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EARTH CORE ROCKFILL DAM: INTERACTION OF WATER AND SOIL

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Abstract: Accuracy of results of numerical modeling of the structure stress-strain state (SSS) depends on adequacy of design scheme. At elaboration of design scheme of an embankment dam the important aspect is selection of force loads from water. Water medium may create several types of force loads on the dam: hydrostatic pressure, buoyancy and seepage volumetrically distributed forces, as well as arise internal pore pressure in soil.

Selection of design scheme of the earth core rockfill dam depends on permeability of the clayey soil of the seepage-control core and the speed of the seepage regime formation. Analysis of the field observations over the state of constructed dams showed that this process may occur in different ways. In some dams after the reservoir impoundment the seepage regime is installed relatively quickly, in other dams there takes place a durable process of pore pressure dissipation and soil consolidation.

For estimation of the effect of the form and method of loads application, the analysis of the stress-strain state of high earth core rockfill dam for three design schemes was carried out. These are cases with conditionally impermeable, well permeable and scarcely permeable core soil. In the first case the core is subject to hydrostatic pressure, in the second to the force from the steady regime of the seepage flow, in the third, inequality of pore pressure and seepage forces takes place.

For analysis there was used the software package MIDAS, which permits conducting combined analysis of SSS and seepage regime, solving tasks related to consolidation of soils. Coulomb-Mohr model was used for modeling of soil.

Analysis showed that the dam SSS in three design schemes varies greatly. The dam is subject to the highest pressures and deformations in the case of the impermeable core at the action of hydrostatic pressure. The dam SSS at well permeable core, where seepage flow quickly becomes steady, is more favorable. The case of scarcely permeable soil is unfavorable from the point of view of core crack resistance. Due to core pressure, buoyant and seepage forces the stresses in soil skeleton are small. At that, horizontal displacements of the dam by value are closer to displacements in case of water impervious soil. This frequent case may be the most dangerous for the dam safety.

Keywords: earth core rockfill dam, stress-strain state, seepage, pore water pressure, loads, numerical analysis, finite element method

КАМЕННО-ЗЕМЛЯНАЯ ПЛОТИНА: ВЗАИМОДЕЙСТВИЕ ВОДЫ И ГРУНТА

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Аннотация: Точность результатов численного моделирования напряжённо-деформированного состояния (НДС) сооружения зависит от адекватности его расчётной схемы. При составлении расчётной схемы грунтовой плотины важнейшим аспектом является выбор силовых нагрузок от воды. Водная среда может создавать на плотину несколько видов силовых нагрузок: гидростатическое давление, взвешивающие и фильтрационные объёмно распределённые силы, а также вызывать в грунте внутреннее поровое давление.

Выбор расчётной схемы каменно-земляной плотины зависит от проницаемости глинистого грунта противофильтрационного ядра и скорости формирования его фильтрационного режима. Анализ натурных наблюдений за состоянием построенных плотин показал, что этот процесс может происходить по-разному. В одних плотинах после наполнения водохранилища фильтрационный режим устанавливался относительно быстро, а в других происходил длительный процесс рассеивания порового давления и консолидации грунта.

Для оценки влияния формы и способа приложения нагрузок был выполнен расчёт напряжённо-деформированного состояния высокой каменно-земляной плотины для трёх расчётных схем. Это случаи

с условно водонепроницаемым, хорошо проницаемым и маловодопроницаемым грунтом ядра. В первом случае на ядро действует гидростатическое давление, во втором – силы от установившегося фильтрационного потока, в третьем происходит постоянное изменение порового давления и фильтрационных сил.
Для расчётов использовался программный комплекс MIDAS, который позволяет вести совместный расчёт НДС и фильтрационного режима, решать задачи о консолидации грунтов. Для моделирования грунта использовалась модель Кулона-Мора.
Анализ показал, что НДС плотины в трёх расчётных схемах сильно различается. Наибольшие напряжения и деформации плотина испытывает в случае водонепроницаемого ядра, при действии гидростатического давления. НДС плотины в случае хорошо проницаемого ядра, в котором быстро устанавливается фильтрационный поток, является более благоприятным. Случай маловодопроницаемого грунта является неблагоприятным с точки зрения трещиностойкости ядра. Из-за порового давления, взвешивающих и фильтрационных сил напряжения в скелете грунта малы. При этом горизонтальные смещения плотины по величине ближе к смещениям в случае водонепроницаемого грунта. Этот часто встречающийся случай может являться наиболее опасным для безопасности плотины.

