Part II Effect of the Pore Pressure on the Stability of Earth Embankments

1. Introduction

The stability analyses of an earth embankment with respect to the pore pressure consist of the following items ;

1) Stability analyses of the dam and the foundation, respectively, during construction (due to the non-steady pore pressure).
2) Stabitity analysis of the dam for rapid-drawdown of the reservoir (due to the transient pore pressure).
3) Stability analyses of the dam and the foundation, respectively, for full reservoir (due to the steady seepage pressure).

For the last item, the analyses can be easily performed by drawing a flow-net for the steady stream flow, and finding the statical head at each point in the figure. The author has pointed out, through the experimental studies of percolating flow using sand models, that the local failure near the upper portion of the surface of seepage is predominant, in the case of the water-retaining embankment which consists of cohesionless materials. In these studies, performing the theoretical investigations concerning these experimental results, a proposed equation which gives the critical hydraulic gradient has been deduced for this kind of local failure, whereby the mechanism of the actual failure of embankments can be analysed[11] Further experimental research has been succeeded towards the disaster preventive methods of embankment design, and a reasonable criterion has been given adequately[12]

Among the first item, the author has made a theoretical study for the stability analysis of the dam foundation during and just after construction, in connection with the execution control of fill work[13] In this study, starting from the fundamental theoretical equation of the two-dimensional consolidation, the distribution of the pore pressure in the embankment foundation has been studied theoretically. Next, by using the above solutions, the plasticity load in the foundation where the pore pressure exists has been obtained throughout the numerical calculation.

Thus the remaining problems, the pore pressure coming into existence in the earth embankment due to the consolidation of the fill material during construction, and the residual pore pressure after rapid-drawdown of the reservoir, are treated as the second part of this paper.

2. Distribution of Pore Pressure during or Just after Construction

(1) General consideration.

In the embankment composed of relatively impervious materials as the earth dam, the pore pressure occurs in itself due to the consolidation of soil during construction[14]. It seems somewhat difficult to estimate the distribution of the pore pressure theoretically, comparing with that of the embankment foundation which has been tried in the preceding paper. The reason is, that the geometry of the surface of earth fill where the boundary conditions should be given varies with the construction process, and that it is not easy to obtain the solution of two-dimensional consolidation, even if the shape of embankment is very simple. As in some conventional studies for estimating the pore pressure during construction of the earth embankment, bold assumptions have been used to simplify the complicated problem, these theoretical solutions are not always satisfactory when compared with the measured data observed in the field[15][16]

In the author's method, the consolidation process is treated as a heaping phenomenon by the sedimentation of unsaturated soils, and he applies the approximate solution using a parabolic pressure curve, which has been proposed by Terzaghi-Fröhlich[17] for the mechanism of consolidation of a clay layer whose thickness varies with time. Though more rigorous solution has been deduced for the mechanism of consolidation of a clay layer[18], it is not necessary to be so rigorous as this in practice.

In the solution which has been proposed by Terzaghi-Fröhlich for the consolidation of sedimentary clay layer, the following fundamental assumptions are involved :

1) one-dimensional consolidation only upwards vertically is treated,
2) coefficient of consolidation $c=k/\gamma_w m_v$ is constant during the sedimentary period,
3) velocity of sedimentation is constant,

4) pores of clay layer are fully saturated, as the sedimentation in water is considered, and
5) excess pore pressure curve is assumed as a parabola approximately.

Because of these assumptions, it is not possible to apply the above solution directly to the consolidation of earth fill during construction without some modifications. Assumptions 2), 3) and 5) are satisfied in this case, whereas it is clear that the first assumption of one-dimensional consolidation and the fourth of fully saturation are not established in the construction of embankment. In the author's method, therefore, to modify these assumptions, he considers the compressibility of containing air in the soil mass due to the unsaturation and the effect of compacting action represented by the coefficient of earth pressure at rest due to the unisotropy of stresses in the earth fill, during one-dimensional consolidation until the completion of filling. For the behavior of dissipation of the pore pressure after the completion of embankment, it is possible to compute the distribution of the pore pressure in the embankment by the approximate step-by-step method, transforming the fundamental equation of the two-dimensional consolidation into the finite differential equation.

(2) Application of Terzaghi-Fröhlich's theory.

Applying the approximate solution using a parabolic pressure curve, which has been proposed by Terzaghi-Fröhlich, to a clay layer having the initial trigonometric excess pressure, whose apex is on the upper permeable boundary, the following solutions are obtained, referring Fig. 1.

