PREDICTION OF PORE PRESSURES IN EARTH DAMS

PREDICTION DES PRESSIONS INTERSTITIELLES DANS LES BARRAGES EN TERRE

B.G. RICHARDS, Senior Research Scientist.
Div. of S. Mechanics, CSIRO, Syndal, Vic. Australia.

C.Y. CHAN, Engineer.
Div. of S. Mechanics, CSIRO, Syndal, Vic. Australia.

SYNOPSIS The performance of earth dams is largely dependent on the transient moisture flow and the resulting pore pressure distribution within and under the embankment. This paper describes the development of a computer program for calculating changes in both positive and negative pore pressures in earth dams. This method involves the solution of the simple two-dimensional flow equation for isothermal liquid phase transfer of moisture in saturated and partially saturated soils. The program has been applied to the clay core of the Flagstaff Gully Dam, Tasmania, in which positive pore pressure readings have been obtained from Casagrande type piezometers, since it was refilled following a catastrophic piping failure. The agreement between predicted and observed positive pore pressures was remarkably good. However, negative pore pressures measured in the laboratory on samples taken from the core were not in such good agreement with the predicted values, but did show the same relative trends.

INTRODUCTION

Soil water potential or pore pressure plays a very important role in governing soil engineering parameters such as shear strength, volume change and deformation moduli, (Bishop et al. 1960; Donald, 1964; Blight, 1965 and 1966; Richards, 1968b). Consequently, as has already been demonstrated, these soil water variables have a large influence on slope stability which is an important consideration in earth dam design (Bishop, 1955). Furthermore, both chemical and physical erosion or piping (Wood et al, 1964; Aitchison et al. 1965), within the embankment is dependent on flow rates and therefore the potential or pore pressure distribution within and under the structure.

It is therefore evident that soil water potentials and pore pressures have a significant influence on the performance of an earth dam at all times during its life and must be considered in its design. Positive pore pressures at any point in time can be measured by piezometric instruments (Casagrande, 1949; Ingles et al. 1968) and techniques are now becoming available for the measurement of negative pore pressures (Richards, 1968a) over the range normally observed in practice. However, pore pressure prediction into the future life of the dam has received little attention.

Present practice tends to consider two cases only, the construction or initial pore pressure distribution and the final ultimate condition, based on flow net techniques (Casagrande, 1937). The former is extremely important and forms the initial or starting condition for the transient conditions considered in this paper. However, the prediction of these initial pore pressures themselves is another problem and beyond the scope of this paper. The ultimate condition is also important, but is often reached in the cores of large dams only after many decades. In the meantime, the modes of dissipation of construction pore pressures and the development of seepage flow conditions are unknown. In addition, the common techniques of constructing flow nets for the ultimate condition are based on assumptions, which are correct only for coarse grained materials in which the so called "capillary rise" is insignificant relative to the dimensions of the structure. In clay cores these assumptions are completely erroneous, as 1) the flow parameters are non-linear, resulting in the breakdown of the orthogonal Laplace condition and 2) flow above the so-called "phreatic" line in the partially-saturated regions is very significant. This latter condition means that the "phreatic" line is no longer a flow line, and flow in fact can cross this imaginary line. It also means that the entry and exit conditions are also subject to modification.

This paper describes a technique which enables the transient flow conditions in both the saturated and partially saturated regions of an earth dam to be analysed at all times in the life of the dam following the establishment of the initial construction pore pressures. The results of this analysis are the subsequent pore pressure distributions as a function of time, which can be used in design considerations.

THEORY

The theory of water movement and equilibria in soils as it applies to soil engineering has already been described in detail by the author (Richards, 1968c), with particular reference to pavements and subgrades. It is therefore sufficient to state that, for the application considered here, isothermal conditions with insignificant solute gradients can be assumed. As the main concern is with clay cores of the embankment whose material generally remains approximately pore space saturated to negative pore pressures less than -100 kg./cm² (-1420 psi) (Holmes, 1955), moisture flow in the vapour phase need not be considered. In any case, in other materials at low suctions and under the boundary conditions applicable here, moisture flow in the vapour phase will be insignificant. Therefore, isothermal flow in the liquid phase only can be represented more than adequately for this application by an equation of the form

$$\frac{\partial u}{\partial t} = \frac{\partial u}{\partial \theta} \nabla (k(u).\nabla (u + z)) \qquad \ldots \ldots (1)$$

u = pore pressure
t = time
θ = volumetric water content
$k(u)$ = permeability as a function of u
z = height above datum level i.e. potential $\phi = u + z$

Equation (1) gives a mathematical expression of water flow in soils which is far more accurate than our present physical knowledge and understanding of flow in soils and our ability to measure the relevant flow parameters and boundary conditions in practice. Further justification for the use of this equation can be derived from the fact that the application here is to a relatively slowly changing system compared with the rapidly changing systems such as irrigation experiments considered with the aid of this equation in agriculture (Philip, 1957).