Ключевые слова: каменно-земляная плотина с ядром, напряжённо-деформированное состояние, фильтрация, поровое давление, нагрузки, численное моделирование, метод конечных элементов

INTRODUCTION

Analysis of stress-strain state (SSS) is an obligatory element of design validation of embankment dams, which refer to the first and the second category hydraulic structures, such requirement is stated in the code of practice SP 39.13330.2012 «Embankment Dams». These calculations are required for assessment of the dam safety.

Due to complexity, SSS analyses may be carried out only by numerical modeling; it permits solving the most complicated tasks. However, accuracy of numerical modeling depends on adequacy of the structure design scheme: on boundary conditions, construction sequence scheme, etc. One of the important components of the design scheme are combination and character of acting loads; for the dam first of all these are loads from water. Water medium may create several types of loads: hydrostatic pressure, buoyant force (Archimedes force), as well as seepage forces. Hydrostatic pressure is distributed along the surface area and buoyant and seepage forces are distributed in volume. Seepage forces are directed toward the side of going down head.

At conducting analyses, it is necessary to assign loads properly in order to adequately simulate the real conditions of the dam performance. For the dams with a central seepage control element in the form of a clay core (for example, earth core rockfill dam) the issue of the design scheme selection and the types of acting loads is not univocal. The design scheme is determined by the fact whether the clay core is pervious or not.

Like all the soils, the clayey soils are pervious, however, their seepage factor is very small. According to Russian building codes, in seepage control elements there should be used the soils with seepage factor less than 0.1 m/day ($1.2 \cdot 10^{-4}$ cm/s); in real dams it is even less (less than 0.005 m/day).

Evidently due to low permeability of the clayey soil the process of water penetration into the core should be durable, therefore, after the reservoir impoundment the seepage regime is established not at once. In this case the core may be considered as impervious.

By the results of numerical modeling of the non-steady regime conducted in 1, 2], penetration of water from the upstream side into the core should take years. In [2] it is stated that even after dozens of years the most part of the core is still dry. However, this concept is not right because initially the clayey soil is placed into the core at optimal humidity, which corresponds to actually full water saturation. Initial water saturation will contribute to quicker advancement of the seepage area boundary.

However, it does not mean that seepage flow in the core quickly becomes steady. The obstacle will be pore pressure formed in the clayey soil at its compaction under the action of external loads. With time the soil is consolidated: due to seepage the pore pressure is gradually dissipated to the level corresponding to the seepage flow pressure, however, this process may be durable.

To estimate the duration of the consolidation process we fulfilled analysis of data of field observations over the state of dams, published in different sources. In [3, 4] there described the results of field observations over pore pressure in the dams constructed in the XX-th century. Field measurements show that the rate of consolidation process may be variable. In Aswan dam (Egypt, 1970, H=111 m) and Pachkamar dam (Uzbekistan, 1968, H=71 m) the process of the core soil consolidation was quick, but in the core of Talbingo dam (Australia) the pore pressure reached considerable values and dissipated slowly [3]. In ultra-high Nurek HPP dam (Tajikistan, 1980, H=300 m) by the moment of completion of construction, which lasted about 9 years and was performed in parallel with the reservoir impoundment, the most part of pore pressure was dissipated by the moment of start of operation [4].

Field measurements on the dams constructed in the XXI-st century [5–10] also confirm that there may be two possible cases.