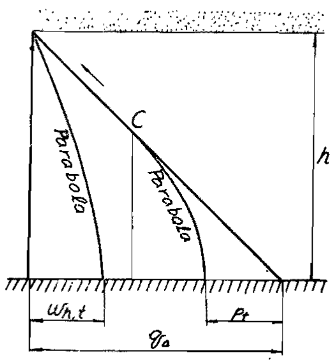

Fig. 1 Parabolic pore pressure curve to a clay layer of initial trigonometric excess pressure in Terzaghi-Fröhlich's theory.

$$\left.\begin{aligned}
&1)\quad 0 \leqq t \leqq \frac{1}{6}\frac{h^2}{c}, \qquad 0 \leqq y \leqq (h-\sqrt{6ct}) : \\
&\qquad w(y,\, t) = \frac{y}{h} q_\varDelta, \\
&2)\quad 0 \leqq t \leqq \frac{1}{6}\frac{h^2}{c}, \qquad (h-\sqrt{6ct}) \leqq y \leqq h :
\end{aligned}\right\} \qquad \cdots\cdots\cdots\cdots(1)$$

$$w(y,\ t)=q_{\Delta}\left[1-\frac{1}{h}\sqrt{\frac{3ct}{2}}\left\{1+\frac{(h-y)^2}{6ct}\right\}\right],$$

3) $\frac{1}{6}\frac{h^2}{c}\leqq t\leqq\infty,\qquad 0\leqq y\leqq h$:

$$w(y,\ t)=\frac{q_{\Delta}}{2}\left\{1-\frac{(h-y)^2}{h^2}\right\}\exp\left\{-\left(\frac{3ct}{h^2}-\frac{1}{2}\right)\right\}.$$

In the consolidation of sedimentary clay layer, the initial distribution of pore pressure can be regarded as a triangle, whereby the above-mentioned solution is applicable. The depth of layer h is, however, not constant but increases with time, being represented as h_t. On the other hand, the tangential point C on the pressure curve in Fig. 1 rises upwards with time, and the vertical component of its velocity is

$$v_c=\frac{dh_c}{dt}=\sqrt{\frac{3c}{2t}},\qquad \cdots\cdots\cdots\cdots\cdots\cdots(2)$$

and the constant velocity of sedimentation is

$$v_s=\frac{dh_t}{dt}=\text{const.}\qquad \cdots\cdots\cdots\cdots\cdots\cdots(3)$$

The difference of these two values divides the mechanism of consolidation. As is seen from Eq. (2), the velocity of consolidation v_c is independent of the thickness of clay layer ; v_c is infinite when $t=0$, and decreases with time. From Eq. (1) it can be seen that Eq. (2) holds only when $0\leqq t\leqq\frac{1}{6}\frac{h^2{}_t}{c}$. So the minimum v_c is

$$v_{c,min}=\frac{3c}{h_t}.\qquad \cdots\cdots\cdots\cdots\cdots\cdots(4)$$

From the above consideration, it is concluded that, when $v_s>v_{c'min}$ there is no effect of increasing thickness of layer on the consolidation. In other words the conclusion is that, there is a definite relation between the velocity of sedimentation v_s [cm/sec] and the coefficient of consolidation c [cm^2/sec], where in $v_s>3c/h_t$, the increase of thickness of the layer has no effect on the consolidation, and in the initial state of $v_s<3c/h_t$, the both interact each other. Investigating this relationship between sedimentation and consolidation, Terzaghi-Fröhlich have shown, from the character of the parabolic pressure curve, that the time t_2 after which the mutual interference diminishes is :

$$t_2=1.41\,t_1=1.41\frac{3c}{v_s{}^2}\qquad \cdots\cdots\cdots\cdots\cdots\cdots(5)$$

The author draws Fig. 2 to facilitate our understanding of the above-mentioned relationship. The figure illustrates the relation between time : t, height of sedimentation : h_t, height of point C : h_c, velocity of sedimentation : v_s, velocity of consolidation : v_c, $v_{c,min}$, and intergranular pressure : p_t. As described above, at the initial state of sedimentation, as the velocity of consolidation v_c is greater than that of sedimentation v_s, the both interfere with each other till t_2 which is given in Eq. (5), and only in the case of $t \geqq t_2$, the increase of the depth of layer has no effect on the consolidation of soils. Consequently, in order to make the computation of consolidation more precisely, it is necessary to make the modification to the time factor as large as $\Delta t = 0.41\, t_1$.

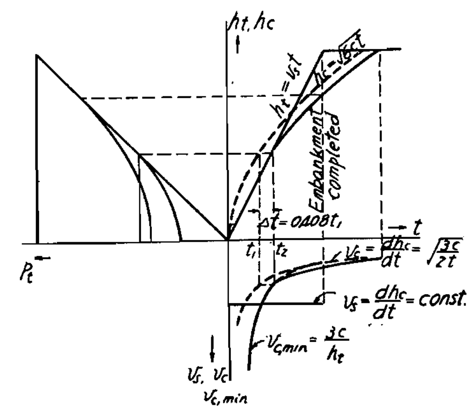

Fig. 2 Correlation between several variables during and just after construction of embankment.

