Research article

Assessment of crack resistance of ultra-high earth core rockfill dam by pore pressure

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Abstract. An important problem at construction of high earth core rockfill dams (ECRD) is danger of hydraulic fracturing of their seepage control element. It is usually considered that the cause of hydraulic fracturing is presence of micro cracks in the core, which open due to deficit of compressive stresses at penetration of reservoir water into it. However, this theory ignores the presence of the force in the soil core which fractures it from the inside, i.e. the pore pressure force. The author proposes the criterion of core crack resistance based on consideration of pore pressure. With the aid of numerical modeling of stress-strain state, the impact of pore pressure on potential development of cracks in the core of a 330 m high ECRD was studied. The considered dam has an inclined sandy loam core. By the results of analysis, it was established that the value and development of pore pressure are greatly influenced by the degree of water saturation S_0 of clayey soil at its placement into the dam body. At $S_0 = 0.9$ pore pressure in the core exceeds 4 MPa, and the zone of high pore pressure covers the most part of the core. At that, stresses in the soil skeleton (effective stresses) are still compressive stresses, which formally evidences about the core integrity. However, at $S_0 = 0.9$ the zone of shear strength loss is formed in the core. It may be expected that failure of the core integrity during shear will result in hydraulic fracturing. Thus, pore pressure induces hydraulic fracturing. To prevent the core hydraulic fracturing, it is necessary to reduce pore pressure. Therefore, the clayey soil should be placed with moisture content by 15 % less than the optimal value.

1. Introduction

Earth core rockfill dam (ECRD) may be of considerable height. The highest dam of this type is 300 m high Nurek dam in Tajikistan, which was built in the USSR in 1980. This dam is second highest in the world. Eleven more ECRDs are more than 200 m high. Four of them were built in the XXIth century: Tehri (H = 260.5 m, 2006), Nuozhadu (H = 261.5 m, China, 2012) [1-3], Changheba (H = 240 m, China, 2017) [1], San Roque (H = 210 m, Philippines, 2003). Besides, in the XXIth century a number of high dams was built: Karkheh (H = 127 m, Iran, 2001) [4-6], Masjed-e-Soleyman (H = 177 m, Iran, 2002) [7], Qiaoqi (H = 125,5 m, China, 2006) [8], Pubugou (H = 186 m, China, 2010) [9], Maoergai (H = 147 m, China, 2011) [8], Upper Gotvand Dam (H = 180 m, Iran, 2012).

Construction of high ECRDs has also further perspectives. In Tajikistan the dam of Rogun HPP with overall height 335 m is constructed and it should be the highest in the world. In China there considered construction alternatives of two ultra-high ECRD: Lianghekou (H = 295 m) [10], Shuangjiangkou (H = 314 m) [10,11].

These facts evidence about actuality in developing the theory of ECRD designing.

One of the important issues in ECRD designing is assessment of crack resistance of the seepage control core. In [12] there is information about failures of earthfill dams where loss of integrity occurred. There known the cases of core integrity loss at dams Balderhead (1967, United Kingdom, H = 48 m) [13], Hyttejuvet (1966, Norway, H = 93 m) [14]. There is information about formation of cracks in the core of El Infiernillo / Adolfo López Mateos dam (H = 149 m, Mexico) in 1969. Loss of core integrity is also proved by holes revealed in 1996 on the crest of Bennett ECRD (Canada, H = 183 m).

The problem of providing crack resistance of clayey cores is also urgent for Russian dams. In 1992 the core of Kureika dam project lost its integrity at the section where the dam height is 25 m [15, 16]. The data of geophysical investigations evidences that the core of the highest in Russia Kolyma dam (H = 134.5 m, 1988) is in a diluted state. Settlements of its crest exceeded 2.5 m.

Most sharp the issue of core crack resistance of rock-earthfill dams became after failure of 90 m high Teton dam (USA) in June 1976 [17]. From this moment the studies commenced on the processes of crack formation and development in ECRD cores. Large contribution in development of crack formation theory in ECRD was made by J.L. Sherard, B. Kjoernsli, G.W. Jaworski, H.B. Seed, S. Ngambi and others.

At present, theoretical studies of conditions leading to crack formation in ECRD cores are carried out as well as experimental tests of clayey soils resistance to rupture [8,18]. Review of development of methods for assessing crack resistance of clayey cores is given in publications of S. Poudel et al. [19], M. Salari et al. [12], D.Q. Tran et al. [20]. The fullest and thorough review is given in [20].