In the first case the soil consolidation is quick and is nearly fully completed during construction period. Such case took place in ultra-high Nuozhadu dam (China, 2012, H=261.5 m) [5, 6]. By the start of operation period the seepage flow stabilized in the core of Eyvashan dam (Iran, 2015, H=71 m) [7, 8]. Such case is possible only when the core soil is sufficiently pervious.

The second case refers to durable soil consolidation, which is not completed during construction period and lasts for a long time during operation period. Such case took place in Karkhe dam (Iran, 2001, H=127 m). In spite of the fact that the reservoir impoundment was in parallel with the dam construction, pore pressure was still high for a long time [9]. Durable high pore pressure is considered to be the cause of increased deformations of Masjed-e-Soleyman dam (Iran, 2002, H=177 m) [10]. This case occurs if the core soil has low water permeability.

The design scheme of loads action corresponds to each of two considered cases. In the first case (permeable soil, quick consolidation) the load from water acts in the form of volumetrically distributed seepage forces.

In the second case (low soil permeability, durable consolidation) the character of loads is more complicated: it varies with time. At first, before the reservoir impoundment, pore pressure appears in the placed soil. After some time, following the appearance of hydraulic connection between the pores there will appear seepage forces in soil corresponding to pore pressure. Then, immediately after the reservoir impoundment the hydrostatic pressure appears on the core upstream face. Then, it will gradually disappear and will be replaced by seepage forces directed inside the core. The seepage regime will gradually change until it becomes steady.

Hydrostatic pressure on the core upstream face may also appear if reservoirs are impounded quickly after completion of the dam construction. Such case took place at construction of Gavshan dam (Iran, 2004, H=123 m) [11], Luding (China, 2016, H=84 m) [12].

As we see, loads from water may not only obtain different forms but may be transformed from one form to another.

At conducting SSS analyses different authors use various schemes of loads application. For example, in calculations [12–16] the core was assumed impervious and was loaded by hydrostatic pressure. In calculations [7, 8] the core was assumed fully permeable. And in calculations [5, 10] the soil was assumed scarcely permeable and SSS analyses were with consideration of PWP.

The aim of this study is revealing real conditions of formation of an earth core rockfill dam SSS and selection of the most adequate design scheme.

Similar study was fulfilled in [17]. At SSS analysis two cases were considered: 1) reservoir is impounded only after the dam is constructed for the full height; 2) reservoir is impounded gradually with growth of the dam height. Calculations showed that impoundment sequence has small effect on the dam stress state, but its integral displacement in case 1 is more than that in case 2. However, in calculations [17] the cases with permeable and low permeable soils were not taken into account. Calculations were carried out for relatively high seepage factor of the core soil (0.006 m/day), therefore, the effect of appearing increased pore pressure in the core was not revealed.

METHODS

For reaching the targeted aim the results of SSS analyses for several possible design schemes were compared. Three schemes were considered.

Scheme №1. The core is assumed to be fully impervious. In this case the load from water acts in the form of hydrostatic pressure on the core upstream face, and the core clayey soil is assumed to be water saturated (due to placement at optimal humidity). This scheme corresponds to the moment of time after quick reservoir impoundment.

Scheme №2. The core is well permeable and the seepage flow in the core is steady. In this case the load from water is realized in the form of seepage forces distributed in the core body. At that, the core soil is in suspended by water state, the pore pressure in it corresponds to the seepage pressure. This scheme is also possible at soil low permeability after the durable period of the dam operation.

Scheme №3 corresponds to the case when the core soil has low water permeability. In this case the increased pore pressure occurs in soil. As compared to scheme №2 in this design scheme the seepage flow in the core is still not stabilized, seepage forces are always variable in time by value and direction.

Calculations by schemes №2 and №3 pose a more complicated task than those by scheme №1. In scheme №2 consideration of seepage forces requires preliminary analysis of steady seepage regime in the core. In scheme №3 solving the task on the dam SSS should be performed jointly with solving the seepage task. For solving the static-seepage task, a special software is required.

