Stresses and Deformations in Cores of Rockfill Dams

Contraintes et déformation dans les noyaux d'étanchéité des barrages en enrochements

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Summary

The clay core in a rock fill dam has a tendency to settle more than the rock shells, thus causing shear stresses to be developed on the contact planes with the shells. Expressions for the stresses are derived, assuming conditions of plastic equilibrium on the contact plane between core and shell. Reduced stresses in the core and some equalization of settlement result.

The interference with free settlement could increase the risk that horizontal fissures will develop through the core, detrimental to its impermeability. Criteria for safety against cracking have been elaborated.

The results of these theoretical considerations are applied to some examples of dams, in which the stresses in the core were measured. Settlement observations in the recently completed Peruča dam are presented and discussed.

Sommaire

Les noyaux d'étanchéité en argile des barrages en enrochements ont tendance à tasser plus que les recharges, ce qui provoque des contraintes de cisaillement à la surface de contact. Les équations donnant les contraintes de cisaillement sont tirées de l'hypothèse que l'équilibre plastique règne sur la surface de contact entre le noyau et les recharges. Il en résulte des contraintes réduites dans le noyau et une certaine uniformisation des tassements.

L'interférence avec le tassement libre peut provoquer la formation de fissures horizontales dans le noyau, au détriment de son étanchéité. La communication donne des critères de sécurité contre la formation de fissures.

Les résultats de ces considérations théoriques sont appliqués à quelques barrages, dans lesquels les efforts dans le noyau ont été mesurés. Les observations de tassements dans le barrage de Peruca construit récemment sont discutées.

Notations

- σ normal stress in the core ; suffixes :
  - h = horizontal ;
  - v = vertical ;
  - b = lateral oblique pressure ;
  - r = on boundary surface ;
  - o = on center line of dam ;
- τ shearing stress in the core ;
- $2b$ width of the core on top ;
- φ angle of sheraing resistance ;
- c cohesion ;
- $\varkappa$ ratio of the normal stresses in the active state ;
- λ ratio of the normal stresses in the passive state ;
- i angle of slope of core ;
- γ_w unit weight of water ;
- γ' submerged density ;
- γ bulk density ;
- p external pressure ;
- K_p coefficient of passive earth pressure ;
- K_a coefficient of active earth pressure ;
- z vertical coordinate on axis of core ;
- x, y Descarte's coordinates ;
- μ angle between slip lines.

1. Introduction

Rock fill dams are generally constructed with a wide impervious core or with an inclined narrow clay core on the upstream side. Sometiems a dam with a narrow central impervious core is very appropriate on account of the morphological characteristics of the site and of the availability of suitable material for the impervious core. But this type of dam is not very much in favour because some designers feel that horizontal fissures may develop through the core where the settlement of the core is greater than that of the rock shells. So far the authors have not been able to trace failures of this type of dam.

Settlement observations on rock fill dams show that the filling settles for a long time after completion as may be gathered from much information published in pertinent literature (Davis, 1949). On some rock fill dams, even after more than 40 years of service, the settlement is not completed. Due to this fact, settlement difference between the core and the shells diminish with time. An analysis of the plastic equilibrium conditions of the core will show that different settlement of the core and of the shells reduces the vertical stress in the core and its settlement as well.

In a narrow clay core zones may develop where the active limit equilibrium, the elastic equilibrium and the passive limit equilibrium develop, simultaneously.

The distribution and size of these zones along the height of the core depend on the characteristics of the materials forming the dam, on the dimensions of the core and the rock shells. The following cases may occur :

(a) in a core of very plastic clay — according to some measurements — a zone of active limit equilibrium develops involving about 70 per cent of its height ; the lower 30 per cent of the core remains in elastic equilibrium,

(b) in a core of clay of low plasticity the active pressure of the rock shells may be higher than the active pressure of the core, and a zone of plastic limit equilibrium develops in a lower section of the core ; in such a section the core may be pushed upwards.

An analysis of a simultaneous co-existence of zones of elastic and plastic equilibrium in a core is beyond the scope of this paper. The conditions of active equilibrium and of passive equilibrium shall be analysed only.