At present the failures of dams Balderhead, Hyttejuvet and others are explained purely by hydraulic fracturing phenomenon. This notion was introduced by B. Kjoernsli. Hydraulic fracturing is connected with arch effect, when deficit of compressive stresses occurs in the core due to redistribution of stresses between the core and shells as well as with development in the core of tensile stresses due to non-uniform deformations of the dam. It is considered that in soil there are always micro cracks which open due to deficit of compressive stresses at penetration of upstream water into them. High hydrostatic pressure ruptures soil from inside, therefore, through cracks are developed in the core and this process is called hydraulic fracturing.

In [12] there given the proposed criteria of assessing potential hydraulic fracturing and their comparison. Classification of prediction methods of hydraulic fracturing is proposed by J. Wang [21]. They are divided into three groups. The first group of methods are analytical methods based on analysis of material stress state at presence of cracks in it. The second group of methods are empirical methods based on the results of experiments with soil samples. The third group of methods is based on experimental models and the theory of material fracture mechanics.

The condition for occurrence of hydraulic fracturing is assumed to be as follows:

$$p_f < \gamma_w h_w,$$

p_f is critical pressure at which hydraulic fracturing occurs; γ_w is unit weight of water; h_w is depth (head) of water.

Value p_f is determined by soil stress state as well as its tensile strength. It was established that hydraulic fracturing appears not only as a result of tensile stresses, but also due to shear stresses, as well as due to joint action of tensile and shear stresses.

Different authors proposed several alternatives of pressure criterial values $\left(p_f \right)$. In Russia it is often assumed that direction of cracks is sub-horizontal (due to methodology of layered soil placement), accordingly, p_f is equal to the sum of vertical normal stress σ_y and tensile strength (cohesion) of soil σ_t. However, such an approach leads to overestimation of core crack resistance, because in other, not horizontal directions, soil is subject to considerably less compressive stresses.

As clear from the book [21; 7], most of the authors (A. Mori, M. Tamura [22]; A.K. Panah, E. Yanagisawa [23]; A. Ghanbari, S.S. Rad [24]) connect critical pressure p_f with total minor (minimum) principle stress $\left(\sigma_3 \right)$ in soil. Some authors express p_f through horizontal stress σ_x, which is close to σ_3. The authors of publication [7] on the example of Bidvaz dam (H = 60 m, Iran) showed with the aid of numerical modeling that the core minimal margin of crack resistance is observed at calculation by Ghanbari

and Rad criterion, where critical pressure is expressed through σ_3, and tensile strength (σ_t) of soil is not taken into account.

However, the conventional theory of hydraulic fracturing has not been sufficiently substantiated theoretically. The following disadvantages may be marked:

1. For hydraulic fracturing by hydrostatic pressure it is necessary that the upstream water should penetrate into the pores or micro cracks of soil. However, as a rule, the core water permeability is sufficiently small for establishment of seepage regime. In [25] with the aid of numeric modeling it was obtained that for hydraulic fracturing along the horizontal crack the initial weakness zone is required. Therefore, failures of ECRD occurred during the initial reservoir impoundment could be explained only due to defects of the core structure.

2. At assessment of crack resistance, it is assumed that along the whole core thickness the pressure in pores will correspond to hydrostatic pressure $(\gamma_w h_w)$, though during seepage the head drop along the core thickness occurs.

3. The theory of hydraulic fracturing ignores the real force which may cause fracturing. This is pore pressure force.

Pore water pressure (PWP) is surplus (as compared to atmospheric) pressure in water and air filling soil pores. It is caused not only by hydrostatic pressure of water filtering in pores but it also appears at soil compaction (at decrease of pore size) under action of external loads. In 1999 in paper [14] A.K.L. Ng and J.C. Small with the aid of numeric modeling showed that pore pressure reduced effective stresses in the core of Hyttejuvet dam actually to 0, which could be the cause of hydraulic fracturing of this dam core.

Starting from the middle of the XXth century the pore pressure phenomenon in ECRD cores was studied by conducting field measurements.

In [26, 27] there given the data of field measurements of pore pressure at a number of ECRDs, built in the middle of the XXth century. These dams are: Nurek (Tajikistan, 1980), Pachkamar (H = 71 m, Uzbekistan, 1967), Charvak (H = 168 m, Uzbekistan, 1970), Aswan (H = 111 m, Egypt, 1970), Talbingo (H = 162 m, Austria, 1970). Their analysis shows that the processes of formation and dissipation of PWP in dams progress in different ways. At some dams (Aswan, Pachkamar) pore pressure in ECRD core did not exceed hydrostatic pressure, but the processes of soil consolidation developed quickly. At other dams (Talbingo, Charvak) pore pressure reached considerable values and dissipated slowly.

In ECRDs, constructed in the XXth century, measurements of pore pressure were also carried out. In the publications we can find the information about field measurements of pore pressure at the dams constructed in the XXIst century in Iran and China.