Such possibility is provided by software package MIDAS, which uses the finite element method (FEM).

Analyses were conducted on the example of the ultra-high Earth Core Rockfill Dam (ECRD), whose structural design is similar to that of Nurek HPP dam (fig. 1). The dam is located on rock foundation. The height of fill in the considered section is 292.5 m.

The dam central core is made of sandy loam, it rests on bulk concrete. The shells are made of gravel-pebble soil and are covered by a layer of rock mass.

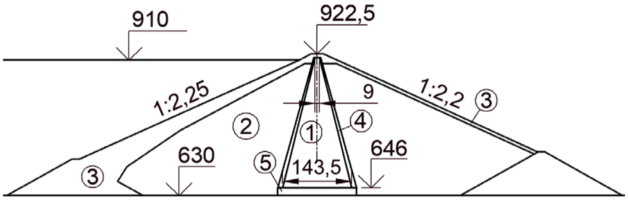


Figure 1. Scheme of rockfill dam structure
1 – core of sandy loam; 2 – shells of gravel-pebble soil; 3 – protection from rockfill; 4 – transition zone; 5 – concrete mass

The dam finite element model consists of 6975 nodes and 6906 finite elements.

During analyses the sequence of dam construction and reservoir impoundment was taken into account; they were realized in parallel. Duration of construction is about 9 years.

Coulomb-Mohr model was used for description of deformation and strength of soils. The following physical and mechanical properties of materials were used during calculations: ρ_d – density in dry state, ρ_{sat} – density in water saturated state; E – modulus of linear deformation; ν – Poisson's ratio, φ – angle of internal friction, c – cohesion. Parameters of shear strength (φ and c) of soils were taken from analogs. They are determined in [18] by processing of experimental results [19, 20]. Cohesion for macro-fragmental soils simulates the effect of gearing.

The adopted in the analysis characteristics of soils are presented below:

For core sandy loam: ρ_d=2.10 t/m^3, ρ_{sat}=2.28 t/m^3, E=40 MPa, ν=0.32, φ=33°, c=100 kPa;

For gravel-pebble soil: ρ_d=2.16 t/m^3, ρ_{sat}=2.35 t/m^3, E=150 MPa, ν=0.27, φ=37.9°, c=112 kPa;

For rock mass: ρ_d=1.96 t/m^3, ρ_{sat}=2.22 t/m^3, E=150 MPa, ν=0.27, φ=39.3°, c=250 kPa;

For soil of transition zones: ρ_d=2.16 t/m^3, ρ_{sat}=2.35 t/m^3, E=55 MPa, ν=0.30, φ=32°, c=60 kPa.

Sandy loam seepage factor was taken equal $8.64\cdot10^{-5}$ m/day, macro-fragmental soils were taken more permeable by several orders.

RESULTS AND DISCUSSION

Results of SSS analyses of the considered ECRD are presented for the moment of completion of construction and reservoir impoundment. Fig.2–5 shows distribution of the dam displacements and stresses for three considered design schemes. In the table there indicated the maximum values of horizontal (U_x) and vertical (U_y) displacements of the dam.

Table. Maximum values of displacements

Scheme	1	2	3
U_x [cm]	287	191	264
U_y [cm]	440	366	346

Analysis shows that the dam SSS in schemes 1, 2, 3 principally differs from each other.

SSS in scheme 1 (impervious core) is characterized by maximum by value stresses and deformations. Several specific features may be marked.

The first peculiarity refers to the fact that in the impervious core the soil is not subject to buoyant force of water and consequently, it has large weight. Therefore, the dam has maximum settlements (440 cm, fig.2,a), and high vertical stresses (4.5 MPa, fig.4,a) appear in the core. The settlement comprises 1.5% of the dam height. Distribution of vertical stresses evidences about presence of arch effect, when there occurs deficit of compressive stresses in the core and their surplus in the shells.