2. Stresses in a narrow clay core in active limit equilibrium

The settlement of a core is generally larger than that of the shells and shear stresses appear on the boundary surface. If this difference is large enough, the core will eventually reach a state of active limit equilibrium defined by Mohr's failure criterion which can be represented on Mohr's circle, Fig. 1a. Mohr's criterion can be written as follows :

$$\frac{\sigma_b}{\sigma_{vr}} = \varkappa = \frac{1 - \operatorname{tg} i \operatorname{tg}(\varphi - i) - \dfrac{2c}{\sigma_{vr}}[\operatorname{tg}\varphi - \operatorname{tg} i + \operatorname{tg}\varphi \operatorname{tg} i \operatorname{tg}(\varphi - i)]}{1 + 2\operatorname{tg}^2\varphi - \operatorname{tg} i \operatorname{tg}(\varphi - i) - 2\operatorname{tg}\varphi \operatorname{tg} i - 2\operatorname{tg}^2\varphi \operatorname{tg} i \operatorname{tg}(\varphi - i)} \quad (2{\cdot}1)$$

$$\frac{\sigma_b}{\sigma_{hr}} = \varkappa_1 = \frac{1 - \operatorname{tg} i \operatorname{tg}(\varphi - i) - \dfrac{2c}{\sigma_{hr}} \operatorname{tg} i}{1 + 2\operatorname{tg}\varphi \operatorname{tg} i - \operatorname{tg} i \operatorname{tg}(\varphi - i)} \quad \dots\dots \quad (2{\cdot}2)$$

The contact plane between the core and the shell is an envelope of slip surfaces of the limit equilibrium state, and a surface of discontinuity as well. The slip surfaces are defined on the centre line of the core by the angle $\mu = (\pi/4 - \varphi/2)$ to the vertical (Fig. 1b). On the center line the stresses 6_{vo} an 6_{ho} are principal stresses and their ratio is therefore :

$$\frac{\sigma_{ho}}{\sigma_{vo}} \cong K_a = \operatorname{tg}^2(45 - \varphi/2) \quad (2{\cdot}3)$$

It is possible to obtain in each particular case the average ratio $\frac{\sigma_{ho}}{\sigma_{vo}}$ exactly.

The stress condition of the core in this case is similar to that of a thin plastic layer compressed between two rigid plates. A general solution for this case was given by Prandtl for $\tau = \text{const} = c$, and $2b \ll L$ (Fig. 1c). Prandtl's solution writes :

$$\sigma_x = p + \tau_{\max} y/b = f(p,y) \text{ (Nadai, 1931)} \quad (2{\cdot}4)$$

If the condition $2b \ll L$ is not satisfied then the magnitude of σ_x changes along the width of the layer approximately linearly as defined by the following expression :

$$\sigma_x = \sigma_{xo}(1 - 0{\cdot}3 \cdot x/b), \quad \dots\dots \quad (2{\cdot}5)$$

so that the stress at the boundary becomes $\sigma_{xr} \cong 0{\cdot}7{\cdot}\sigma_{xo}$.

If the rigid surfaces are inclined and the shear strength is proportional to the normal stress the stress pattern will not change (Sokolovski, 1946).

Supposing a parabolical distribution of the vertical stress σ_v long any horizontal section we can write

$$\sigma_{vo} = \alpha{\cdot}\sigma_v \quad \dots\dots \quad (2{\cdot}6)$$

and

$$\alpha = \frac{3\varkappa}{2\varkappa + K_a\varkappa_1} = \frac{3}{2 + K_a\varkappa_1/\varkappa}$$

with σ_v meaning the average vertical stress.

According to (2·4) we can assume that

$$\sigma_{ho} = \sigma_{hr}, \quad (2{\cdot}8)$$

this approximation being on the safe side.

Using the relations (2·2), (2·3). (2·6) and (2·8) we can express σ_b and σ_h by σ_v.

From the equilibrium condition on a differential element, Fig. 1*d*, we get for $\Sigma Z = 0$ and with $\tau = C + \sigma_b \operatorname{tg}\varphi$

$$2\gamma\left[b_{(z)} + \frac{dz}{2}\operatorname{tg} i\right]dz + 2[\sigma_b \operatorname{tg} i - \tau]\,dz - 2d[\sigma_v b_{(z)}] = 0 \quad \dots\dots \quad (2{\cdot}9)$$

and

$$\frac{d\sigma_v}{dz} + \frac{\sigma_v}{b_{(z)}}[\operatorname{tg} i + \varkappa_1 K_a \alpha(\operatorname{tg}\varphi - \operatorname{tg} i)] = \gamma - \frac{c}{b_{(z)}} \quad \dots\dots \quad (2{\cdot}10)$$

If we introduce the approximations $\alpha = \text{const}$ and $\varkappa_1 = \text{const}$ the differential equation (2·10) becomes linear; it may be written as

$$\frac{d\sigma_v}{dz} + \frac{\sigma_v}{b_{(z)}}A = \gamma - \frac{c}{b_{(z)}} \quad (2{\cdot}11)$$

where

$$A = \operatorname{tg} i + \varkappa_1 K_a \alpha(\operatorname{tg}\varphi - \operatorname{tg} i) \quad \dots\dots \quad (2{\cdot}12)$$