After construction of Karkhe dam (Iran) measurements of PWP were carried out [4-6]. It reached 1.0 MPa, i.e. comprising approximately half of total vertical stresses in the core. Consolidation of the core soil (dispersion of pore pressure) lasted about 5 years. In the inclined core of Bidvaz dam (H = 60 m, Iran) PWP exceeded 0.5 MPa [12]. In the core of 177 m high Masjed-e-Soleyman dam (Iran) PWP reached nearly 2.3 MPa [7].

In China measurements of pore pressure were conducted in the core of Nuozhadu dam (China, H = 261 m) [2,3]. This dam core is made of compact soil: gravel-clay soil, therefore, PWP is comparable with hydrostatic pressure [3].

Thus, in certain circumstances, pore pressure in ECRD cores may reach considerable values and become the cause of failure and loss of integrity of their cores. Therefore, the author proposes to assess core crack resistance by the value of pore water pressure. The following criterion of crack formation may be proposed:

$$\sigma_3 - p_w \geq \sigma_t,$$

where σ_3 is total minor (minimum) principle stress (positive stresses correspond to compression); p_w is pore water pressure; σ_t is tensile strength.

Difference $(\sigma_3 - p_w)$ is minimum effective stress in soil $\sigma_{3,ef}$, i.e. minor (minimum) principal stress is soil solid phase.

For assessment of core crack resistance marginal value, the factor of core crack resistance is calculated based on the proposed criterion:

$$k = \frac{\sigma_3 + \sigma_t}{p_w} = 1 + \frac{\sigma_{3,ef} + \sigma_t}{p_w}.$$

With consideration of micro cracks in soil there may be considered that hydraulic fracturing may occur even at $k < 1.1 \div 1.2$.

Pore pressure should be determined by calculation for using the proposed approach of assessment of core crack resistance.

Methods of analyses of pore pressure in soils are based on works of K. Terzaghi (USA), H.M. Gersevanov (Russia), others. V.A. Florin (Russia) and M.E. Biot (Netherlands) created theories of soil consolidation. In them soil is considered as a three-phase medium, which consists of solid particles, pore water and air; soil deformation is considered as a dynamic process of these phases' interaction.

In Russia A.A. Nichiporovich and T.I. Tsybulnik developed an analytical method of calculating the value of pore pressure and its time dependent measurement for ECRD cores. This method of analysis is based on seepage consolidation theory developed by Florin and is an individual case of Biot theory.

For determination of pore pressure appearing at soil compaction under action of external forces (mostly of soil dead weight) on the core axis $(x = 0)$, the following formula is used:

$$P_g (x = 0, y) = \frac{4}{\pi \mu} v_\sigma \cdot f(t).$$

In this formula $P_g (x = 0, y)$ is pore pressure from overlying soil weight at height y along the core axis (i.e. with $x = 0$); v_σ is rate of load rise in the process of construction; μ is coefficient related to coefficient of consolidation; $f(t)$ is dispersion function (time dependent decrease) of pore pressure at height y.

Averaged by time rate of load rise v_σ is expressed through pressure σ at height y, duration of construction t_{EOC} and time t_y of core construction to height y:

$$v_\sigma = \frac{\sigma}{t_{EOC} - t_y},$$

σ is average total stress in soil medium in the moment of construction completion from the weight of overlying soil at height y.

Coefficient μ is expressed through characteristics of physical and mechanical properties of soil: seepage coefficient k_f, coefficient of porosity ε and coefficient of compression m, as well as width B of the core at height y:

$$\mu = \frac{\pi^2 (1+\varepsilon) k_f}{4 B^2 \gamma_w m}.$$

Values of k_f, ε and m are assumed to be constant at calculation. For consideration of their variation in the process of external loads' perception, these values are assumed to be averaged for the interval of stress variation from 0 to σ. Compression coefficient m [MPa^{-1}] expresses the material deformation:

$$m = \frac{\Delta \varepsilon}{\Delta \sigma}.$$

The function of time dependent pore pressure variation is calculated from formula:

$$f(t) = \sum_{i=1,3,5}^{\infty} \frac{1}{i^3} \left[e^{-i^2 \mu \alpha (t - t_{EOC})} - e^{-i^2 \mu \alpha (t - t_y)} \right],$$

where i is odd even figures; α is pore pressure coefficient expressing maximum pore pressure in fractions from total (full) average stress in soil medium; t is current moment of time (from the moment of construction commencement); t_y is time passed from the start of construction to construction of the core to height y, t_{EOC} is duration of construction.

The formula is valid only for the operation period, i.e. at $t \geq t_{EOC}$.

This analytical method permits not only predicting the value of pore pressure and time dependent progress of soil consolidation processes. Namely in paper [28] this method was used for prediction of pore pressure in the core of constructed Rogun dam.