(3) **Pore pressure during construction.**

Neverthless, it seems to be of little use to apply this consideration to the consolidation of fill material of the actual embankment. The reason is that, in the construction of the earth embankment, as the speed of filling v_s is pretty large, compared with the coefficient of consolidation c, in contrary to the geological sedimentation of earth ground, the time t_2 given by Eq. (5) is very small, accordingly the modification of the time factor is still small in practice. For example, on Fresno Dam (in U.S.A.) to which the author tries numerical calculation, whose height $h_f = 15$ m, construction period $t_f = 3$ months, speed of filling $v_s = 15\text{m}/3\text{ months} = 1.93 \times 10^{-4}$ cm/sec, and coefficient of consolidation $c = 1 \times 10^{-3}$ cm^2/sec (assuming this value from the dam material of clay gravel), we obtain from Eq. (5) $t_2 = 1.14 \times 10^5$ sec $= 31.6$ hr, being very small compared with the whole construction period of $t_f = 3$ months, and $\Delta t = 0.41\, t_1 = 3.30 \times 10^4$ sec $= 9.16$ hr only.

Fig. 3 (a) shows the result of calculating the distribution of pore pressure in Fresno Dam just after completion of 3 months' construction, under the

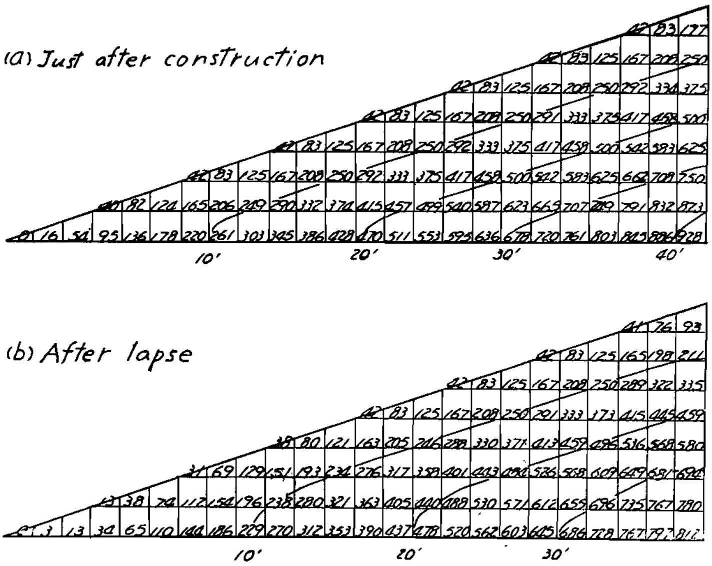

Fig. 3 Distribution of pore pressure in the embankment of 1 : 3-slope ; (a) just after 3-months' construction and (b) after 1-year from completion.

above-mentioned consideration. At the calculation, the left half of the symmetrical dam is divided into 24×8 lattices, and the pore pressure at every nodal point just after dam construction is obtained, using Eq. (1) to each vertical column thus divided. Values marked beside each nodal point represent the pore pressure, using the dam height as the measuring unit. In this figure the time factor is

$$T_v=\frac{c(t_f-\Delta t)}{h^2}=\frac{c(t_f-0.41\,t_1)}{h^2}=3.44\times10^{-3}.$$

Provided that the overall pore pressure coefficient $\bar{B}$, which has been introduced by Skempton-Bishop, i. e.

$$\frac{\Delta w}{\Delta\sigma_1}=\bar{B}=B\left\{1-(1-A)\left(1-\frac{\Delta\sigma_3}{\Delta\sigma_1}\right)\right\} \qquad \cdots\cdots\cdots\cdots\cdots\cdots(6)$$

is adopted $\bar{B}\simeq0.6$ for clay gravel, considering the unsaturation of fill material and the unisotropy of stresses in the embankment as explained later, and assuming the unit weight of fill material $\gamma_s=1.7\,\mathrm{g/cm^3}$, equi-pressure lines in Fig. 3(a) are shown in piezometric height, whereas the measured data in the field are given in Fig. 4(a).

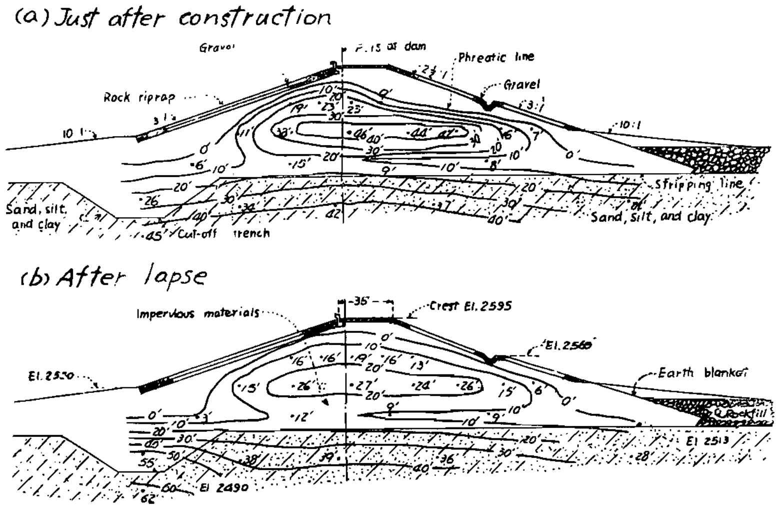

Fig. 4 Measured data of the distribution of pore pressure in Fresno Dam due to construction ; (a) just after 3-months' construction and (b) after 1-year from completion.