The general solution of equation (2·11) writes :

$$\sigma_v = e^{-\int \frac{A}{b(z)}dz}\left[L + \int \gamma e^{\int \frac{A}{b(z)}dz} \cdot dz\right] \quad \dots\dots \quad (2{\cdot}14)$$

where L is a constant determined by the condition

$$\sigma_v = 0 \text{ for } z = 0. \quad (2{\cdot}15)$$

A simple computation gives the solution in its final form :

$$\sigma_v = \frac{\gamma b}{A + \operatorname{tg} i}\left[1 + \frac{z}{b}\operatorname{tg} i - \frac{1}{\left(1 + \frac{z}{b}\operatorname{tg} i\right)^{A/\operatorname{tg} i}}\right] - \frac{c}{A}\left[1 - \frac{1}{\left(1 + \frac{z}{b}\operatorname{tg} i\right)^{A/\operatorname{tg} i}}\right] \quad (2.16)$$

For the core with vertical boundaries the differential equation is

$$\frac{d\sigma_v}{dz} + \sigma_v\frac{A_1}{b} = \gamma - \frac{c}{b} \quad (2{\cdot}17)$$

where

$$A_1 = \alpha K_a \operatorname{tg}\varphi \quad (2{\cdot}18)$$

Its solution is given as :

$$\sigma_v = \frac{b}{A_1}\left(\gamma - \frac{c}{b}\right)\left(1 - e^{-A_1\frac{b}{y}}\right) \quad \dots\dots \quad (2{\cdot}19)$$

It is easy to see that $A_1 = A(i = 0)$.

The constants $\varkappa$ and $\varkappa_1$ can be simplified with sufficient accuracy in the case that $c = 0$ to :

$$\varkappa = \frac{1}{1 + 2\operatorname{tg}^2\varphi}, \quad \varkappa_1 = \frac{1}{1 + 2\operatorname{tg}\varphi \operatorname{tg} i} \quad \dots\dots \quad (2{\cdot}13)$$

