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PORE PRESSURE PREDICTION BY A NUMERICAL METHOD IN EARTH DAM CONSTRUCTION

PREVISION DE LA PRESSION AU NIVEAU DES PORES PAR METHODE NUMERIQUE DANS LA CONSTRUCTION DES BARRAGES EN TERRE

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SYNOPSIS : A method which can simulate the pore pressure development in the embankment is presented in this paper. Since the pore pressures in a core zone of the earth-fill dam is accumulated with the increase of dam height in construction stage, it is well known that its magnitude is closely related to the dam stability. The method is based on a combination of a proposed pore pressure model and the hyperbolic model which can calculate incremental stresses. The pore pressure model is formulated in terms of the compressibility of air-water mixture within compacted fill materials, and thus the proposed constitutive equation can be coupled to both the confining pressure and the deviator stress. The parameters required to computation in the proposed model can be readily determined from the conventional triaxial compression tests.
The proposed model is incorporated into a finite element analysis for actual application. The pore pressures within the core zone upon the completion of construction of Juam dam are computed with the software program. The results are compared with the measured pore pressures of the core zone of the dam. It is shown from the comparison that the proposed method can predict pore pressures in earth dam construction with satisfaction.

INTRODUCTION

Compacted clayey materials has been widely used as a core element in the fill dam construction. The development of high pore water pressures within the clay core is an important factor in evaluating their stability since it may cause either excessive deformations or shear failures during embankment construction. It is well known that the magnitude of the pore water pressures depends on the intensity of the stresses induced by the accumulation of soil layers and on the rate of pore pressure dissipation.

The core materials used to the earth dam construction are partially saturated soils. The behavior of partially saturated soils is governed by the magnitude of the pore air and pore water pressures developed. Hilf(1948) formulated an equation possible to determine the pore air pressure within an unsaturated soil which developed due to total stress changes under undrained conditions. Skempton(1954) and Bishop(1954) derived an equation for the B pore pressure parameter. This equation, however, did not take account the effect of surface tension, which is difference of pressures in the air phase and that of water phase.

Simulating the actual construction sequence of an earth dam, the changes of stesses in earth dams are possible to calculate based on hyperbolic stress-strain relationships. Duncan et al.(1980) provided the procedure for obtaining hyperbolic parameters available to the finite element analysis from a series of conventional triaxial compression tests. On the other hand, Kim(1982) proposed a method evaluating the compressibility of the pore fluid. These two works form the starting point of this paper.

The purpose of this paper is to present a model which can predict pore water pressure within a core zone upon the completion of dam construction. The model takes into account the nonlinear compressibility of pore air-water mixture. With the model incorporated into FEM software program the pore pressure development of a dam is estimated and the results are compared to measured values.

PORE PRESSURES DEVELOPED BY TOTAL STRESSES

Clough and Woodward(1967) have examined the usefulness of both incremental finite element analyses in which the placement of successive layers was simulated and simple gravity turn-on finite element analyses in which the gravity body forces were applied to the entire structure at one time. Their study indicates that incremental finite element analyses correspond well to stresses and strains measured for real embankments. It is thus apparent that if finite element techniques are to be employed in calculating pore pressures developed during dam construction, the incremental pore pressure parameters should be required.

For the pore pressures developed by a triaxial stress condition, pore pressure changes can be divided into two components as follows:

$$\Delta u_w = \Delta u_{wi} + \Delta u_{wd} \tag{1}$$

where Δu_{wi} is pore pressure change under isotropic condition and Δu_{wd} is pore pressure change due to deviator stress. Skempton(1954) described incremental pore pressure in terms of pore pressure parameters and stress changes:

$$\Delta u_w = B\Delta\sigma_3 + D(\Delta\sigma_1 - \Delta\sigma_3) \tag{2}$$

where B and D are pore pressure parameters; $\Delta\sigma_3$ and $(\Delta\sigma_1-\Delta\sigma_3)$ are changes of isotropic stress and deviator stress respectively.

Skempton(1954) and Bishop(1954) derived the following equation for the B pore pressure parameter:

$$B = \Delta u_{wi} / \Delta\sigma_3 = (1 + n\, C_w/C_{c3})^{-1} \tag{3}$$

where n is porosity; C_w is compressibility of air-water mixture; and C_{c3} is the compressibility of the soil skeleton being subject to isotropic compression stress under drain condition.