(4) Pore pressure after construction.

When the distribution of pore pressure in the embankment just after completion has obtained as above, then the behavior of dissipation of the pore pressure is known as follows ; the fundamental differential equation of two-dimensional consolidation

$$\frac{\partial w}{\partial t} = c\left(\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2}\right) \qquad \cdots\cdots\cdots\cdots\cdots\cdots(7)$$

is transformed into the following finite differential equation :

$$\left.\begin{aligned} &w_0(t+\delta t) = \beta(w_1 + w_2 + w_3 + w_4 - 4w_0) + w_0(t)\ , \\ &\text{where} \quad \beta = \frac{c \cdot \delta t}{(\delta h)^2}, \qquad \delta x = \delta y = \delta h \end{aligned}\right\} \qquad \cdots\cdots\cdots(8)$$

The smaller the distance between nodal points of lattices δh and also the time interval δt are taken, the more precise the solution of this equation becomes[19]

To know the distribution of residual pore pressure in Fresno Dam after one year from completion (i. e. after fifteen months from starting of the construction), the numerical calculation is performed. Using $\delta h = h_f/8 = 187.5$ cm,

$\delta t = 1$ month $= 2.59 \times 10^6$sec, i. e. $\beta = c \cdot \delta t/(\delta h)^2 = 0.0737$, the approximate step-by-step method of twelve reiteration has been succeeded for all 108 nodal points. The result is given in Fig. 3(b) and the observation in the field is shown in Fig. 4(b). Comparing with each figure, owing to the existence of some permeable stratum adjacent to the dam base in the latter, the dissipation of pore pressure is sooner than the former.

As is seen by Fig. 3 and Fig. 4, the high pore pressure remains in the embankment just after completion or for a considerable period after that. The author has tried a theoretical study on the execution control of fill work on a soft foundation, as summarized in **1.**, but it is also necessary to study the same consideration even when the fill material consolidates itself. Though the above Fresno Dam was finished in three months actually, if the construction would last over one year slowly, the distribution of pore pressure is shown in Fig. 5 (a) for just after completion and in Fig. 5(b) for three months after (i. e.

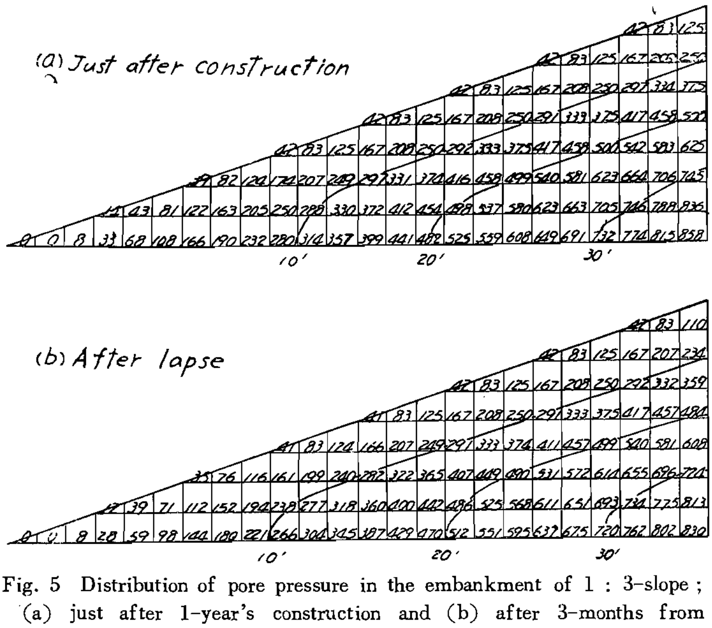

Fig. 5 Distribution of pore pressure in the embankment of 1 : 3-slope ; (a) just after 1-year's construction and (b) after 3-months from completion.

after fifteen months from starting of the construction), in which the pore pressure is pretty small except in the central part of the embankment. In this virtual case, the speed of filling is $v_s = 15$ m/1 year $= 4.74 \times 10^{-5}$ cm/sec, and the time factor at the completion is $T_v = 1.36\times10^{-2}$. At the calculation of Fig. 5 (b), $\beta = 0.0737$ is used as the same as in Fig. 3(b).

As is described in the preceding part of this paper, the large amount of residual strength is reserved in the central part of the wedge-shaped embankment. As in the slope regions, however, the reserve is very small, in the case of slow dissipation of pore pressure due to the high speed of filling, it is necessary that the execution speed is lowered to decrease the pore pressure, as is shown in the above numerical calculations.

