably be determined from the original \( u-\sigma \) curve depending on the total stress that has existed. If there is a winter shutdown during the construction period, pore pressure development after the shutdown is somewhat influenced by the change in compressibility of soil structures due to consolidation and/or cementation effect of fine soils. It has been noticed (Bishop, 1957) that the value of \( B \) generally tends to decrease in the following construction season rather proportionally to the degree of consolidation achieved during the shutdown season. In the case such winter shutdowns are anticipated in advance, additional investigations should be made in oedometer tests to estimate characteristics of \( B \) coefficient before and after shutdowns (Eisenstein and Law, 1977).
Fig. 4. U-T relationship: (a) variations of pore pressure with time during construction, (b) plot of observed data on U-T diagram

U versus T Relationship (Yamaguchi and Ohne, 1973).

Let us first describe a method of composing the relationship between the degree of consolidation $U$ and the time factor $T$ on the basis of pore pressure measurements in the field. The variations of pore pressures and bank load with time at a point in an earth dam are schematically illustrated in Fig. 4(a). With the overburden pressure (total stress) on the point of pore pressure measurement given through the placement data, generated pore pressure $u_H$ is obtained by use of the $u-a$ relationship shown in Fig. 2. The value of $u_H$ indicates the maximum value of pore pressure development at the point during construction. By plotting observed pore pressures $u$ on the same figure, the degree of consolidation can be defined at any time by the equation $U = (u_H-u)/u_H$, because the difference between $u_H$ and $u$ may represent pore pressure dissipation. On the other hand, the time factor $T$ can be defined with the values of the coefficient of consolidation $c_v$ and the shortest drainage distance $H$ of the point under consideration to the drains known, as $T = c_v t / H^2$ for any time $t$. A typical $U$-$T$ relationship thus obtained for a centrally located core of an actual earth dam is shown in Fig. 4(b). In this figure, the observed data are plotted by open circles and the bounds of scattered data and the mean of them are represented by two solid lines and a dashed line, respectively. Also shown by a chained line is the Terzaghi's theoretical solution for a constant load.

In Fig. 5, mean $U$-$T$ curves are drawn to compare four different types of dams shown below, where the drainage distance $H$ taken in the calculation is also shown for each dam. Construction materials used in these dams are classified as SM and/or SC to ML by the unified soil classification. It is seen in Fig. 5 that pore pressures dissipate faster in order of dam types as inclined core, central core, homogeneous (with a vertical drain) and with horizontal drains. Also noticed is the fact that the observed $U$-$T$ relationships generally show moderate rates of dissipation particularly in the range of $U > 50\%$ as compared with the Terzaghi's theoretical solution. The differences in consolidation behaviors of these four types of dams are considered to be mainly caused by the discrepancy in horizontal and vertical permeabilities due to roller compaction and that in drainage system. It may be inferred for instance that the slowest rate of dissipation shown in the dam with horizontal drains is mostly due to the anisotropic permeability.
of rolled fills which is hard to evaluate in laboratory oedometer tests, and that the dissipation progresses somewhat slower in the homogeneous dam than in core-type dams because drainage conditions in the former are rather close to one-way drainage in one-dimensional consolidation.

Considering that each observed curve shown in Fig.5 has been composed for a corresponding specific dam and that observed data spread in a fairly wide range as is seen in Fig.4, U–T relationships thus obtained are by no means generally valid. However, in a rough estimation of pore pressure behaviors as performed in a preliminary design, these relationships are considered applicable to a certain extent for dams having similar sections and construction materials. It is hereby recommended in the present study that either of the four curves in Fig.5 be applied in the consolidation analysis corresponding to the type of the dam analyzed.

A convenient use of Fig.5 for estimating construction pore pressures is illustrated in Fig.6 for a hypothetical homogeneous dam with a vertical drain. With the location of point A specified and with the values of the drainage distance \( H \) and the construction period \( t \) required to place the remaining body of the dam above the point assigned, the value of the time factor is determined as \( T = \frac{\epsilon \tau}{H^2} \). It should be noted here that the value of \( H \) is taken in this case as either shorter one of \( H_A \) or \( H_A' \). The degree of consolidation \( U \) of the point at the end of construction is then obtained from the corresponding \( U-T \) curve (3) in Fig.5 with the value of \( T \) known. On the other hand, pore pressure \( u_H \) set up under undrained conditions can be evaluated from the \( u-\sigma \) relationship by taking \( \sigma = \gamma_i h_A \) as shown in Fig.6(b) where \( h_A \) is the overburden height above the point A. Residual pore pressure of the point is thus estimated as \( u = (1-U)u_H \). The validity of this method, referred to as "simple calculation" in the present paper, will be discussed later.