The second peculiarity is related to the fact that at the boundary of the core with the upstream shell there appears a jump in value of horizontal stresses. It is caused by hydrostatic pressure on the core upstream face. Concentrated action of water causes increased horizontal displacements of the core near the upstream face (fig.4,a). It is scheme 1 where the dam is subject to maximum by value horizontal displacements (287 cm).

In scheme 2 the core is subject to pore pressure corresponding to the steady seepage regime (fig.6.a). It was determined at each stage by the seepage analysis.

In scheme 2 the dam SSS is characterized by less level of compression in the core as compared to scheme 1. Due to buoyant force of water the vertical stresses in the core do not exceed 3.5 MPa (fig.4,b). the dam maximum settlement amounts to 366 cm (fig.2,b), which is by 17% less, than in scheme 1.

At analysis by scheme 2 the dam has minimal horizontal displacements; they do not exceed 2 m (fig.3,b). They are less by one third that in scheme 1. The characteristic feature of SSS is low level of compression in the core near the upstream face, but toward the downstream face the horizontal compressive stresses gradually increase (fig.4,b).

In scheme 3 SSS is not only of another character than in schemes 1 and 2, but is not within the limits between them. Specific features of SSS refer to appearance of increased pore pressure in the dam core due to external loads.

Maximum dam settlement in scheme 3 amounts to 346 cm, which is even less than in scheme 2 (fig.2,c). This decrease is due to presence of seepage gradient directed upward. The dam maximum horizontal displacement amounts to 264 cm (fig.3,c); by value it is close to the displacement in scheme 1.

Maximum difference of SSS is revealed in the core stress state, because increased pore water pressure (PWP) acts in it. By the results of analysis, the growth of PWP occurs during the whole construction period. By its completion the maximum PWP reaches 3 MPa (fig.6.b). This is relatively small due to long duration of construction and other factors [21].

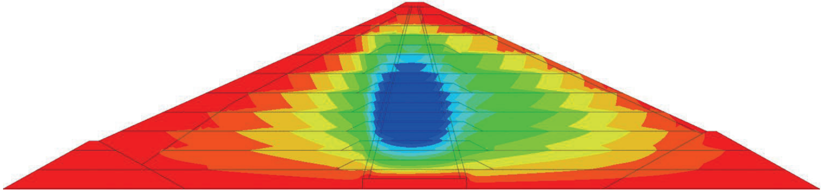
Due to increased PWP for the dam SSS in scheme 3 the minimum level of compression in the core is a characteristic feature. Horizontal stresses are noticeably low than in scheme 2. The field of low compressive horizontal stresses is located in the core lower part (up to 0.5 MPa, fig.5,c). Considerable decrease of compression also refers to vertical stresses. No tensile stresses are formed in the core, but in the upper part of the core the vertical stresses are small (fig.5,c). The low level of effective compressive stresses creates threat for the core hydraulic break and development of fractures in it.

One might say that out of the three considered design schemes it is scheme 3 that turned to be the most unfavorable. This is confirmed by the experience in operation of Masjed-e-Soleyman dam [10], where the durable process of soil consolidation became a possible cause of the core seal failure.

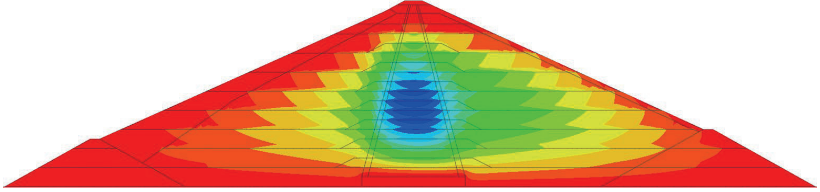
Besides, attention should be paid to one more characteristic feature of the dam SSS in scheme 3. It refers to increased level of compression in the part of the downstream shell adjoining to the core. Increase of compression is observed both in vertical (fig.4,c), and horizontal (fig.5,c) stresses. It is explained by the fact that due to presence of pore pressure the most part of load is transferred from the core to the downstream shell.