(5) Determination of pore pressure coefficient.

Pore pressures coming into existence during construction under consideration and after rapid-drawdown of the reservoir described later, occur by the change of the principal stresses in the earth embankment. Skempton[20] has given next representation of this relationship.

$$\Delta w = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)], \qquad \cdots\cdots\cdots(9)$$

wherein A, B are the pore pressure coefficients. As is shown in Fig. 6, the

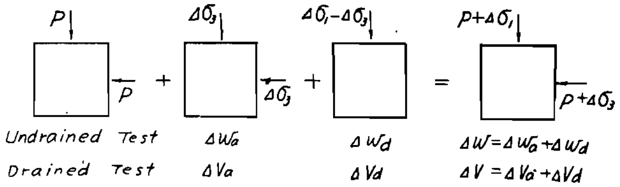

Fig. 6 Pore pressures in the triaxial test.

change of pore pressure Δw can be considered as the sum of the pore pressures by the change of ambient stress $\Delta\sigma_3$ and of deviator stress $(\Delta\sigma_1 - \Delta\sigma_3)$. Thus Eq. (9) is :

$$\Delta w = \Delta w_a + \Delta w_a, \qquad \cdots\cdots\cdots(10)$$

and the pore pressure coefficients are :

$$\left.\begin{aligned} B &= \frac{\Delta w_a}{\Delta\sigma_3}, \\ A\cdot B &= \frac{\Delta w_d}{\Delta\sigma_1 - \Delta\sigma_3} \end{aligned}\right\} \qquad \cdots\cdots\cdots(11)$$

In computing the pore pressure during the fill construction or after rapid-drawdown of the reservoir, Eq. (9) is conveniently transformed into Eq. (6) by Bishop[21] $\bar{B}$ in this equation is called the overall pore pressure coefficient.

Generally the coefficient A is smaller than unity and in many cases is nearly equal to zero. As $\Delta\sigma_3 < \Delta\sigma_1$ in the earth embankment under construction, the overall coefficient $\bar{B}$ is smaller than the coefficient B.

The author has measured the overall pore pressure coefficient $\bar{B}$, using the triaxial compression apparatus for three kinds of soil ; coarse sand, silty sand and disturbed clay. In the measurement, the applied stress has been increased in two stages so as to use Eq. (9). The procedure is that, as is shown in Fig. 6, in the first stage, the specimen is consolidated under the all-round effective pressure p and then the change of the pore pressure Δw_a accompanied with the application of the change of the ambient pressure $\Delta\sigma_3$ under the undrained condition is measured. In the second stage, the change of deviator stress $(\Delta\sigma_1 - \Delta\sigma_3)$ is given to the specimen, and the corresponding change of the pore pressure Δw_a is measured. The measurement of the pore pressure has been performed by the porous pilot installed in the middle height of the specimen, which has been led to the manometer of no-flow type. The result is tabulated in Table 1.

Table 1 Pore pressure coefficient measured by triaxial tests.

	p(kg/cm^2)	$\Delta\sigma_3$(kg/cm^2)	Δw_a(kg/cm^2)	B	$\Delta\sigma_1$(kg/cm^2)	Δw_a(kg/cm^2)	$\bar{B}$
Coarse sand	2.0	0.2	0.10	0.50	0.50	0.08	0.36
Silty sand	2.0	0.2	0.03	0.15	0.40	0.02	0.125
Clay (disturbed)	2.0	0.5	0.45	0.90	0.95	0.20	0.68

The overall pore pressure coefficient $\bar{B}$ is much larger in clay than in sandy soil. For the clay, $\bar{B}=0.68$ has been obtained in the remolded state. This specimen is the pure clay from Ōsaka alluvial stratum. As the fill material in the actual embankment is considered to have greater permeability, $\bar{B}=0.6$ has been adopted in the preceding numerical calculations.

(6) Stability computation.

The author has performed the stability computation for an earth embankment just after construction, which has a symmetrical section of 1 : 3-slope, shown in Fig. 3. Fig. 7(a) shows the distribution of pore pressure in the embankment, which is drawn from Fig. 3(a), taking the overall pore pressure coefficient $\bar{B}=0.6$ as is described above. Assuming the mechanical properties