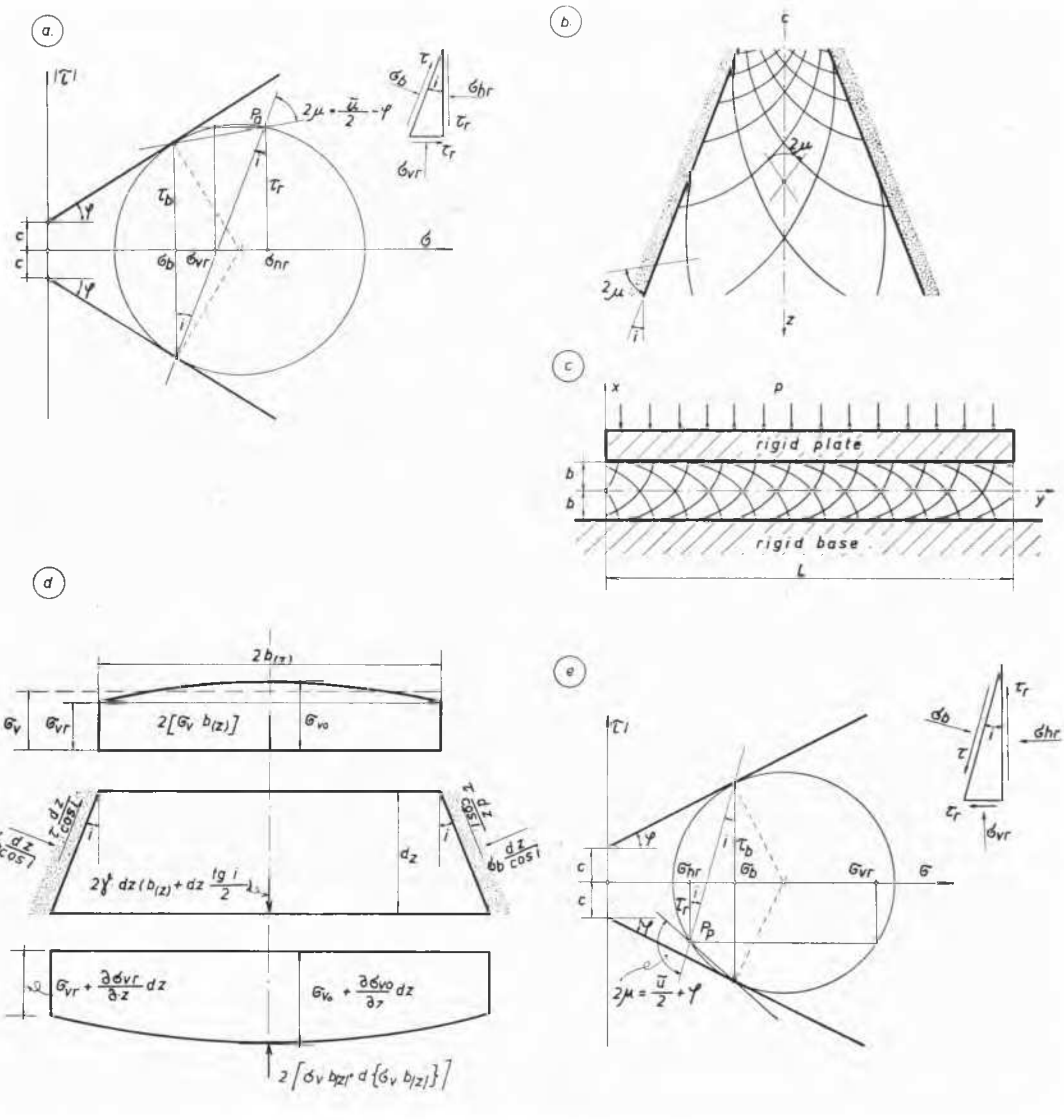

Fig. 1 Equilibrium conditions of plastic cores: (a) stresses in the active state; (b) slip lines in the active state; (c) thin plastic layer between two rigid plates; (d) equilibrium of forces on a differential element; (e) stresses in the passive state.

Conditions d'équilibre des noyaux plastiques : (a) tensions dans l'état actif ; (b) lignes de glissement dans l'état actif ; (c) couche plastique mince entre deux plaques rigides ; (d) équilibre des forces sur un élément différentiel ; (e) tensions dans l'état passif.

The computation of actual stresses from (2·16) can be simplified by reading the constant A from the diagram on Fig. 2*a*.

3. Stresses in the passive limiting state

From the conditions of plastic yield on the contact surface of the core, Fig. 1*e*, we obtain :

$$\frac{\sigma_v}{\sigma_{vr}} = \lambda = \frac{1 + \mathrm{tg}\, i\, \mathrm{tg}\,(\varphi + i) + \dfrac{2c}{\sigma_{vr}}\,\mathrm{tg}\, i}{1 - 2\,\mathrm{tg}\, i\, \mathrm{tg}\, \varphi + \mathrm{tg}\, i\, \mathrm{tg}\,(\varphi + i)} \qquad (3\cdot1)$$

$$\frac{\sigma_b}{\sigma_{hr}} = \lambda_1 = \frac{1 + \mathrm{tg}\, i\, \mathrm{tg}\,(\varphi + i) + \dfrac{2c}{\sigma_{hr}}\,[\mathrm{tg}\,\varphi + \mathrm{tg}\,\varphi\, \mathrm{tg}\, i\, \mathrm{tg}\,(\varphi + i) + \mathrm{tg}\, i]}{1 + 2\mathrm{tg}\,\varphi\,(\mathrm{tg}\, i + \mathrm{tg}\,\varphi) - \mathrm{tg}\, i\, \mathrm{tg}\,(\varphi + i)\,(1 + 2\mathrm{tg}^2\,\varphi)} \qquad (3\cdot2)$$

With the same assumption as before, that $\sigma_{ho} = \sigma_{hr}$, it follows that $\sigma_{hr} \cong \sigma_{vo}{\cdot}K_p$, where

$$K_p = \mathrm{tg}^2\,(45 + \varphi/2),$$

It is possible to obtain in each particular case the exact average ratio $\dfrac{\sigma_{ho}}{\sigma_{vo}}$.

Assuming again a parabolic distribution of 6_{vx}we have

$$\sigma_v = \sigma_b\,\frac{2\lambda + K_p\lambda_1}{3K_p{\cdot}\lambda\,\lambda_1} = \frac{1}{\beta}\,\sigma_b. \qquad (3\cdot3)$$

If $c = 0$ we can take approximately :

$$\lambda = \frac{1 + \mathrm{tg}\, i\, \mathrm{tg}\,(\varphi + i)}{1 - 2\,\mathrm{tg}\, i\, \mathrm{tg}\,\varphi}, \qquad \lambda_1 = \frac{1 + \mathrm{tg}\, i\, \mathrm{tg}\,(\varphi + i)}{1 + 2\,\mathrm{tg}\,\varphi\,(\mathrm{tg}\, i + \mathrm{tg}\,\varphi)}$$

In this case the vertical stress in the core σ_v will be governed by the lateral active pressure of the shells σ_b. For convenience of calculation a diagram of $1/\beta$ values is given in Fig. 2*b*.

The active pressure of the shells on the contact surface σ_b can be obtained using Caquot-Kerissel's tables, or in any other convenient way.

The passive state in the core will develop if the condition

$$\sigma_b\ (\text{Caquot}) \geqq \sigma_b\ (2{\cdot}16)$$

4. Safety against development of fissures

The safety criteria against the appearance of fissures in a thin core can be established in accordance with the reasons for their formation. If the settlement of the core differs from the settlement of the shells fissures are likely to develop. Neutral stresses in the core caused by percolation forces will increase the probability of development of fissures.