In the present finite element approach, incremental pore pressures \( \Delta u_{it} \) of a point in a dam are calculated for subsequent placement loads with incremental total stress analysis and the use of \( u-\sigma \) relationship, as illustrated in Fig.7. Assuming that each of \( \Delta u_{it} \) dissipates independently according to the corresponding time measure \( t_i \), pore pressure dissipation is calculated in the similar manner as described above by applying one of \( U-T \) curves in Fig.5 for each pair of \( \Delta u_{it} \) and \( t_i \). Residual pore pressure is then given at any time \( t \) by the sum of every remaining part of \( \Delta u_{it} \) under consideration (shown by \( \Delta u_{it} \)).

**COMPARISONS WITH THEORETICAL SOLUTIONS**

**One-dimensional Consolidation of a Clay Fill**

In order to examine the applicability of the proposed method, an example analysis was made on the consolidation of a semi-infinite clay fill which increases in thickness with time in the z-direction, as shown in Fig.8(a). It was assumed that the fill was fully saturated (\( B=1 \)) and underlain by a pervious base foundation (two-way drainage). Pore pressures were calculated for a 45-m high
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Fig. 8. One-dimensional consolidation of a clay fill

fill placed in 15 months at a constant rate, with \( c_0 = 0.002 \text{ cm}^3/\text{s} \) and \( \gamma_i = 19.6 \text{ kN/m}^3 \).

Distributions of residual pore pressure at the end of construction are compared in Fig. 8(b) for different analytical approaches, in which solid curve represents the solution obtained by the present analysis using the Terzaghi's theoretical \( U-T \) relationship, the dashed one is that by the method proposed by Eisenstein et al. and the chained one is that by the the finite difference approach presented by Gibson. It is obviously seen in the figure that a very close agreement is attained between the latter two methods of analysis, while residual pore pressures obtained in the present analysis are lower in every height by around 20 percent as compared to others.

This type of problem where a fill increases in thickness with time, so that the drainage distance \( H \) varies with time, is often called a moving boundary problem. In the present analysis, the value of \( H \) is taken as the shortest distance to both end surfaces, that is, either shorter one of \( h_A \) or \( z_A \) at a point \( A \) in Fig.8(a). This treatment, however, was anticipated to yield erroneous situations while \( h_A \) is less than \( z_A \) where the values of \( T \) and \( U \) tend to decrease with time as the value of \( H (=h_A) \) increases. In order to avoid this and to hold continuity in pore pressure dissipation, the value of \( U \) for the increment \( \Delta U_0 \), for instance in Fig.7, was assumed to vary by \( \Delta U = U(c_0 t_1)H_2^2 - U(c_0 t_2)H_2^2 \) during the second stage of the analysis, in which \( H_2 \) is the drainage distance taken in this stage. Hence these adjustments cannot be an essential dissolution of the discrepancy to the theoretical solution, it should be noted that the present analytical approach has some inherent shortcomings in applying to a moving boundary problem.

Pore Pressure Behaviors within a Central Core

As the second example analysis, pore pressure behaviors within a part of the centrally located core of a dam were investigated, as shown in Fig.9(a). It was again assumed that the core was fully saturated \( (\bar{B} = 1) \) and the pore water flowed only in the horizontal direction (two-way drainage). Pore pressure dissipations were calculated for an increasing uniform overburden pressure \( p \) acting on the surface with \( c_o = 0.002 \text{ cm}^3/\text{s} \) and a constant filling rate of 62.5 kN/m² per month.

The variations of pore pressure with time at points a through e are shown in Fig. 9(b), in which solid curves are the solutions ob-
obtained by the present method using the theoretical $U-T$ relationship and the dashed ones represent graphic solutions of one-dimensional consolidation for an increasing overburden load. It is evident that both solutions are in good agreement except in the vicinity of the drain. This suggests that in such cases where horizontal flow is considered predominant and the drainage distance may be assumed constant during construction as in core-type dams and in homogeneous dams with vertical drains, the present approximate method of analysis can be used effectively without any shortcomings which arised in the former example.

**EVALUATION OF PORE PRESSURES IN ACTUAL DAMS**

As mentioned previously, four representative $U-T$ relationships in Fig.5 have been composed on the basis of field measurements of pore pressure in corresponding dams. In order to examine validities of the proposed analytical procedures and the use of mean $U-T$ curves in the analysis, finite element back analyses were made here on the following three dams which had presented original observed data.

*Tonogawa Dam (Fukuoka Prefecture)*

Tonogawa dam is a 38-m high earth dam constructed in 1966. The dam is almost homogeneous in materials consisting of SM to CH soils, but has near vertical drains in the central part and three stages of horizontal drains in the upstream and downstream parts of the dam in order to promote pore pressure dissipation. The main transverse section of the dam is shown in Fig.10 with the locations of pore pressure meters. Mean $U-T$ curves (1) and (4) presented in Fig.5 have been composed based on the observed data in the core and in the upstream and downstream parts of the dam, respectively.