Thus the pore pressure distributions in the embankment can be analysed in terms of only 3 parameters, viz. pore pressure (u), k(u), and $\frac{\partial u}{\partial \theta}$ $(=\frac{\partial u}{\partial \theta}(u))$. Neither k(u) nor $\frac{\partial u}{\partial \theta}$ are single-valued functions of pore pressure (u), as they also depend on the previous stress and moisture history of the soil. Consequently these parameters must be determined over the appropriate stress and moisture paths as will be applicable to the field conditions.

Equation (1) can be solved simply and rapidly with a modern high speed digital computer. Expressing equation (1) in its two-dimensional difference form

$$\Delta u_5 = \frac{\Delta t}{(\Delta l)^2} \cdot C_5[\bar{k}_1(u_1 + \Delta l - u_5) + \bar{k}_2 (u_2-u_5) + \bar{k}_3(u_3-\Delta l-u_5) + \bar{k}_4(u_4-u_5)] \ldots \ldots \ldots (2)$$

$\bar{k}_1$ = mean permeability for u_1 and u_5 etc.
h_5 = suction at the point considered
$C_5 = \frac{\partial u}{\partial \theta}$ at the point considered
u_1, u_2, u_3 and u_4 are suctions at adjacent points of the mesh above, right, below and left of point 5 respectively.

Although this equation appears to be rather approximate, it can be accurate if the stability condition in equation (3) is observed where

$$\frac{\bar{k}\,\Delta t}{(\Delta l)^2} \cdot C_5 \leq 1/4 \qquad \ldots \ldots (3)$$

Any errors developed by the use of equation (2) in the step-wise numerical procedure below will tend to diminish and become insignificant. As

$$u_{5,(t+\Delta t)} = u_{5,t} + \Delta u_5 \qquad \ldots \ldots (4)$$

where $u_{5,t}$ and $u_{5,t+\Delta t}$ are suctions at point 5 at times t and $t + \Delta t$ respectively and knowing the values of u at t = 0, successive values of u at $t = \Delta t, 2\Delta t$, etc. can be readily calculated.

The major limitations in applying equations (2) and (4) to actual engineering problems lie in the accuracy of definition of the flow parameters u, k(u) and $\frac{\partial u}{\partial \theta}$ (u) and the boundary conditions.

FLOW PARAMETERS

(a) Pore Pressure (u)

The initial pore pressures at the site to be analysed and the relevant pore pressures for the boundary conditions can now be readily measured both in situ in the field and in the laboratory (Richards, 1968a).

(b) Permeability k(u)

The permeability of samples taken from the site should be determined as a function of u, using the pressure plate outflow technique for the case of negative pore pressures and the consolidation technique for positive pore pressures in clay materials (Richards, 1965). For positive pore pressures in materials of low clay content, the permeability is best measured in standard transmission type tests.

For the purposes of using these permeability functions in the computer program, a mathematical expression of the form

$$k(u) = \frac{D}{A+B.(-u)^m+C(-u)^n} + E \qquad \ldots \ldots (5)$$

where A, B, C, D, E, m and n are constants, is useful to fit the experimental curve as shown in a typical example (Figure 1).

Field permeability may also be determined from field pore pressure readings (Aitchison et al., 1965). Equation (2) can be rewritten with $\bar{k}_1$ to $\bar{k}_4$ equal to $\bar{k}(u_5)$ in the form

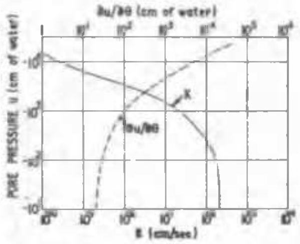

Fig.1. Flow parameters

$$\bar{k}(u_5) = \frac{\Delta u_5.(\Delta l)^2}{\Delta t.C_5} \cdot \frac{1}{(u_1+u_2+u_3+u_4-4u_5)} \quad (6)$$

C_5 can only be determined in the laboratory at present.