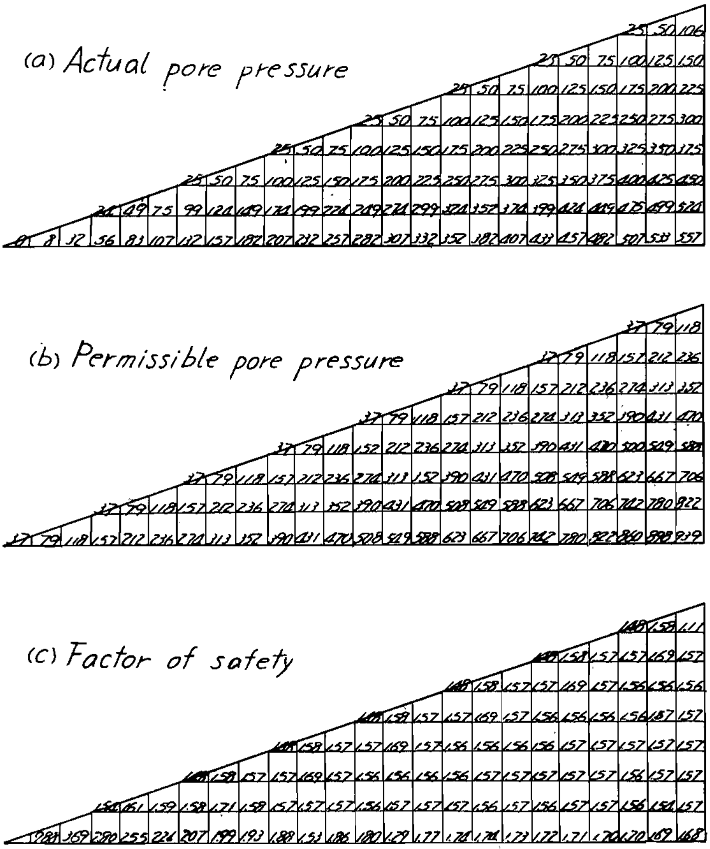

Fig. 7 Distribution of (a) actual pore pressure, (b) permissible pore pressure and (c) factor of safety in the embankment of 1 · 3-slope just after construction.

of the fill material as $K_1=1.0$, $\tan\varphi=0.5$ ($\varphi=26°34'$) and $C=0$, the permissible or critical pore pressure is shown in Fig. 7(b), calculating from the residual shearing resistance explained in the preceding part of this paper. The ratio of the permissible pore pressure (Fig. 7(b)) to the actual pore pressure (Fig. 7(a)) gives a factor of safety of the embankment just after construction, with respect to the failure due to pore pressure. Fig. 7(c) shows this value, where it is known that, in spite of the great magnitude of residual strength in the central part of the embankment, the factor of safety is small in this part, because the pore pressure occuring during construction has a considerable value.

In this calculating example, in order to compare with the measured data on Fresno Dam, the construction period is assumed three months for which Fig. 3(a) is applicable. But the larger period of construction gives higher factor of safety than that of Fig. 7(c).

3. Distribution of Residual Pore Pressure after Rapid-Drawdown of the Reservoir

(1) Pore pressure and residual strength after rapid-drawdown.

Among the various data of earth dam failure, sloughing of upstream slope due to rapid-drawdown of the reservoir is one of the most predominant causes, whereof many old-typed French earth dams have been destroyed[22]. This type of failure is not disastrous generally, but it is costly for restoration.

In this case, the reason why the earth embankment becomes unstable is that, in consequence of having discharged the water load, the residual strength is being decreased remarkably, by the decrease of total normal stresses and of the insufficient decrease of the pore pressure that should be accompanied with that of the former.

The author draws Fig. 8 to explain the mechanism of this earth dam

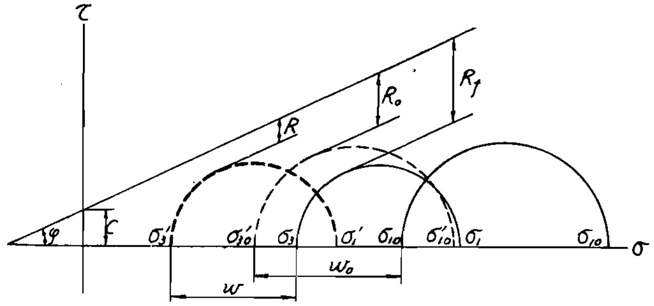

Fig. 8 Mohr's stress circles and residual strengths in the embankment before and after drawdown.

failure. This figure shows Mohr's circles at any point in a soil element near the upstrem slope, from which the position of stress circle and the residual strength just after drawdown and in the following period are cleared, referring Table 2.

As is shown in Fig. 8, at the instance of rapid-drawdown, it is known that the residual strength in the embankment decreases, resulting that the struc-

Table 2 Relation between stresses, pore pressure and residual strength in the earth embankment before and after drawdown.

	Before drawdown	Just after drawdown	After lapse
Total stresses	σ_{10}, σ_{30}	σ_1, σ_3	σ_1, σ_3
Pore pressure	w_0	w	$w \rightarrow 0$
Effective stresses	σ_{10}', σ_{30}'	σ_1', σ_3'	$\sigma_1' \rightarrow \sigma_1$, $\sigma_3' \rightarrow \sigma_3$
Residual strength	R_0	$R(<R_0)$	$R \rightarrow R_f$

ture approaches to its dangerous state.