Stress state of the upstream shell in all the schemes is actually the same.

a) scheme 1 with waterproof core



b) scheme 2 with a permeable core



c) scheme taking into account pore water pressure

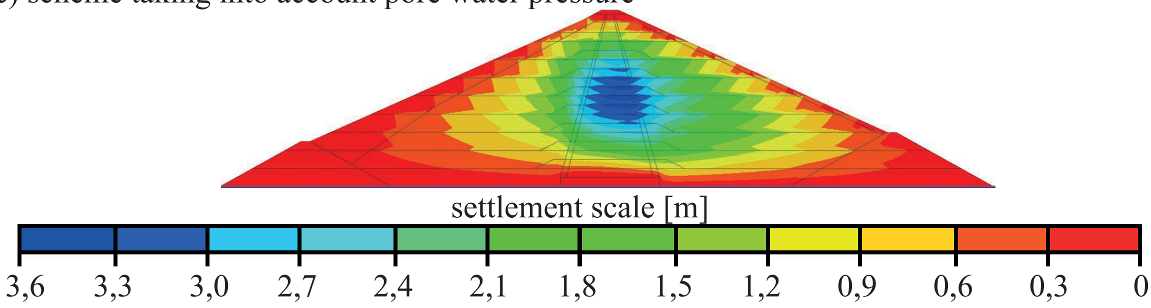
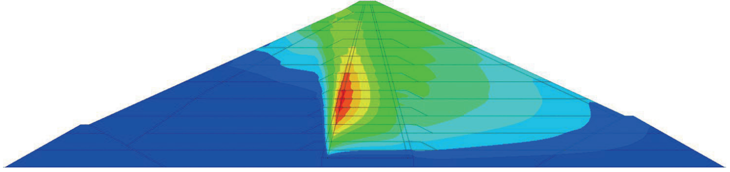
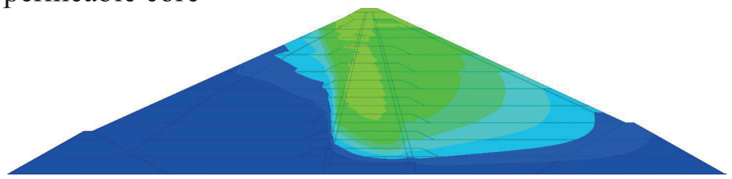


Figure 2. Dam settlements at the end of the construction period

a) scheme 1 with waterproof core



b) scheme 2 with a permeable core



c) scheme taking into account pore water pressure

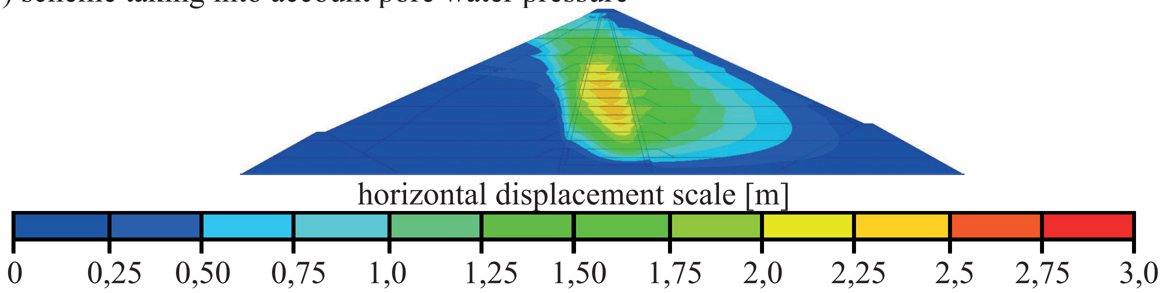
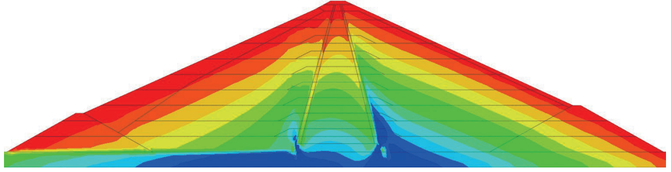
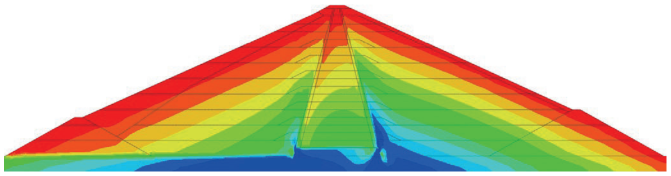