Differences between the laboratory and field permeabilities are not unusual, as laboratory determinations of permeability are generally much lower than those measured in the field. Because of disturbance and stress release in the laboratory sample and its relatively small size, it is unlikely to be representative of the soil in situ.

The permeability functions of the type expressed by equation (5) are single-valued and will undoubtedly be in error due to the effects of previous moisture and stress history, as already discussed. Whether or not the errors will be significant and what effect they will have is unknown at this stage. However, in many engineering problems the profile under or in the structure is either steadily wetting up or drying out. For this condition the appropriate single-valued function (i.e. wetting up or drying out from the initial condition) should give a sufficiently close approximation.

(c) Inverse differential water capacity $\frac{\partial u}{\partial \theta}$

$C_5 = \frac{\partial u}{\partial \theta}$ is also determined during the determination of k(u) using the pressure plate outflow technique. It can be approximated by an expression of the form

$$C_5 = \frac{\partial u}{\partial \theta} = a + b\ (-u)^p + c\ (-u)^q \quad \ldots \ldots (7)$$

where a, b, c, p and q are constants, fitted to the drying curve as shown in Figure 1.

BOUNDARY CONDITIONS

(a) Embankment dimensions

The physical dimensions of a typical embankment for the purposes of this computer program are shown in Figure 2. This configuration considers up to seven regions each with different material properties, which may also be anisotropic if necessary by allowing $\bar{k}_1$ and $\bar{k}_3$ to be determined from $k(u)_V$ vertically and $\bar{k}_2$ and $\bar{k}_4$ from $k(u)_H$ horizontally. The mesh size, Δl, is variable, depending on the dimensions of the structure and the basic time interval Δt is 1 week, but this can be changed if stability requirements so dictate.

(b) Embankment surface

The surface is defined as that zone within $\Delta l/2$ of the true surface, vertically for a slope less than or equal to 45° or horizontally otherwise. This ensures that all mesh points inside the surface zone have four effective mesh points surrounding them, enabling equation (2) to be employed.

In the surface zone, two distinct boundary conditions occur, viz.

1. Surface below either upstream or downstream water levels. In this case the pore pressure u_5 is put equal to the hydrostatic pressure at that point over the time interval. It should be noted that the co-ordinates of the upstream and downstream water levels are not necessarily fixed and can vary as functions of time, t.

2. Surface above the upstream or downstream water levels. In this case equation (2) can be rewritten, assuming a moisture loss or gain $e_{surface}$, normal to the surface, e.g. for the left hand boundary, and to a first approximation,

$$\Delta u_5 = \frac{\Delta t}{(\Delta l)^2}.C_5.\{e_{surface}.\Delta l + 1.5\ [\bar{k}_2(u_2-u_5) + k_3(u_3+\Delta l-u_5)]\} \quad \ldots \ldots \ldots (8)$$

$e_{surface}$ can readily be calculated from the difference between evapotranspiration and infiltration determined from meteorological data for the site (Richards, 1968c).

(c) Other Boundaries

In all practical applications, the conditions applying to these boundaries are generally known and can be adapted to this analysis.

COMPUTER PROGRAM

The program was written in Fortran II for the CSIRO CDC 3600 computer. The initial condition can be set up by a 53 by 97 array PWP(I,J), using some relevant mathematical expression, which is a function of position. The program then successively travels from point to point for each time interval, calculating new values of horizontal and vertical permeabilities and then the new pore pressure value. For this basic time interval Δt = 1 week, the running time for an equivalent real time of 10 years is of the order of 40 minutes.

APPLICATION

Flagstaff Gully Dam, a town water supply reservoir of 50 million gallons near the eastern suburbs of Hobart, Tasmania, is a rockfill structure with clay core, having a maximum height of 51 ft and a crest length of 600 ft. In July 1963, three weeks after first filling, this dam failed catastrophically, due to a piping failure which initiated at the junction of the clay core and the bedrock.