An outline of construction process of the dam and a $u-\sigma$ relationship defined from oedometer tests on materials used are shown in Figs.11(a) and (b), respectively. The dam was built up to EL 59 m (around the level of the 3rd horizontal drain) in about 190 days in the first construction season, then followed by a 75 day winter shutdown, and completed in 100 days in the following season. As mentioned before, the $u-\sigma$ relationship ($\bar{B}$ coefficient) may be anticipated to change considerably after an intervening winter shutdown. Due to the lack of data on this respect, however, and considering that the most part of the dam had been built before

![Fig. 10. Tonogawa dam](image-url)

![Fig. 11. Basic data of Tonogawa dam: (a) construction process, (b) $u-\sigma$ relationship](image-url)
the winter shutdown, the curve presented in Fig. 11(b) was used here in the whole process of calculation without any modifications. It was also assumed in the analysis that actual construction was simulated by 20 lifts and that the core part of the dam rested on an impervious foundation. Concerning the $U$-$T$ relationship, the curves (1) and (4) in Fig. 5 were applied for the core and the upstream and downstream parts of the dam, respectively. The other necessary material parameters were determined from oedometer tests and placement data as $c_v = 0.002 \text{cm}^2/\text{s}$ and $\gamma_f = 19.2 \text{kN/m}^3$ on the average.

Computed and observed pore pressure variations with time are compared in Figs. 12(a) through (f) at six representative points in the dam. In every figure, thin solid and chained lines, respectively, represent variations of the computed maximum principal stress $\sigma_1$ and those of generated pore pressure $u_H$ given from the $u$-$\sigma$ curve in Fig. 11(b) taking $\sigma = \sigma_H$ and open and solid circles are computed and observed pore pressures, respectively. It should be noticed in these figures that computed results on the whole provide fairly good agreements with the observed ones up to the start of the winter shutdown. During and after the shutdown season, however, computations have yielded much larger values of pore pressure than the observed ones in the upstream and downstream parts of the dam, though they show similar tendencies in pore pressure generation and dissipation. These discrepancies are considered mainly due to the facts that the influence of winter shutdown on $B$ coefficient was not included in the present analysis, resulting in rather high development of pore pressures after the shutdown, and that the use of the $U$-$T$ curve (4) in Fig. 5 was not necessarily appropriate for all points in the upstream and downstream parts of the dam. Concerning the latter, it is well recognized for instance in Fig. 12(c) that the flow of water at point T-3 near the vertical drain is considered predominant in the horizontal direction due to anisotropic permeability, so that an assumption of vertical flow (the use of the curve (4) in Fig. 5) may have resulted in erroneous solutions. This is also confirmed in Figs. 12(a), (d) and (f), where the difference between computed and observed results decreases as the point under consideration is located farther from the vertical drains.

Computed and observed pore pressure distributions within the embankment are compared in Figs. 13(a) and (b). In Fig. 13(b), the present finite element solution is represented by solid contour lines on the upstream side and the result of simple calculation explained before by dashed contour lines on the downstream side. These results are almost symmetric with respect to the center line of the dam. Due to the same reasons as mentioned above, both results of the finite element and simple calculations in the upstream and downstream parts of the dam provide considerably high values of pore pres-
pressure and concentration of contour lines in the vicinity of the vertical drains. It is also noticed in the core part of the dam that both calculations yield rather high maximum pore pressures on the base foundation while the maximum contour of the observed values appears in the lower half of the core. This is caused by the fact that an assumption of impervious foundation could not satisfy actual partially drained conditions. Comparing between the calculated results in Fig.13(b), it is recognized that the simple calculation is not inferior to the finite element analysis.

Isaka Dam (Mie Prefecture)

Isaka dam is a 36-m high earth dam with an inclined core constructed in 1965. The base foundation in the circumference of the damsite consists of alternate layers of sand and sandy mudstones, and weathered mudstones classified in SC to ML were used as core materials. Based on the corresponding $U-T$ curve (2) in Fig.5, finite element consolidation analysis was made on the inclined core of the dam with the average material parameters of $c_v=0.0036 \text{ cm}^2/\text{s}$ and $\gamma_s=18.6 \text{ kN/m}^3$.

Computed and observed results are compared by pore pressure variations with time at three representative points in Figs.14(a) through (c) and distributions of pore pressure in the inclined core in Figs.15(a) and (b). In Fig.15(b), the result of simple calculation is represented by dashed contour lines. It was assumed in the present analysis that the mudstone base foundation on which the inclined core rested was impervious and the upstream shell of sandy gravels was sufficiently pervious. It is obviously seen in Fig.