Though the intensity of the residual pore pressure is variable by the geometry of the embankment, physical properties of the material and the degree of compaction, it is possible to express these factors by a unique coefficient. The residual pore pressure just after rapid-drawdown is given in the following form, referring Fig. 9 :

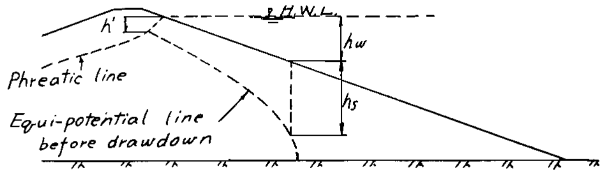

Fig. 9 h_s, h_w and h' before drawdown.

$$w=\gamma_w\{h_s+h_w(1-\bar{B})-h'\} \qquad \cdots\cdots\cdots\cdots(12)$$

As is seen from Eq. (12), the smaller the value of the overall pore pressure coefficient $\bar{B}$ is, the larger the pore pressure w remains, and the embankment becomes unstable, whereas from Eq. (6) when A=1, $\bar{B}=B=1$ and for $A<1$, $\bar{B}>B$, resulting $\bar{B}>1$. As a safe side, taking $\bar{B}=1$ follows :

$$w=\gamma_w(h_s-h') . \qquad \cdots\cdots\cdots\cdots(13)$$

This gives the residual pore pressure of the worst case. It should be noted that in this case, B is less than unity but, as its magnitude depends on the sign of the change in stress, the values of A and B measured in the conventional undrained test with increasing principal stresses as is described in **2. (5)** are not applicable.

In order to estimate the residual pore pressure after rapid-drawdown of the reservoir from Eq. (13), the potential drop h' in Fig. 9 should be known at each situation in the earth embankment, drawing the flow-net of steady seepage flow at full reservoir. There are various means of establishing the flow-net in the embankment, among which the author applies the relaxation method, which has been developed by Southwell[23], to Fresno Dam described in the preceding article. Fig. 10 illustrates the distribution of residual pore pressure

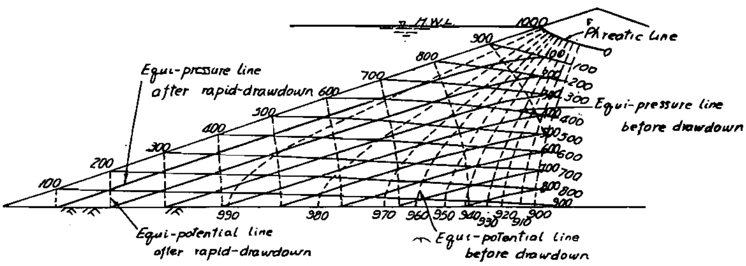

Fig. 10 Distribution of residual pore pressure in the upstream region of the embankment of 1 3-slope just after rapid-drawdown.

in the upstream region just after rapid-drawdown calculated by Eq. (13). In the figure, equi-potential lines and equi-pressure lines before drawdown and equi-potential lines after drawdown are marked, in connection with equi-pressure lines after drawdown under consideration.

From the above calculation it is known that, at the instance just after rapid-drawdown of the reservoir, in spite of the remarkable decrease of total normal stresses in the upstream region, pretty large amount of the residual pore pressure exists, making the reserve strength in the embankment extremely small. To prevent the dam failure by this effect, it can be known that, as well as the installation of an adequate filter-drain at the toe of upstream slope to decrease the pore pressure, the definition of a permissible speed of the drawdown of the water level, or the limitation of partial discharge instead of integral discharge is an important factor in the design and the hydraulic treatment of the earth embankment[24]

(2) **Stability computation.**

For example, the stability computation after rapid-drawdown of the reservoir for an earth embankment is performed, which has a symmetrical section of 1: 3-slope shown in Fig. 3. From the above-mentioned consideration, the overall pore pressure coefficient $\bar{B}$ corresponding to the stress change of drawdown is adopted $\bar{B}=1$ for the sake of safety. Fig. 11(a), (b) shows the dis-

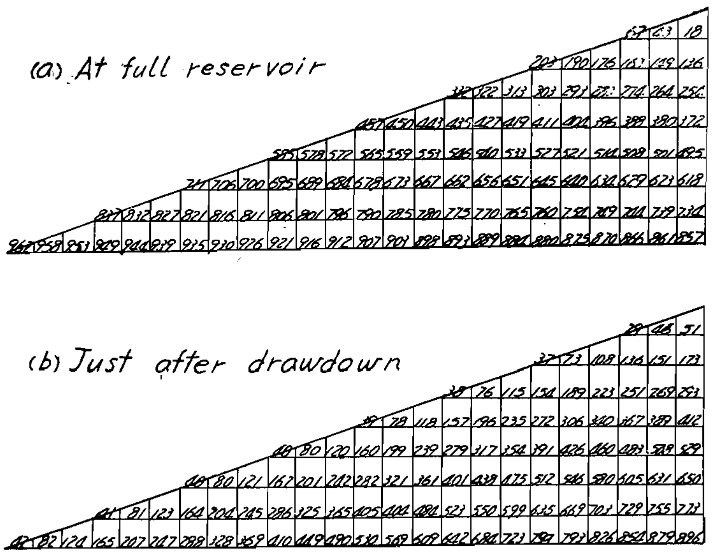

Fig. 11 Distribution of actual pore pressure in the embankment of 1 : 3-slope ; (a) at full reservoir and (b) just after drawdown.