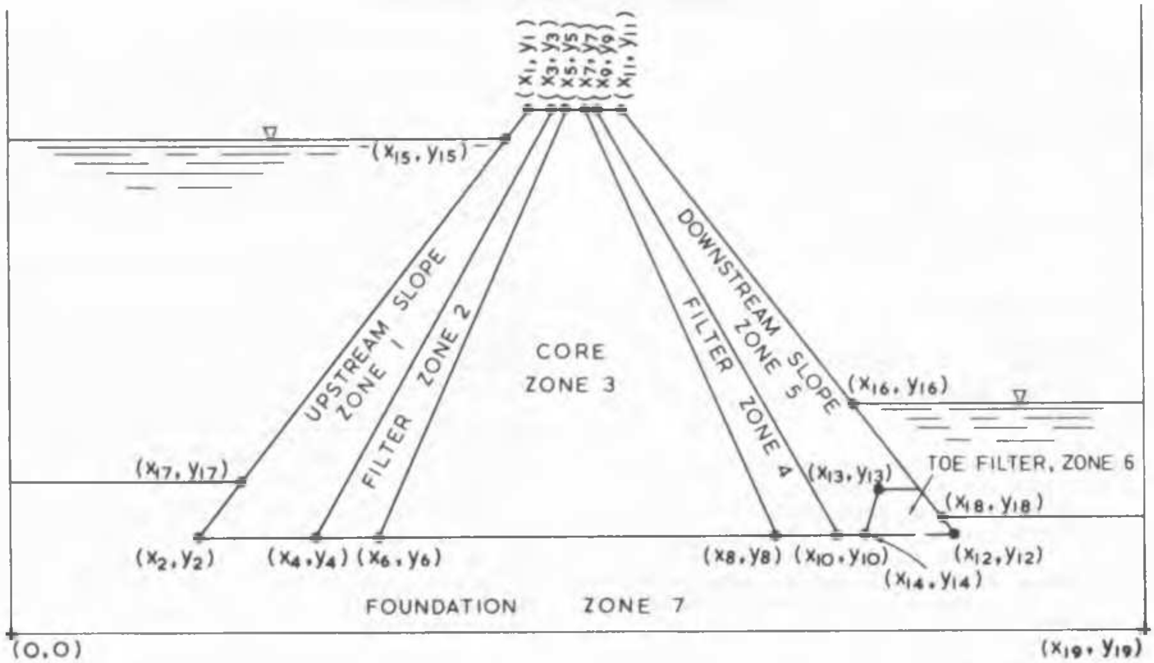

Fig.2. General Geometric Representation Of The Embankment Used In The Analysis

After reconstruction of the damaged portion of the dam, it was considered wise to monitor conditions in the clay core when the dam was refilled and brought into service. Consequently eleven Casagrande type piezometers were installed in the clay core at the cross-section of maximum height sufficiently distant from the reconstructed zone to be free from any interaction (Ingles et al., 1968). Piezometers numbered 1 to 6 were installed in July 1964, and, based on the information obtained from these, those numbered 7-11 were installed in October 1967. Details of these installations, observations and methods of interpretation have been described elsewhere (Richards, 1968c), but their position is clearly shown in Figure 4.

At frequent intervals over a four year period measurements have been taken of the water levels in the piezometer tubes installed in the dam. After any initial equalization movements had settled, these observations were used as a direct measure of the positive pore water pressure in the clay core at the position of the tip and at a particular time. Figures 4 and 5 show the variations of pore pressure during this time.

Undisturbed soil samples taken from the holes made for the installation of the second set of piezometers were laboratory tested for bulk density, water content and soil suction. Both the total and solute suctions (Aitchison et al. 1965) were determined by the psychrometric technique (Richards, 1968a) and hence the matrix suction or pore pressure found by difference using the total suction corrected for overburden pressure, assuming unity for the pore pressure parameter, $\bar{B}$. Both the uncorrected total suction and the calculated

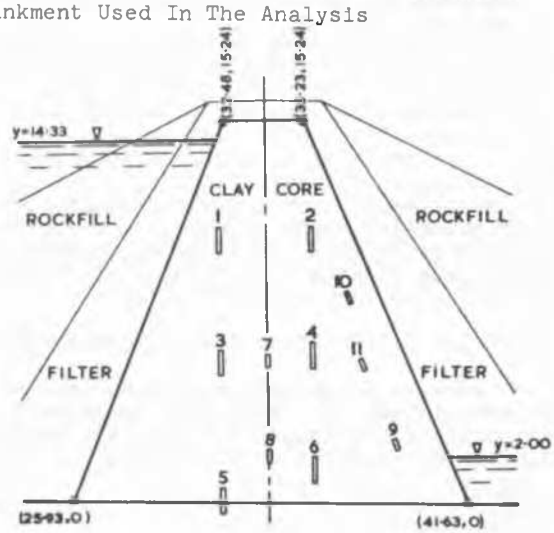

Fig.3. Clay Core Of Flagstaff Gully Dam

pore pressure are shown in Figures 6(a) and 6(b).