In analysis of partially saturated soils, it is assumed that the pore space is filled with an equivalent fluid whose compressibility is the same as that of the air-water mixture. The physics of pore air-water mixture is too complex, however, the compressibility of air-water mixture depends mainly on the degree of saturation.

Skempton and Bishop(1954) calculated the compressibility of air-water mixture with the aid of Boyle's law and Henry's law of solubility as follows:

$$C_w = (1 - S_0 + S_0 H)\, u_{ao}/u_w^2 \qquad (4)$$

where S_0 is initial degree of saturation; H is coefficient of solubility, which is equal to 0.02 at 20°C; u_{ao} is initial pore air pressure; and u_w is pore water pressure.

In equation (4) the surface tension has been neglected for the sake of simplicity. An attempt to take into account the influence of the surface tension on the compressibility of air-water mixture was made by Schuurman(1966). His theory is available when the degree of saturation is in excess of 85% in which the air is assumed to be present in the form of bubbles. Standard optimum moisture content and the wet side of OMC are usually in excess of 85% of degree of saturation.

In Schuurman's study, C_w is expressed as a function of the variable air volume. It is not possible to express C_w explicitly in terms of pore water pressure, because air volume cannot be eliminated.

A simple model has been proposed by Kim(1982) in which the difference between the air and water pressure is assumed to remain constant over the pressure ranges encountered in practice. The difference between the air and water pressure increases with increasing pore water pressure since the radius of the air bubbles decrease as the pressure in the surrounding pore water increases. With the assumption of constant pressure difference T, Kim(1982) has derived the following equation:

$$C_w = (1 - S_0 + S_0 H)\, u_{ao}/(u_w + T)^2 \qquad (5)$$

The value of constant pressure difference T can be determined from trial simulation of undrained isotropic compression tests with pore pressure and volume change measurements.

The relationship between the B pore pressure parameter and the pore water pressure is obtained by substituting equation (5) into equation (3).

$$B = \frac{\Delta u_{wi}}{\Delta \sigma_3} = \frac{(u_{wi}+T)^2}{(u_{wi}+T)^2 + a_i} \qquad (6)$$

where u_{wi} = pore pressure developed by isotropic stress
$a_i = n_o\, u_{ao}(1 - S_o + S_o H)/C_{c3}$

The magnitude of B is significantly affected by the compressibility of the soil skeleton. As the compressibility of the soil skeleton increases, the pore pressure response increases and vice versa. In the case of compacted soils, the compressibility of the soil skeleton can be assumed to be constant since the change of the compressibility C_{c3} is very slight.

With the assumption of constant compressibility C_{c3}, a first-order differential equation(6) can be solved. By separating variables and integrating we obtain as:

$$u_{wi}^2 - (\sigma_3 - T + c_i)u_{wi} - [(\sigma_3 + c_i)T + a_i] = 0 \qquad (7)$$

Since c_i is constant, this may be solved as follows:

$$u_{wi} = \frac{1}{2}(\sigma_3 - T + c_i) + \frac{1}{2}\left[(\sigma_3 - T + c_i)^2 + 4\{(\sigma_3 + c_i)T + a_i\}\right]^{1/2} \qquad (8)$$

When the cell pressure is kept constant and if the soil element is elastic and isotropic, the D pore pressure parameter can be expressed as:

$$D = \Delta u_{wd}/(\Delta\sigma_1 - \Delta\sigma_3) = (3 + n\, C_w/C_{c1})^{-1} \qquad (9)$$

where C_{c1} is the compressibility of the soil skeleton obtained from laboratory compression tests under uniaxial loading with zero excess pore water pressure.