A criterion of safety against development of fissures can be obtained from equation (2·16). Fissures can develop if $\sigma_v = 0$, and for this condition we obtain from (2·16) :

$$\frac{\gamma' b}{A + \mathrm{tg}\, i}\left[1 + \frac{z}{b}\,\mathrm{tg}\, i - \frac{1}{\left(1 + \dfrac{z}{b}\,\mathrm{tg}\, i\right)^{A/\mathrm{tg}\, i}}\right] \geqq \frac{c}{A}\left[1 - \frac{1}{\left(1 + \dfrac{z}{b}\,\mathrm{tg}\, i\right)^{A/\mathrm{tg}\, i}}\right] \qquad (4\cdot1)$$

From equation (4·1) we can easily determine the necessary minimum width (2*b*) of the core.

For the core with vertical boundaries we obtain from (2·19) the condition

$$b \geqq \frac{c}{\gamma'} \qquad (4\cdot2)$$

If some other cause is responsible for the appearance of fissures in the core, e.g. unequal settlements of the foundation of the dam, seismic action etc., than the fissure will have the properties of a drainage path. In this case the tendency of the fissure to close will be opposed by the hydrostatic pressure acting on the plane of the fissure. It will be able to close as long as the condition :

$$\sigma_v \geqq h_w \gamma_w, \qquad (4\cdot3)$$

is fulfilled, where σ_v is the average vertical stress computed from (2·16) or (2·19), taking for γ the dry unit weight of the clay. In the passive state the condition of safety is

$$\sigma_v \leqq \gamma' z, \quad \ldots\ldots \qquad (4\cdot4)$$

σ_v taken from (3·3).

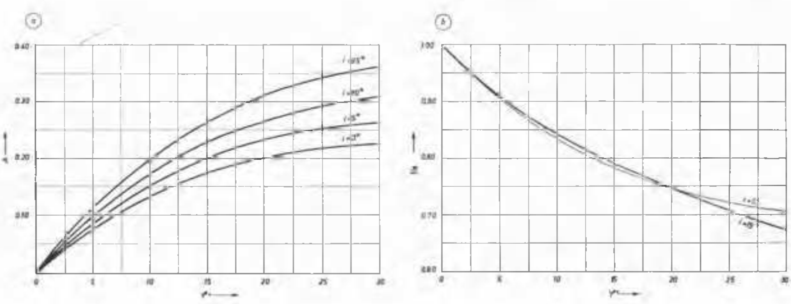

Fig. 2 Coefficients for equations (2,16) and (3,3) : (a) coefficient A, equ. (2,16) ; (b) coefficient $1/\beta$, equ. (3,3).
Coefficients pour les équations (2,16) et (3,3) : (a) coefficient A, équ. (2,16) ; (b) coefficient $1/\beta$, équ. (3,3).

5. Stresses measured in existing dams

In some dams in Sweden stresses were measured in thin clay cores (WESTERBERG, PIRA, HARGRUP, 1951 and LOFQUIST, 1951). On Fig. 3 a comparison is given of the vertical

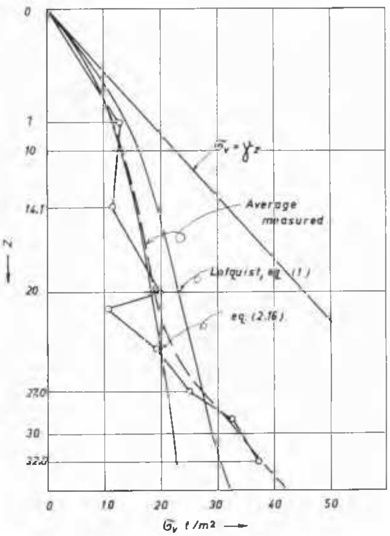

Fig. 3 Stresses in the core of Harspranget dam.
Tensions dans le noyau du barrage Harspranget.

stresses in the core of the Harspranget Dam as they were established by measurement, of the stresses computed by Löfquist's equation and by equation (2·16). It can be seen that the values of equation (2·16) agree fairly well with the smoothened average curve of the actually measured stresses down to a depth of 23 m, i.e. to about 70 per cent of the height of the dam. On the foundation plane the rate of increase of the vertical stress is :

$$\frac{d\sigma_v}{dz} = \gamma.$$

It is evident that the influence of arching cannot reach to foundation level because there the settlement of the core and of the supporting shell are both zero. Some settlement difference is necessary to bring into action the shear strength of the core, which is instrumental for the arching effect.

Similar observations can be made also for the Hölle dam mentioned by LÖFQUIST (1951).

From these examples it is evident that equation (2·16) gives a good approximation of the actual stresses in a thin clay core between supporting shells.