When these results are compared with direct pore pressure measurements, as in Figure 5 (b), good agreement is seen for the position of the "phreatic" surface on the centreline of the dam (Holes 7, 8). On the downstream face of the clay core (Holes 9, 10), the agreement was not so good, due possibly to the inherent inaccuracies in the corrections made for overburden pressure and solute suction, and to uneven infiltration of rainfall at the downstream face of the core.

In order to complement the field and

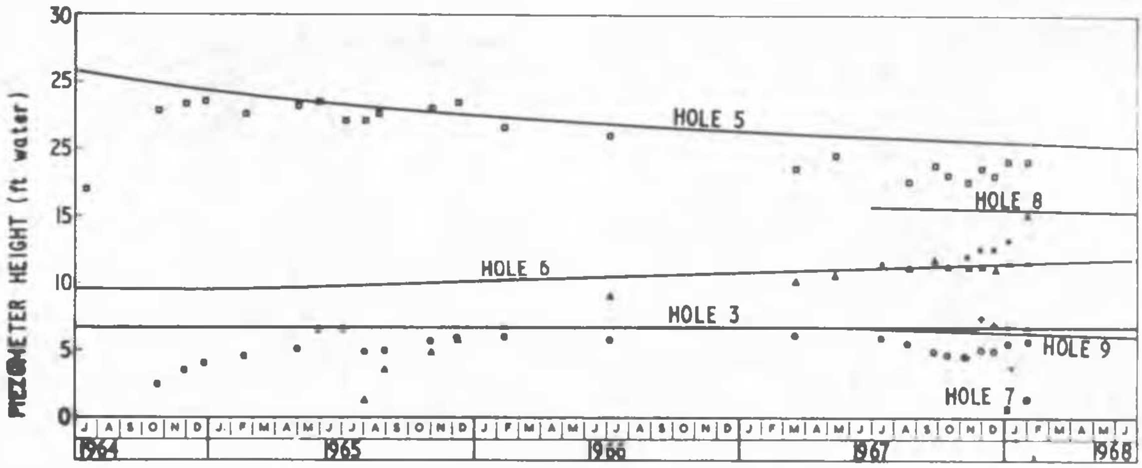

Fig.4. Piezometric Level Observations

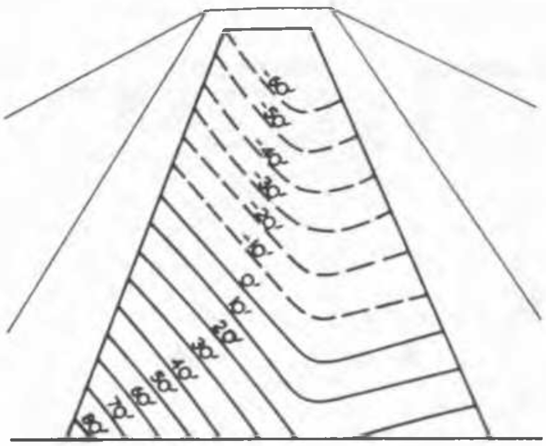

Fig.5(a) Initial Comparison

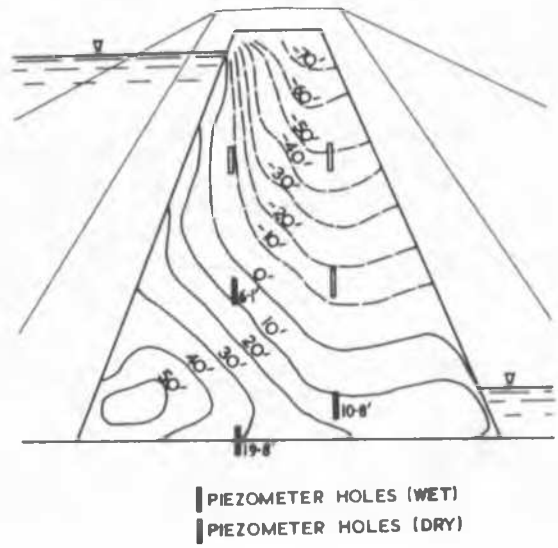

Fig.5(b) Mid-1967 Comparison

laboratory observations it was decided to make a mathematical analysis of the flow conditions using the method of analysis described beforehand. The initial pore pressures shown in Figure 5(a) were set up by a mathematical formulation to give a reasonable fit with the observed data. Although this formulation was only empirical, it was equivalent to assuming an initial compaction pore pressure of -2.85 Kg./cm² (-40.5 p.s.i.), $\bar{B}$ equal to unity and a height of overburden and water approximately equal to the actual heights. Improved stress distribution analyses applied to this problem would therefore be a necessary prerequisite for determining the initial conditions in any analysis of this type prior to commencement of construction.

The geometric representation of the problem is shown in Figure 3. In this case, the clay core only was analysed, assuming that the slope and filter materials were infinitely more permeable than the core, i.e. the upstream and downstream water levels were rapidly established against the faces of the core. The base of the core was also taken as the lower boundary of the mesh network at which two boundary conditions were assumed, viz.

1. the foundation material was impervious, allowing no flow across this boundary;

2. the foundation material was extremely pervious with a trapezoidal or linear pore pressure distribution on the base of the core.

Alternative estimates of the permeability at approximately zero pore pressure were obtained from four different types of measurement and together with the final adopted figure are summarized in Table I.