However, due to the adopted assumptions the Nichiporovich and Tsybulnik method is an approximation method. For predicting PWP it is required to determine the values of average stresses σ in the core. As σ is formed not only under the action of soil dead weight, but also hydrostatic pressure on the core, its value may be analytically approximate. This decreases accuracy of PWP analyses. More precise load on the core may be accomplished with the aid of numerical modeling of the dam stress-strain state (SSS).

Development of SSS numerical modeling methods permitted obtaining more strict solutions on determination of pore pressure. With the aid of numerical modeling the analyses of pore pressure were conducted already in 1970-s at designing Mika dam (H = 243 m, USA, 1973) [29].

At present with the aid of numerical modeling, SSS analyses are conducted with consideration of PWP. Such analyses on the base of seepage consolidation theory were conducted with the aid of numerical modeling for Nuozhadu dam in China [3].

SSS analyses by the consolidation theory proposed by Biot, permits application of software package FLAC 3D. With its aid, SSS was numerically modeled with consideration of PWP for several dams in Iran: Bidvaz dam [12], Masjed-e-Soleyman dam [7], Siah Sang dam [12].

However, in these investigations the assessment of crack resistance with consideration of pore pressure was not fulfilled. Therefore, the aim of our study is assessment of high ECRD core crack resistance with consideration of pore pressure.

2. Materials and methods

The studies were conducted on the example of an ultra-high ECRD 330 m high. The constructed Rogun dam in Tajikistan is of such a height.

The considered design of the dam is a schematized structure of the dam of Rogun HPP. The dam has a flattened profile, the upstream slope is 1:2 ÷ 1:2.4, the downstream slope is 1:2 (Fig. 1). The dam lateral zones are made of gravel-pebble soil, but are weightened with rockfill. The dam core is constructed with inclination towards the upstream side, the angle of inclination with respect to vertical is 17°. The core is made of sandy loam. The core width at the bottom comprises 139.5 m, i.e. 42.3 % of the dam height.
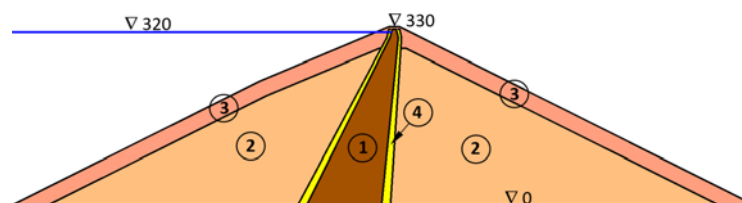

Figure 1. Structural diagram of ultra-high earth core rockfill dam
1 – sandy loam of core; 2 – gravel zone; 3 – rockfill zone; 4 – filter zone.

Analyses of the dam SSS were conducted in 2D formulation. The process of the dam SSS formation was modeled with consideration of sequence of its construction and reservoir impoundment. Firstly, the process of soil placement by layeres was modeled, then gradual impoundment of the reservoir.

Deformation properties of the core sandy loam was assumed based on the results of experimental data on tests conducted in a stabilometer. In [27] there given a compression curve of sandy loam in the range of compressive stresses up to 3.2 MPa. Relationship between stresses and deformations are close to a linear one, therefore, modulus of linear deformation may be assumed to be constant. It is equal to 47 MPa. It was assumed that this compression curve corresponds to drained soil.

During analyses there was applied the model of soil deformation non-linearity which takes into account the following non-linear effects:

- presence in soils of two branches of loading (active loading and de-stressing),
- possible loss of shear strength or tensile strength,
- decrease of deformation at increase of lateral compression.

For consideration of soil deformation decrease at increase of lateral compression there was used dependence between soil solid skeleton modulus of linear deformation and compression stresses:

$$E = E_1 \left(\sigma_{3,ef} \right)^k ,$$

where E_1 is modulus of linear deformation at compression stress 0.01 MPa; k is power index; $\sigma_{3,ef}$ is total minor (minimum) principal stress, characterizing the minimum level of lateral compression.

Poisson's ratio of soils in a strong state is considered to be constant. The parameters of the model are given in Table 1.

Mohr-Coulomb strength condition was used for determination of soil shear strength. At determining shear strength of coarse soils consideration was taken of decrease of internal friction angle value φ with increase of compressive stresses σ. Relationship between φ and σ was assumed to be linear at the section from φ_0 (at $\sigma = 0$) to φ_k (at σ = 1.5÷2.0 MPa).