6. Deformations observed on the Peruča dam

The Peruča Dam is a rock fill structure 60 m high with a narrow vertical clay core recently completed in Yugoslavia (Fig. 4). The core was compacted in layers by sheepsfoot rollers, the rock shells were dumped from 6 to 20 m high banks and sluiced with heavy water jets. The clay for the core was highly plastic (*CH*), placed slightly above optimum. The consolidation settlement, as computed from confined compression tests, amounted to about 2·0 m. The expected settlement of the rockshells was about 1 per cent of the height, i.e. some 60 cm. A substantial differential settlement between core and shells would result. A more detailed analysis based on the previously developed equation (2·16) showed that the postconstruction settlement of the core after construction would be much nearer to that of the shells.

In order to get a closer picture of the behaviour of similar rock fill dams, two crossarm settlement gauges of the USBR type were installed in the core. Details of the gauges are shown on Fig. 4d. Each gauge has 25 cross arms.

Some typical settlement curves are shown on Fig. 5 together with the diagram of load increase during construction of the dam. The settlement curves recorded on the cross arms show :

(a) Very heavy settlement during construction (see curve 14, Fig. 5),

(b) Some negative settlement on the lower cross arms (see curve 3, Fig. 5).

On Fig. 6a the recorded settlement of the layer between the crossarms 5 and 10 is represented (curve *c*). On the same diagram the theoretical consolidation settlement curves for full vertical stresses in the core, and for vertical stresses reduced according to eq. (2·16). are shown (curves (*a*) and (*b*) resp.). Similar curves were obtained for all the other layers of the core, and summary curves are shown on Fig. 6b.

From the observed settlement curve (*c*) the compression of air, computed under the assumption that full pore pressure developed during construction, was subtracted and the reduced curve (*d*) was obtained.

A comparison of the theoretical consolidation settlement curves (*a*) and (*b*) with the observed settlement curves (*c*) and (*d*) shows :

(*a*) During construction the settlement and the rate of settlement are much greater than the computed values,

(b) The actual rate of settlement after construction is much lower than the computed value.

The large settlement during construction can be explained as a consequence of adjustments of the lateral pressure on the transition plane core/shell. The initial pressure of the shell on the boundary with the core will probably be near the active pressure because the space left between higher lifts of the rock fill and the lower surface of the core was dumped with only limited sluicing (space *A* on Fig. 4a). The distribution of the active pressure of the shells, computed with $\varphi = 45°$, is shown on Fig. 7. The core, which is compacted in thin layers, exerts initially a lateral pressure which is near to the "at rest" condition. During construction of the dam this pressure is much higher than the active pressure of the core. Thus a lateral movement has to be initiated, resulting in a reduction of the pressure of the core, and in an increase of the resistance of the shells until lateral pressure equilibrium is reached. Since the movement necessary to activate the passive resistance is much higher than that sufficient to reach the active limiting state, fairly large movements may occur in some parts of the core.

At a later stage when the vertical settlement of the core induces shear stresses on the boundary plane with the shells, the vertical and the lateral pressure is reduced in agreement with eq. (2·16). Even this reduced lateral pressure is higher than the active pressure of the shells.

One piezometer point was placed in the core at El. 345 and it revealed that the pore pressure increased very slightly during construction ; it consolidated quickly to zero. So there is no reason for hydrodynamic consolidation of the clay, nor for substantial compression of air in the voids. From these observations it seems that the settlement of the core was not governed primarily by hydrodynamic consolidation of the clay.