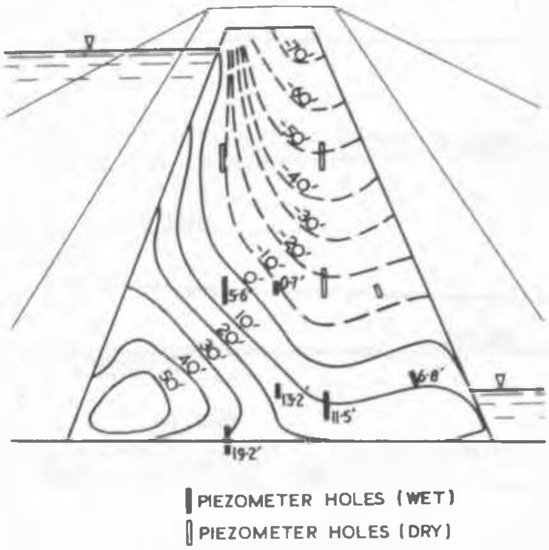

Fig.5(c) January 1968 Comparison

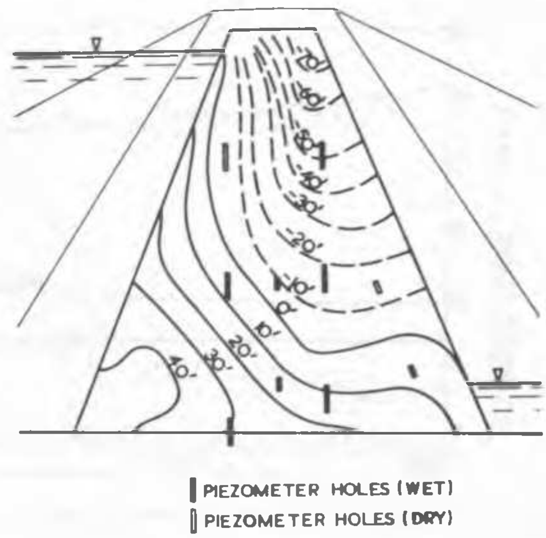

Fig.5(d) December 1971 Comparison

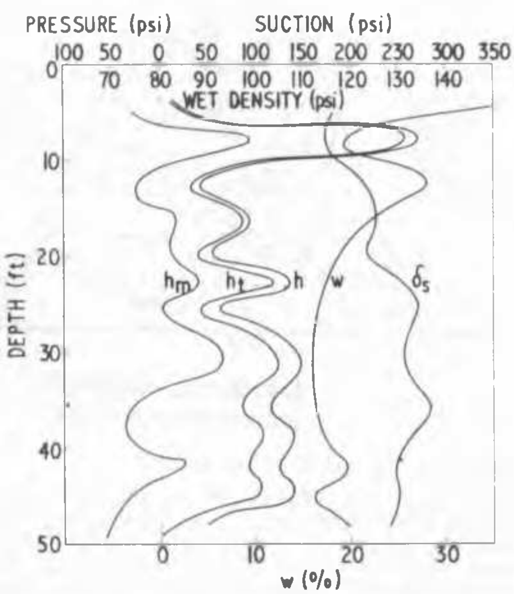

Fig.6(a) Vertical Profile at Centreline

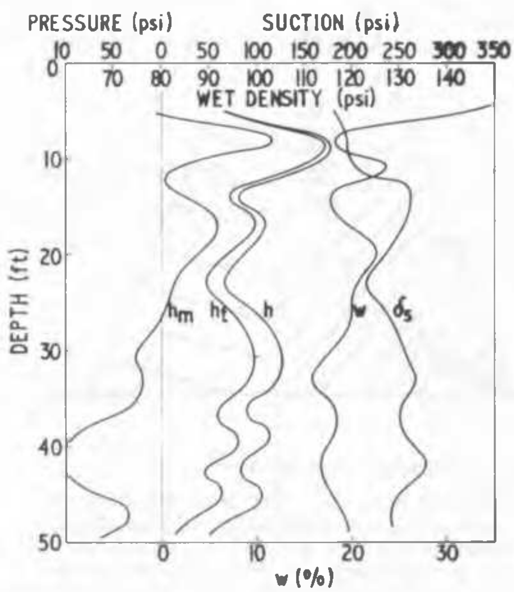

Fig.6(b) Sloping Profile through Centreline

Permeability Values Table I

Method	Permeability (cm./sec.)
Laboratory permeability test	3×10^{-7} to 3×10^{-8}
Field infiltration	1×10^{-4} to 1×10^{-6}
Initial piezometer response	1×10^{-7} to 5×10^{-8}
Piezometer recovery response	1×10^{-6}
Computer analysis	2×10^{-6}

These values show considerable variation and as a starting point, a value of 1 x 10^{-8} cm./sec. was assumed and the full k(u) function was chosen based on experience with a similar clay from the Horsham area, Victoria (Richards, 1965). The final k(u) and $\frac{\partial u}{\partial \theta}$ functions adopted are shown in Figure 1.

RESULTS

The assumption of an impervious foundation was first used, as this supposedly was the design criteria used in the design of the dam. The results obtained suggested a reasonable agreement upstream, but poor agreement downstream. Due to this result, the fact that the dam core had failed in piping at the junction of the core and the fissured bedrock and the significant seepage and rise in water table downstream of the dam, the second assumption of trapezoidal pore pressure distribution on the base of the core was tried. This gave remarkably good agreement (within a few feet head of water) between the calculated and observed pore pressures at all points in the core over the period of observations. However, the results from piezometer No.5. do suggest that this assumption could have been even further improved if necessary.

Furthermore, the observed rates of change of the pore pressures as shown in Figure 4 were matched with the calculated values. The parameters best fitting the observed data were obtained by trial and error and are shown in Figure 1, giving the value of k(u) at zero pore pressure of 2 x 10^{-6} cm./sec. in Table I. This was compatible with the values determined by the other methods.