Table 1. Parameters of physical-mechanical model of materials

Material	Density [g/cm³]		Deformation parameters			Strength indices		
	ρ	ρ_{sat}	E_1 [MPa]	k	v	φ_0	φ_k	c [kPa]
sandy loam of core	2.25–2.28	2.30	42.3	0.0	0.35	29°	29°	10
gravel	2.20	2.38	43.2	0.35	0.2	49°	38°	0
rockfill	2.00	2.25	43.2	0.35	0.2	50°	34°	0

Designations:

ρ, ρ_{sat} is soil density respectively at placement (with consideration of moisture content) and in a saturated state; v is Poisson's ratio; φ_0, φ_k is initial and final angle of internal friction respectively; c is specific cohesion.

Pore pressure is considered by introduction of an additional parameter, i.e. modulus of volumetric deformation $E_{0,w}$ of pore liquid. $E_{0,w}$ expresses relationship between pore pressure and volumetric deformations of soil. Expression for determination of $E_{0,w}$ was obtained from R. Boyle – E. Mariotte law:

$$E_{0,w} = \frac{p_a \left(1 + \varepsilon_0 \right)}{\varepsilon_{0,air} + \beta \varepsilon_w - \Delta \varepsilon} .$$

In this formula p_a is atmospheric pressure; $\varepsilon_{0,air}$ is coefficient of soil porosity corresponding to pores filled with air, before start of compaction process; ε_w is coefficient of soil porosity corresponding to pores filled with water; β is coefficient of air solubility in water which may be assumed to be equal to 0.0245; $\Delta \varepsilon$ is variation of soil porosity coefficient at compaction.

When all air is squeezed from pores, the denominator of the formula will be equal to 0 and all the load will be transferred to water. In this case $E_{0,w}$ is equal to modulus of volumetric deformation of water (2000 MPa).

Dispersion of pore pressure was not taken into account in calculations.

Calculations were conducted for three alternatives of the value of initial degree of water saturation S_0 of core soil. S_0 is ratio between soil moisture content at placement and moisture content in a saturated

state. In alternative 1 S_0 = 0,80; alternative 2 S_0 = 0,85; alternative 3 S_0 = 0.90. Consequently, the core soil density varied from 2.25 to 2.28 g/cm³.

3. Results and Discussion

Analysis of the dam SSS was carried out for the moment of construction period completion when the reservoir had been impounded to ▽ 320 m (Fig. 2–5).

Fig. 2 shows distribution of pore water pressure (PWP) in the core. Maximum by value PWP is typical for the core lower part, because this zone is subject to maximum loads. Analyses revealed that PWP is greatly dependent on initial degree of soil water saturation S_0. This is explained by the fact that at high degree of water saturation the volume of air in pores is small and at loading the soil is transferred more quickly from a three-phase state (solid particles, water, air) to a two-phase state (solid particles and water). In a two-phase state of soil all load is transferred only to pore water, which will lead to intensive growth of PWP. The more is S_0, the more by value is the zone of high PWP and the higher is the maximum value of PWP. In alternative 1 the maximum value of PWP reaches 2.6 MPa (Fig. 2a), in alternative 2–3.4 MPa (Fig. 2b), and in alternative 3–4.3 MPa (Fig. 2c).

Obtained in alternatives 2 and 3 maximum values of PWP exceed the value of upstream hydrostatic pressure. In alternative 3 PWP exceeds hydrostatic pressure by 30 %.

For checking the obtained results there was calculated the maximum value of pore pressure with the aid of analytical method of Nichiporovich and Tsybulnik. It is reached at the core foot and axis at the moment of construction completion $\left(t = t_{EOC}\right)$. In this calculation the coefficient of the core soil compression m was assumed equal m = 0.029 MPa⁻¹, and seepage coefficient k_f = 1·10⁻⁸ m/s. Pressure from external loads was assumed equal σ = 4.6 MPa and coefficient of pore pressure α = 0.75. Construction duration was proposed to last 5 years. Similar calculation of PWP was fulfilled in [28].

By the results of analysis by the method of Nichiporovich and Tsybulnik the maximum value of PWP comprised 3.5 MPa. This value is in the interval of PWP values obtained with the aid of numerical modeling, which evidences about agreement of both methods.