The relationship between the D pore pressure parameter and the pore water pressure is obtained by substituting equation (4) into equation (9).

$$D = \frac{\Delta u_{wd}}{(\Delta\sigma_1 - \Delta\sigma_3)} = \frac{u_{wd}^2}{3u_{wd}^2 + a_d} \qquad (10)$$

where Δu_{wd} = pore pressure developed by deviator stress
$a_d = n_o\, u_{ao}(1 - S_o + S_o H)/C_{c1}$

With the assumption of constant compressibility C_{c1}, the first-order differential equation(10) can be solved. By separating variables and integrating we obtain as:

$$3u_{wd}^2 - [(\sigma_1 - \sigma_3) + c_d]u_{wd} - a_d = 0 \qquad (11)$$

Since a_d is constant, this may be solved as:

$$u_{wd} = \frac{1}{6}[(\sigma_1 - \sigma_3) + c_d] + \frac{1}{6}\left[\{(\sigma_1 - \sigma_3) + c_d\}^2 + 12a_d\right]^{1/2} \qquad (12)$$

Adding the pore pressure developed by the isotropic stress to that resulting from the deviator stress, it gives:

$$u_{wi} = \frac{1}{2}(\sigma_3 - T + c_i) + \frac{1}{2}\left[(\sigma_3 - T + c_i)^2 + 4[(\sigma_3 + c_i)T + a_i]\right]^{1/2} + \frac{1}{6}[(\sigma_1 - \sigma_3) + c_d] + \frac{1}{6}\left[\{(\sigma_1 - \sigma_3) + c_d\}^2 + 12a_d\right]^{1/2} \qquad (13)$$

APPLICATION OF PORE PRESSURE MODEL

To examine the effectiveness of the proposed model for the analysis of zoned embankments, the finite element analysis taking account of incremental construction procedure was performed for Juam dam where is located in Junnam Province, South Korea. Shown in Fig. 1 is the finite element mesh employed in the analysis. The mesh comprises 14 layers, each of which simulates a construction lift. The values of dry unit weight, water content and shear strength for the core materials are shown in Table 1.

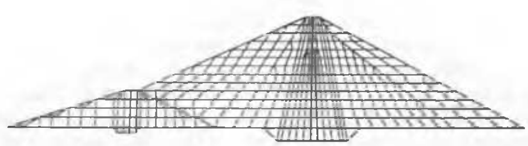

Fig. 1. Finite element mesh of Juam dam

Table 1. Dry unit weight, water content and shear strength for the core materials

γ_d (t/m^3)	w (%)	c (t/cm^2)	φ (°)
1.778	16.47	5.88	19.45

In order to calculate stresses by the finite element method, it is important that the stress-strain characteristics of the soils be represented in a reasonable way. It is not easy task because the stress-strain characteristics of the soils is nonlinear. In spite of this difficulty the hyperbolic model is known to be effective in calculation of incremental stresses in earth dam construction. In this paper a computer program FEADAM84 is used to calculate the total stresses. This is a finite element program for static analyses of earth and rockfill dams, with which the nonlinear and stress-dependent stress-strain properties of soils can be approximated by using a hyperbolic model developed by Duncan et al.(1980).

The hyperbolic model and the pore pressure model use curve fitting procedures to produce a family of the constitutive parameters. This conventional procedure of calibration via some kind of curve-fitting can be viewed as a suboptimization technique, since a few data is quality of the calibration, a global optimization technique should be adopted. All available data are then included, although with different weights that reflects their relative importance and reliability in the objective function to be optimized.

To determine the parameters of the pore pressure model, the cost function can be used. The distance between the experimental points and the theoretical curve is computed using Euclidean norm.

$$d_{ij} = (u^{e}_{ij} - u^{c}_{ij})^2 \qquad (14)$$

where u^{e}_{ij} is experimental pore pressure; u^{c}_{ij} is computed pore pressure; and the subscripts i and j denote the experiment and the experimental point respectively.

Based on the distance d_{ij} for all available experimental points, the composite norm for experiments is defined as:

$$E_i = \frac{1}{k}\sum_{j=1}^{k} d_{ij} \qquad (15)$$

where k is the number of experimental points. These norms are composed to the following cost function:

$$J = mE_{max} + \sum_{i=1}^{m} W_i E_i \qquad (16)$$

where m is the number of experiment. The weight W_i characterizes the importance of a certain experiment and E_{max} represents the worst computation.

The stress-strain curves and the stress-pore pressure curves can be readily obtained from the results of a series of conventional triaxial compression tests. During construction of Juam dam triaxial compression tests were carried out to obtain input data for computation. From the test results stress-pore pressure curves can be aquired as shown in Figure 2 and Figure 3.