Figure 3. *Horizontal displacements of the dam during the construction period*

a) scheme 1 with waterproof core



b) scheme 2 with a permeable core



c) scheme taking into account pore water pressure

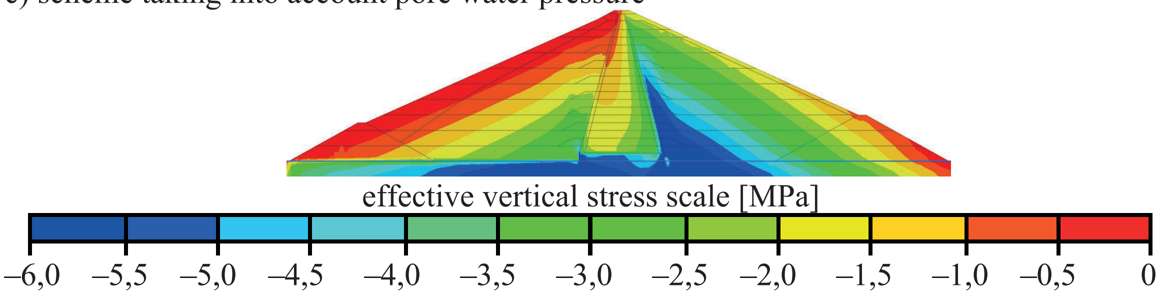
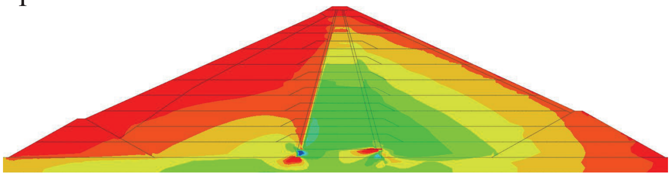
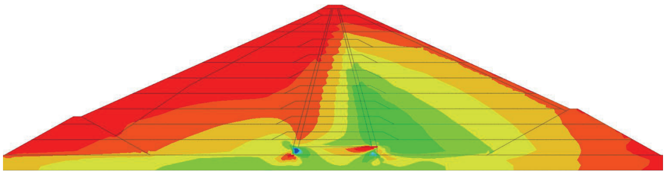