However, it should be noted that there is a difference in PWP distribution width-wise and height-wise the core. In the method of Nichiporovich and Tsybulnik pore pressure considerably decreases in the core lateral zones and nearly uniformly decreases height-wise. On the contrary, by the results of numerical modeling, PWP in the core upper part is considerably less than in the lower part and actually does not change width-wise the core.
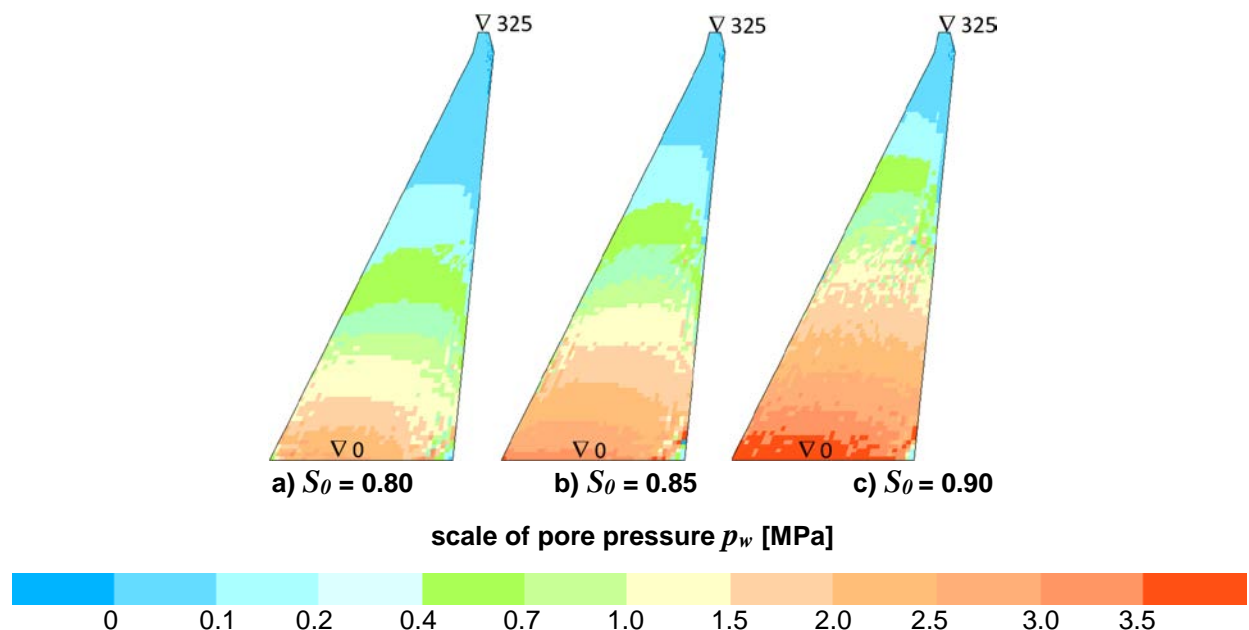

a) S_0 **= 0.80**   **b)** S_0 **= 0.85**   **c)** S_0 **= 0.90**

scale of pore pressure p_w **[MPa]**

Figure 2. Value of pore water pressure in the core for different alternatives of initial water saturation degree S_0**.**

High values of water pore pressure, obtained as a result of analysis, evidence about the fact that it threatens the core strength and crack resistance. For assessment of PWP effect on the core strength there were calculated the values of the core crack resistance coefficient k by the criterion suggested by the author. By the results of analysis, the core lower part where pore pressure is maximum has the lowest crack resistance safety factor (Fig. 2). Analysis shows that the core crack resistance is greatly dependent on coefficient of initial water saturation S_0. In alternative 1 (Fig. 3a) the core crack resistance is provided by safety factor $k > 1,5$, and in alternative 3 (Fig. 3c) in the core lower part the crack resistance coefficient is lower than 1.2.

Minimum value of k exceeds 1, which from the formal point of view, testifies about provision of the core crack resistance, however, comprehensive analysis of SSS showed that it is not true. The thing is that at high pore pressure there is a decrease of effective stresses (stresses in solid phase) in soil and in its shear strength. By the results of calculations in alternative 3 in the core lower half the effective stresses σ_y comprise approximately 2.2÷2.3 MPa (Fig. 4b), which is less than PWP: the main portion of load is perceived by pore water.
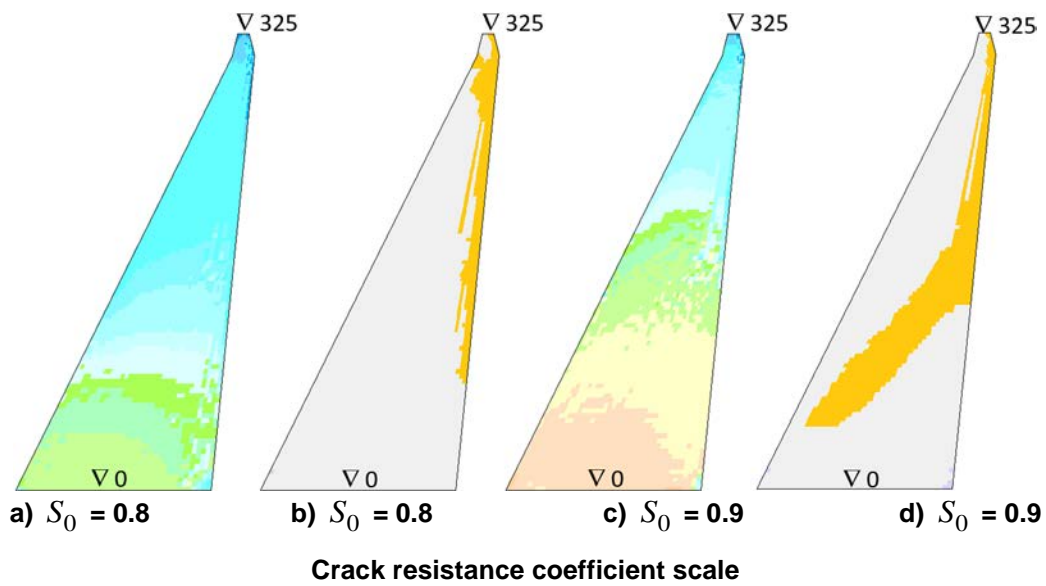