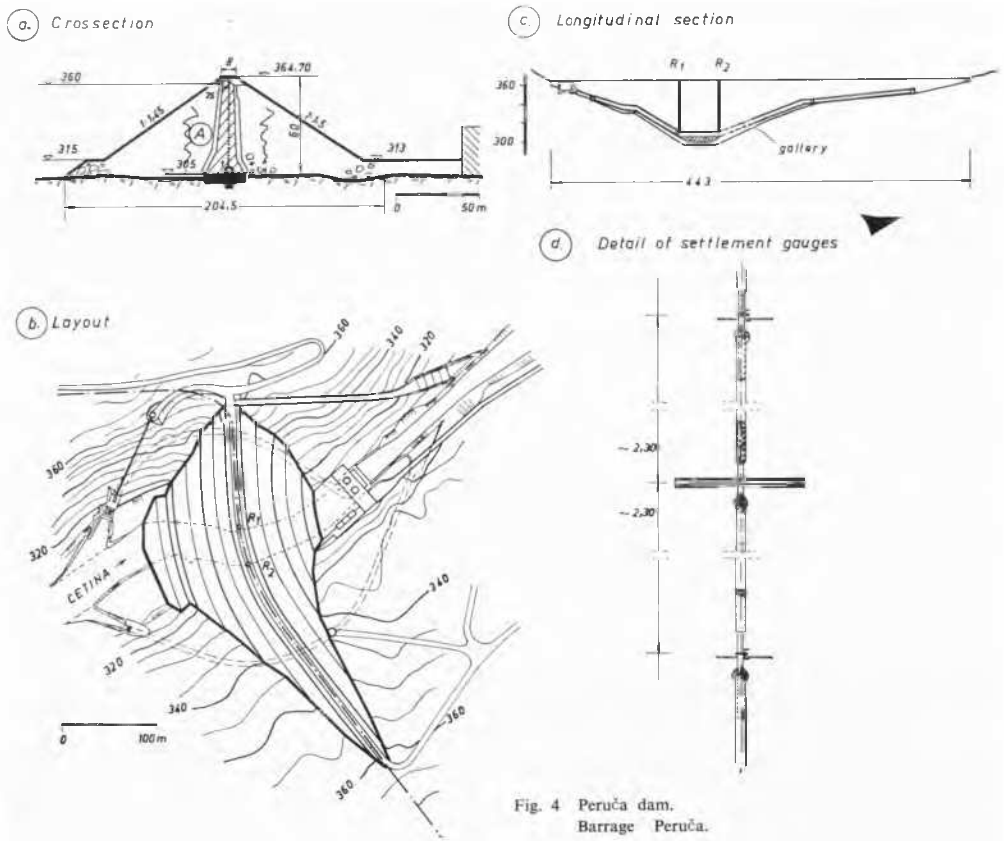

Fig. 4 Peruča dam.
Barrage Peruča.

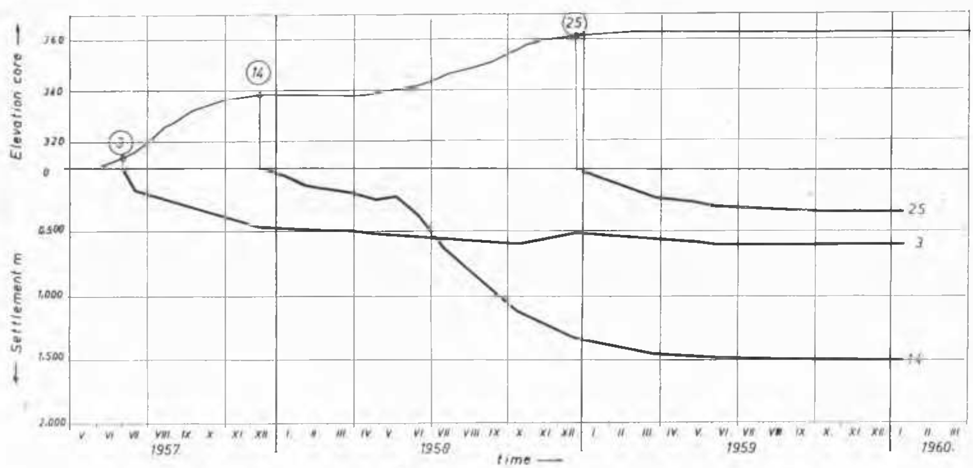

Fig. 5 Typical settlement curves of cross arms, Peruča dam ; (3), (14), (25), number of cross arm.
Courbes typiques provenant du tassement des cross arms, barrage Peruča ; (3), (14), (25) numéro du « cross arm ».