The results of the final analysis using the parameters in Figure 1 and the latter boundary condition at the base of the core, are summarized in Figures 4 and 5. These figures indicate the excellent agreement that is possible using this method of analysis with suitable flow parameters and boundary conditions, but it must be realized that such good agreement would not be possible without the knowledge of the performance of the dam, as in this case. Consequently, it is not so much the mathematical analysis, but the understanding and measurement of the flow parameters and boundary conditions, which require further development. However, this is not to suggest that useful information of flow conditions in earth dams cannot be obtained at this time with our present level of understanding of this problem.

CONCLUSIONS

The method described in this paper for the mathematical analysis of moisture flow conditions in earth dams does provide a practical method for the prediction of future pore pressure distributions in these structures. When tested against the observed conditions in the Flagstaff Gully Dam it was shown to be capable of giving excellent results. However, its usefulness in the design of earth dams in general will be limited by our understanding of the relevant flow parameters and boundary conditions.

The results of analyses of real practical problems do suggest that:

1. The dissipation of initial construction pore pressures and the establishment of seepage flow is extremely slow in clay cores.

2. The transient flow patterns bear no relation to the ultimate flow pattern determined by conventional flow nets.

3. The ultimate flow pattern is not necessarily that predicted by conventional flow nets, particularly in respect to the significant flow possible in the region of negative pore pressures.

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