![Fig. 13. Pore pressure distributions (Tonogawa dam)](image)

![Fig. 14. Pore pressure variations with time (Isaka dam)](image)
15 that these assumptions are quite reasonable, approximating actual boundary conditions. Also noticed in Figs. 14 and 15 is the fact that satisfactory agreement between computed and observed pore pressures is attained both in magnitude and distribution.

**Togo Dam (Aichi Prefecture)**

Togo dam is a homogeneous earth dam of about 30-m in height having a vertical drain on the downstream side. The base foundation consists of alternate layers of silty clay and sand, slightly sloping down to the downstream. Most of construction materials used in the dam are classified as MH or ML, but some of them in the upstream slope are SC and CH. By using the corresponding $U-T$ curve (3) in Fig. 5, pore pressure analyses were performed assuming that the dam rested on a pervious foundation. Although the drainage distance $H$ in this case should be taken in the horizontal direction as illustrated in Fig. 6, the values of $H$ were taken here in the vertical direction for points in the lower part of the dam near the assumed pervious foundation in order to avoid erroneous results which arose in the case of Tonogawa dam. From basic data of the dam, material parameters were taken as $c_v=0.002$ cm$^3$/s and $\gamma_f=17.6$ kN/m$^3$ on the average.

Variations with time and distributions of computed and observed pore pressures are compared in Figs. 16(a) through (d) and in Figs. 17(a) and (b), respectively. Since $c_v$ values in oedometer tests scattered in a wide range, additional finite element computation was carried out in this case for another value of $c_v$ in order to examine their influence on pore pressure dissipation. The result is presented in Fig. 17(b), drawing pore pressure distribution for $c_v=0.005$ cm$^3$/s by dashed lines.
At the time when about half of the dam had been built, high excess pore pressures and large deformations caused by sliding were observed in the upstream part of the dam and construction was ceased for about eight months to prevent subsequent failures. Due to the similar reasons as before, the effect of this intervening shutdown on $\bar{B}$ coefficient was not taken into account in the present computations. This probably resulted in comparatively high computed pore pressures in the upper half on the dam as shown in Fig. 17, in which the discrepancy between the computed results for two different $c_e$ values is somewhat large but not so remarkable in the lower half of the dam. Also found in Fig. 17 is the fact that observed pore pressures differ much from the computed results in the upstream part of the dam, showing considerably high values and concentration of contour lines along the surface. This is partly because the dam was not completely homogeneous having clayey soils of SC and CH with high water content in the upstream slope and because the present analysis has some shortcomings in portions far from the drain where the drainage distance cannot be defined clearly whether in the vertical or in the horizontal. As far as the central part of the dam is concerned, however, it should be noted here again that the present analytical approach on the whole yields good correspondence with pore pressure behaviors in actual dams.

**SUMMARY AND CONCLUSIONS**

A simplified method of estimating construction pore pressures in earth dams was proposed in the present study on the basis of the concept of one-dimensional consolidation. The method, unlike recent rigorous coupling approaches, was developed from a clear-cut point of view in analytical procedures and in determining material parameters, aiming at simple and practical applications in the preliminary design. Although some inherent shortcomings were pointed out depending on the situations analyzed, it should be emphasized that the present finite element approach and also the simple calculation showed quite satisfactory results especially in such cases where the drainage distance can be defined clearly as core-type dams, offering sufficient accuracy for practical purposes. The simple calculation is therefore quite advantageous when only pore pressure distributions at the end of construction are concerned.

The present finite element analysis comprises deformation analysis as well as pore pressure analysis. Attention was paid here, however, only on the pore pressure behaviors in earth dams, because procedures for deformation analysis are the same as those presented by Eisenstein et al. (1976). As far as pore pressure analysis is concerned, it is of great importance in the present approach to evaluate appropriate values of material parameters such as $\bar{B}$, $c_e$ and $U-T$ relationship.

Concerning the $\bar{B}$ coefficient, the proposed method of composing $u-\sigma$ relationship from oedometer tests is considered valid for soils having high degree of saturation, while characteristic effects of a winter shutdown on the $\bar{B}$ coefficient are to be clarified for individual situations of dam construction. Since the coefficient of consolidation $c_e$ is also stress dependent, its dependency on the effect of consolidation should be taken into account in precise investigations. It is, however, rather important in this type of approximate analysis that actual field placement conditions are to be reproduced as closely as possible in laboratory tests. For instance, it is desirable to perform large-scale oedometer tests in order to approximate actual material grain sizes in the laboratory, and it is necessary to investigate consolidation characteristics of soils with horizontal drainage as well as vertical drainage when anisotropic permeability is expected remarkable due to roller compaction. As for $U-T$ relationships in Fig. 5, their reliability and applicability should be examined continuously by utilizing many other available data in the field. Since, however, earth dams individ-
ually consist of different dam sections and materials and experimental results more or less spread in a certain range, some engineering judgements are also required in determining material parameters and interpreting analytical results.

REFERENCES