a) $S_0 = 0.8$ b) $S_0 = 0.8$ c) $S_0 = 0.9$ d) $S_0 = 0.9$

Crack resistance coefficient scale

| 1.0 | 1.1 | 1.2 | 1.3 | 1.5 | 2.0 | 2.5 | 3 | 4 | 5 | 8 | 10 | 20 | 40 |

Figure 3. Coefficient of core crack resistance and location of zones with failure of shear strength.

a) σ_y ($S_0 = 0.8$)   b) σ_y ($S_0 = 0.9$)   c) σ_x ($S_0 = 0.8$)   d) σ_y ($S_0 = 0.9$)

stress scale [MPa]

| −4.0 | −3.6 | −3.2 | −2.8 | −2.4 | −2.0 | −1.6 | −1.2 | −0.8 | −0.4 | 0 | 0.4 | 0.8 | 1.2 | 1.6 | 2.0 |

Figure 4. Effective stresses σ_y and σ_x in the core.

Effective stresses σ_x in alternative 3 are even less: 0.7÷0.8 MPa (Fig. 4d). This creates the danger of soil shear strength loss. By the results of analyses in alternative 3 in the core lower part a zone of shear strength loss is formed (Fig. 3d). It is located at an angle of 47° to horizontal. SSS calculation for the case where S_0 = 0.92 showed that the zone of shear failure increases. The effect of forming inclined shear zones in the core at high pore pressure was also obtained by numerical modeling in [7].

As shear failures are accompanied by development of cracks in soil, there arises the danger of the core hydraulic fracturing. Thus, pore water pressure is a potential cause of ECRD failures. Based on the obtained result there may be recommended the corrected criterion of the core crack resistance: K > 1.3.

Comparison of the results of analysis showed that development of pore pressure in the core greatly affects SSS of not only core, but of the whole dam as well. Coefficient of initial water saturation S_0 conditions shears U_x and U_y of the dam settlement.

In alternative 3 due to high PWP the core has great vertical stiffness. Due to this, decrease of the core settlements and the upstream shell occurs. While in alternative 1 maximum construction settlements of the dam reach 490 cm (Fig. 5a), in alternative 3 they comprised only 303 cm (Fig. 5b).
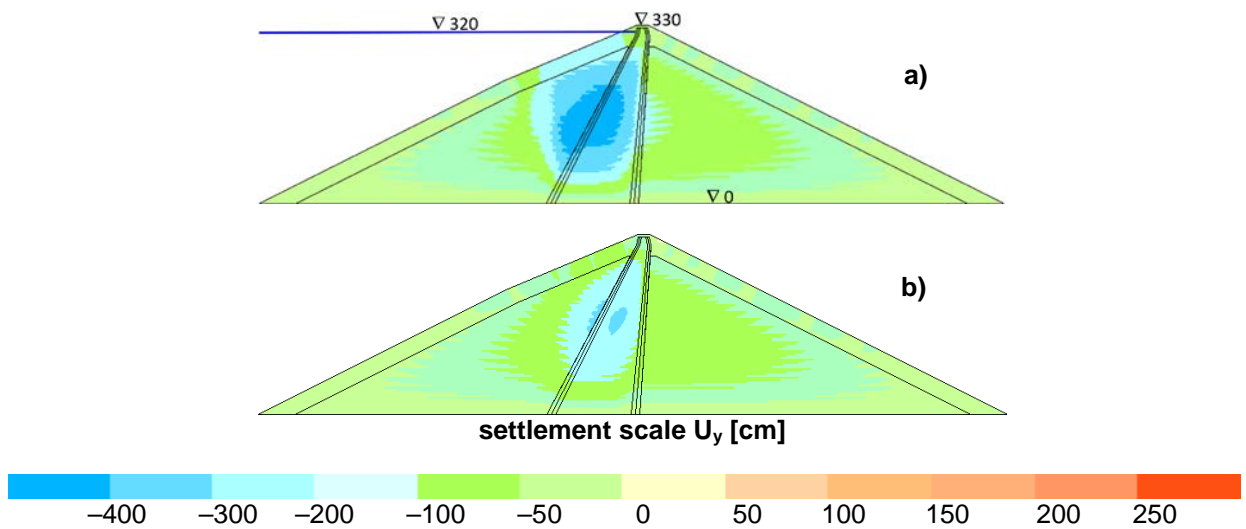

settlement scale U_y [cm]