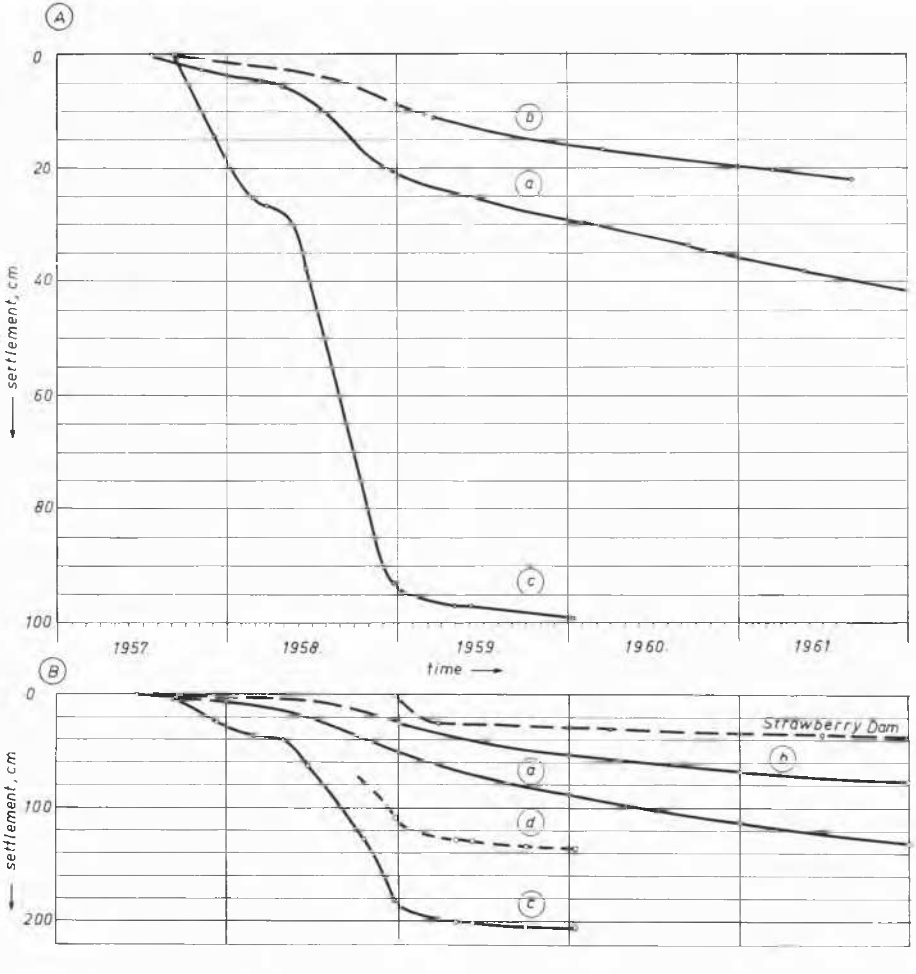

Fig. 6 Typical settlement curves Peruča dam : (A) layer between cross arms 5 and 10 ; (B) layer between cross arms 2 and 25.
Courbes de tassement par le temps, barrage Peruča : (A) couche entre les « cross arms » 5 et 10 ; (B) couche entre les « cross arms » 2 et 25.

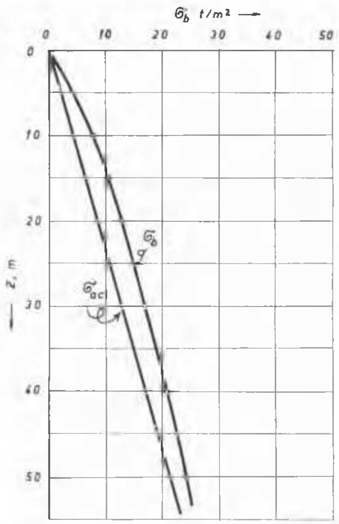

Fig. 7 Computed lateral pressures on boundaries of core, Peruča dam : σ_{act}, active pressure of shells ; σ_b, lateral pressure of core according to equ. (2,16).

Pression latérale sur le noyau, barrage Peruča : σ_{act}, pression active de la recharge en enrochements ; σ_b, pression latérale du noyau d'après l'équ. (2,16).

The authors compare the post construction settlement curve of a typical rockfill dam with the settlement of the core at Peruca. The settlement curve as recorded for the Strawberry Dam, a 150 ft high rock fill dam with upstream reinforced concrete diaphragm, is shown on Fig. 6b. The post construction settlement curve for the core of the Peruca Dam from 1959 is nearly parallel to the settlement curve of the Strawberry Dam. It appears in this case that the post-construction settlement of the core is probably tied to the settlement of the rock fill shells and is not a consequence of hydrodynamic consolidation of the clay.

The analysis of stresses in the two Swedish dams shows good agreement between measurement and computation. At Harspranget, the rock fill up to 10 m from the clay fill was well compacted, at Hölle the supporting shells consisted of compacted sand and gravel. In both dams the supporting shells must have exerted from the very beginning a lateral pressure on the core much higher than the active pressure. The comparison of the observed deformations of the Peruca Dam with deformations resulting from theoretical computation disclose differing amount and development of settlements. This can be explained only by the lower compaction of the rock in the zones adjacent to the core.

7. Conclusion

The results of this theoretical study show that a thin plastic core in a rockfill dam cannot settle independently from the rockfill.

Some danger of cracking due to arching exists, in practice, however, the risk is slight and with a well constructed filter fissures would be self-healing.

The analytical approach to this problem shows that the vertical stresses in the core must be considerably reduced due to potential settlement differences between the core and the shells.

The analysis of the observed settlement curves of the Peruca Dam does not show good agreement with the theoretical ones computed from consolidation tests proving that some other influences prevail.

The results of deformation and settlement measurements on the Peruca dam show that plastic deformations occur in the central part of the dam during the construction period. These deformations could be reduced by compacting the rockfill zones adjacent to the core. With a sufficiently plastic clay core however this plastic deformation does not endanger its scope.

The observed post construction settlement of the core is much less than it would be from hydrodynamic consolidations of the clay. It is in fact similar to the long term settlement of plain rockfill.

Fissures may develop in narrow cores under special conditions, which are given on the basis of the theoretical analysis.

Aknowledgement

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