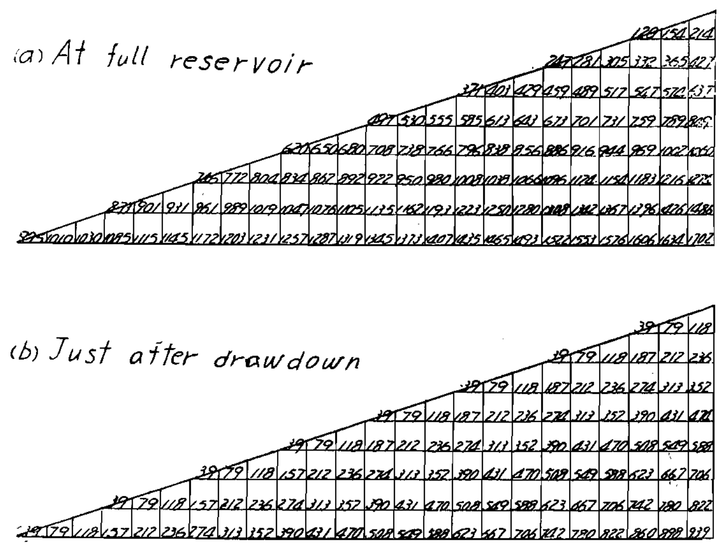

Fig. 12 Distribution of permissible pore pressure in the embankment of 1 : 3-slope ; (a) at full reservoir and (b) just after drawdown.

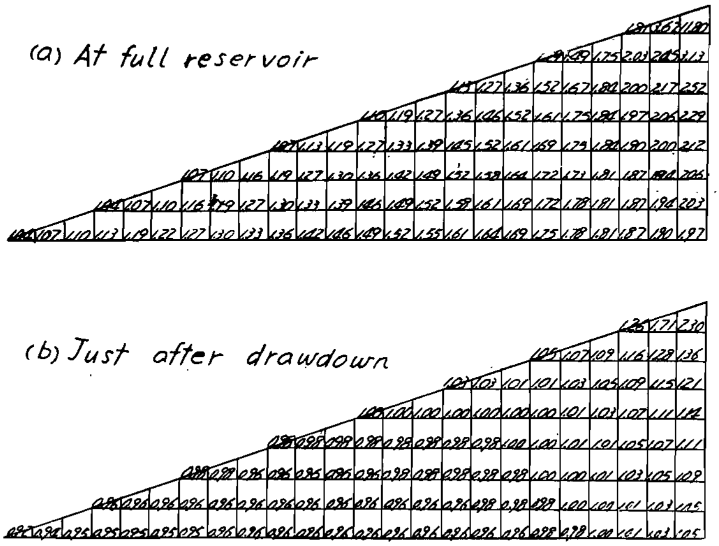

Fig. 13 Distribution of factor of safety in the embankment of 1 3-slope ; (a) at full reservoir and (b) just after drawdown.

tribution of the pore pressure before and just after drawdown, respectively, copying from Fig. 10. Fig. 12(a), (b) is the distribution of the permissible pore pressure calculated from the residual shearing resistance, using tan $\varphi=0.5$ ($\varphi=26°34'$) and $C=0$ ($K=1.0$ for (a) and $K=0.7$ for (b)). In Fig. 13(a), (b), the factor of safety of the embankment before and after drawdown is shown, respectively, which represents the ratio of the permissible pore pressure in Fig. 12 to the actual pore pressure in Fig. 11.

As is seen from calculating example, the factor of safety at the upstream toe just after drawdown decreases extremely, resulting the danger of sloughing to be approached. Therefore, the stability of the embankment against rapid-drawdown should be established by the installation of the filter-drain at this part, or by the reasonable treatment of discharging the reservoir.

4. Conclusion

In the present part of this paper, among the effects of the pore pressure on the stability of earth embankments, an analytical study is performed for the pore pressure coming into existence due to the consolidation of fill material during construction, and for the residual pore pressure after rapid-drawdown of the reservoir, respectively. In the result, it is known that such pore pressures have a serious effect on the stability of the earth embankment, according to the numerical calculations for the above both criteria. And the conclusion is that, as well as the design of the adequate drainage to accelerate the dissipation of the pore pressure, the critical permissible pore pressure must be checked in the execution control of earth embankment and the treatment of drawdown of the reservoir.

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