The initial portions of the stress-pore pressure curves obtained from the back analyses are not coincided with that resulting from the tests. Two types of deviations from the actual behavior come from the characteristics of the proposed model. These deviation cause overestimation of the pore pressures under low confining pressures. To reduce overestimation of the pore pressures it would be convenient to make the initial portion of the stress-pore pressure curves to be straight lines, which start at the origin.

Using the optimization technique described above, the values of the required parameters for the core materials were calibrated as shown in Table 2 and Table 3.

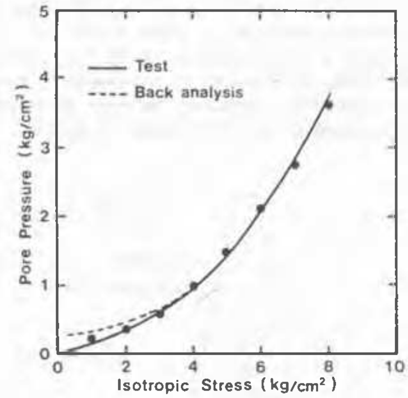

Fig. 2. Pore pressure variation with isotropic stress

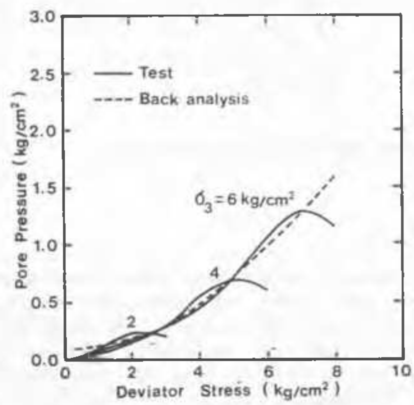

Fig. 3. Pore pressure variation with deviator stress

Table 2. Nonlinear hyperbolic parameters

K	n	R_f	K_b	m
485	0.61	0.83	370	0.53

Table 3. Pore pressure parameters

a_i	a_d	c_i	c_d	T
1.57	0.38	-4.65	-3.38	0

COMPARISON OF RESULTS

To examine the degree to which the results by the proposed pore pressure model reflect the variation of pore pressures in situ, the computed pore pressures have been compared to the pore pressures measured by instruments installed in the embankment of Juam dam. Space limitations preclude a complete comparison, and thus only computed and measured pore pressure distributions are presented in Fig. 4.

The computed and measured values are relatively in good agreement at the center of the core. The greatest departure is observed near the downstream filter where some pore pressure dissipation may have occured. Measurements in the upper third of the core are lower than computations, probably because the initial suction remains high under low overburden pressures and the initial portions of the stress-pore pressure curves by the proposed model deviate from that by the tests. In the lower third of the core, computed pore prssures are high, with maximum values at the center of the core. These high values are mainly due to the compression of the core materials. The numerical analysis made no allowance for the change in water content and therefore a substantial discrepancy between measurements and computations is presented at the bottom center of the core.

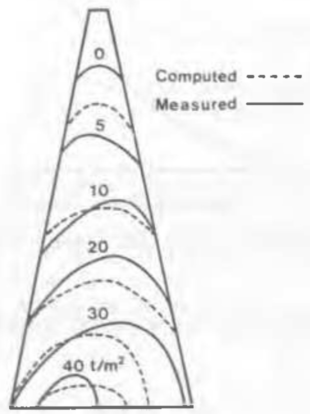

Fig. 4. Measured and computed pore pressures

CONCLUSIONS

From above discussion it has been shown that pore pressue developing in clay zone of earth dams can estimate using a numerical model. Incremental stresses with incresing dam height can calculate based on hyperbolic model, and pore pressure increments caused by stress changes can be obtained from the pore pressure model presented here. The proposed pore pressure model is formulated in terms of the comprssibility of air-water mixture. The surface tension effects are included in the proposed pore pressure model in the form of a constant pressure difference between the pore air and the pore water.

The input data for caculation are easily obtained from conventional triaxial compression tests. Using the model developed a prediction is made for Juam dam which has completed the construction recently. In spite of the improvement of the parameter estimation by cost function, the discrepancy between the computations and measurements is still considerable in both lower and upper part of the core zone. This is because the laboratory investigations cannot simulate completely the natural conditions. In central part of the zone, however, the proposed model has shown in good agreement with measurements.

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