Figure 5. Dam settlement.

However, high PWP in the core of alternative 3 contributes to growth of horizontal displacements U_x in the upper part of the dam. In alternative 1 maximum horizontal displacement is observed in the core lower part and comprises 131 cm (Fig. 6a), and the crest displacement amounts to 43 cm. In alternative 3 maximum horizontal displacement (126 cm) is observed in the upper part of the dam, at that, the crest displacement reaches 74 cm (Fig. 6b).
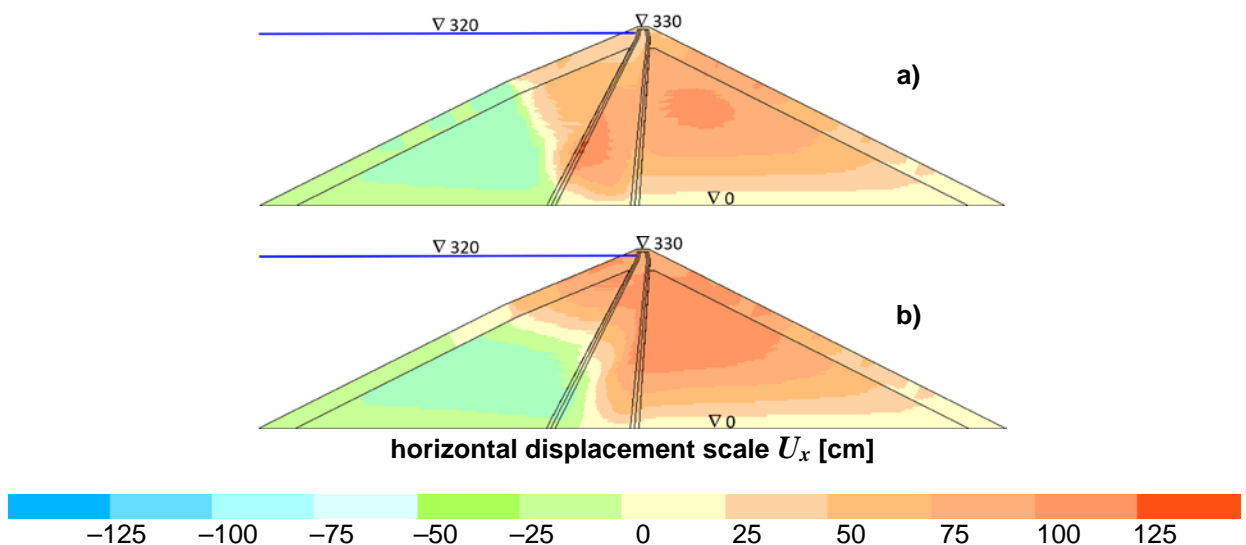

horizontal displacement scale U_x [cm]

Figure 6. Dam horizontal displacements.

4. Conclusion

1. Pore water pressure in the dam core plays an important role in formation of stress-strain state of not only core but of the whole dam. At numerical modeling of stress strain state of high ECRD it is necessary to take into account pore water pressure in the core. The core soil should be considered as a system consisting of several phases (solid, liquid and gas).

2. Pore water pressure is one of the main causes of earth core hydraulic fracturing. First of all, pore water pressure creates a force, wedging pores of soil, and secondly, it contributes to shear deformations. Micro cracks formed by pore water pressure initiate the core hydraulic fracturing.

3. Assessment of crack resistance of dam cores should be carried out with consideration of pore water pressure. The author proposed a criterion of assessment of the dam core hydraulic fracturing by pore water pressure.

4. The important role in formation of pore water pressure in the core of an ultra-high dam is played by soil moisture content at its placement into the dam body, to be more precise, the degree of its initial water saturation. Change of S_0 by 10 % leads to cardinal quantitative and qualitative changes of stress strain state of the dam and its core crack resistance. For preventing development of high pore water pressure in the core and crack formation, the coefficient of initial water saturation S_0 for sandy loam should not exceed 0.8÷0.85, for loam and clay approximately 0.9.

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