Figure 4. *Vertical stresses in the dam soil*

a) scheme 1 with waterproof core



b) scheme 2 with a permeable core



c) scheme taking into account pore water pressure

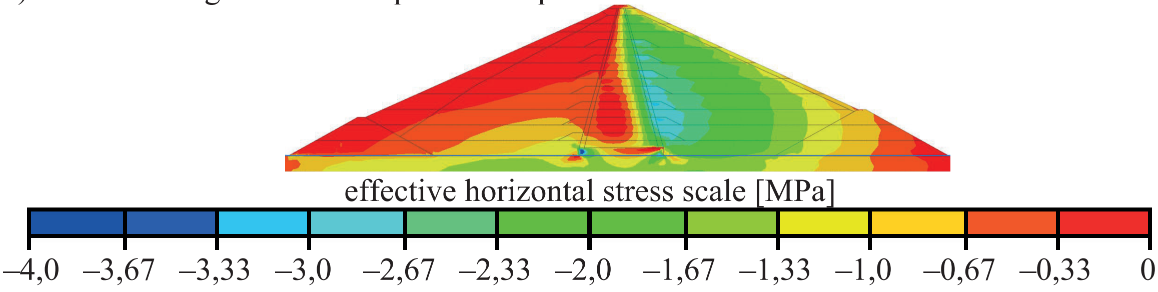


Figure 5. Horizontal stresses in the dam soil

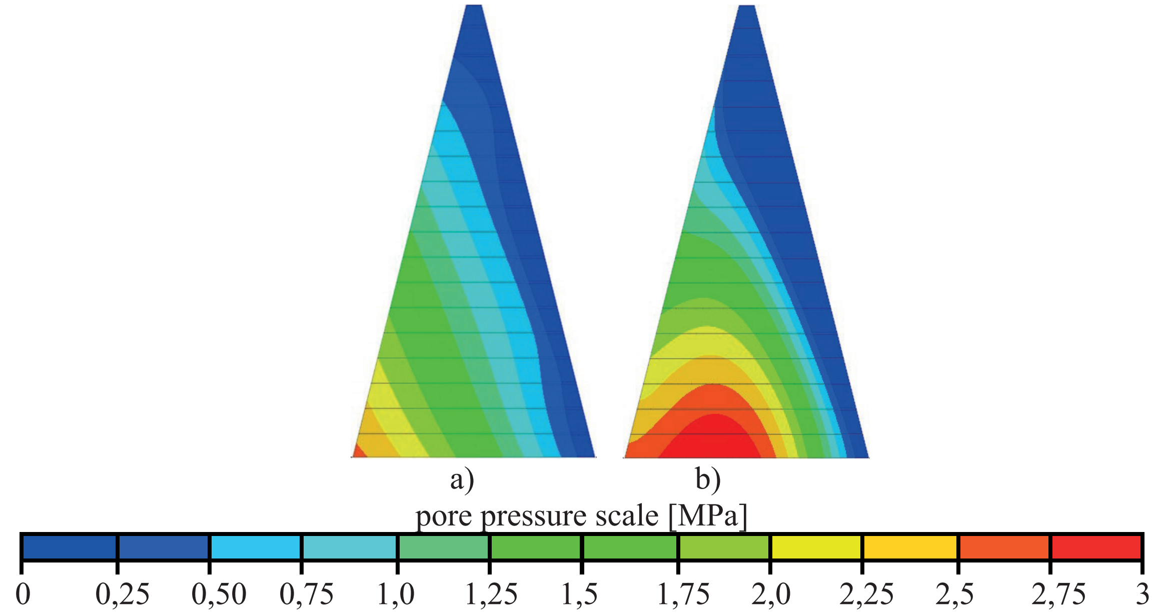


Figure 6. Pore water pressure in the dam core:
a – filtration pore pressure at steady state filtration conditions, b – pore pressure at the time of completion of construction in soil consolidation mode

CONCLUSION

1. The method of application of loads from water greatly affects the displacements of an earth core rockfill dam and stress state of its seepage control core. Therefore, accuracy of numerical modeling of stress-strain state of such dam is greatly dependent on adequacy of its design scheme and, first of all, on the scheme of application of loads from water.
2. Field investigations of stress-strain state of earth core rockfill dams show that different forms of loading by water forces on the structure may take place. They are determined by core soil permeability. The design scheme should be chosen depending on this factor.
3. Stress-strain state of an earth core rockfill dam obtained in each of the considered schemes of loads application is characterized by their own unfavorable effects. At the impervious core the dam is subject to maximum displacements and stresses. At rather permeable core, steady seepage flow, the dam SSS is the most favorable. The case with low permeable soil is characterized by low level of compression in the core due to appearance of pore pressure in soil. The dam SSS in this case may present danger from the point of view of the core crack resistance.
4. The dam seepage regime varies with time, and loads from water transfer from one type to another: from hydrostatic pressure to the forces from steady seepage flow. At that, the dam stress-strain state also transforms. For the considered dam the intermediate state at unsteady seepage regime turned to be dangerous.

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