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Analysis of normal operation of a rockfill dam with combination of seepage-control elements: reinforced concrete face and clay-cement-concrete wall

Анализ работоспособности каменной плотины с комбинацией противофильтрационных элементов – железобетонного экрана и глиноцементобетонной стены

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Key words: concrete faced rockfill dam; cut-off wall; stress-strain state; numerical modeling

Ключевые слова: каменная плотина с железобетонным экраном; стена в грунте; напряжённо-деформированное состояние; численное моделирование

Abstract. The paper considers results of stress-strain state analysis for a 87 m high rockfill dam that is located in the layer of soil foundation and has a combined seepage-control element. In the lower part of the dam as well as in its foundation there is fixed a grout curtain by using slurry trench cut-off walls, and in the upper part there is fixed a reinforced concrete face. The wall and the face are conjugated via the reinforced concrete gallery. Analyses showed that both seepage-control elements have a favorable stress-strain state: no tensile stresses occur in them. In the considered dam the reinforced concrete face operates in more favorable conditions than in a traditional dam with a reinforced concrete face. This allows recommending this type of the dam for practical application in hydraulic engineering. The alternative is proposed with the design of conjugation of a reinforced concrete face and a seepage-control wall.

Аннотация. Рассматриваются результаты расчёта напряжённо-деформирования состояния высокой каменной плотины высотой 87 м, расположенной на слое нескального основания и имеющей комбинированный противофильтрационный элемент. В нижней части плотины, а также в её основании методом «стена в грунте» устроена противофильтрационная завеса, а в верхней части – железобетонный экран. Сопряжение стены и экрана осуществляется через железобетонную галерею. Расчёты показали, что оба противофильтрационных элемента имеют благоприятное напряжённо-деформированное состояние – в них не возникает растягивающих напряжений. В рассмотренной плотине железобетонный экран работает в более благоприятных условиях, чем в классической плотине с железобетонным экраном. Это позволяет рекомендовать данный тип плотины для внедрения в практику гидротехнического строительства.

Introduction. Task formulation

The necessity to enhance safety of embankment dams makes search for their new structural solutions, including new structural designs of their seepage-control elements.

Among the not-natural (rock or soil) seepage-control elements of the embankment dam body the reinforced concrete face is considered to be the most promising today. The height of constructed rockfill dams with reinforced concrete faces already exceeded 230 m. However, at some of these dams emergency situations occurred, where cracks appeared in the face [1–8]. One of the causes for crack formation is tensile stresses in the face. Tension may occur due to bending deformation [3], as well as due to longitudinal tensile deformation [9].

Among seepage-control measures in the earth foundation under earthfill dams the most popular were cut-off walls [1–14]. Design studies show [15], that the stress-strain state of such walls may be favorable, if they are made of clay-cement. Under the dam dead weight the foundation soil settles and presses the seepage-control wall, which prevents development of tensile stresses. Recently cut-off walls began to be applied not only in foundations of earthfill dams but as seepage-control elements of the dam body [16, 17].

In case the construction of a rockfill dam is planned on a thick layer of the earth, it is reasonable to consider the dam alternative, where in the dam body the seepage-control element is presented by a reinforced concrete face and in the structure foundation is like a cut-off wall. This is an eartfill dam with a combined seepage-control element.

This article deals with the analysis of workability of such a dam based on the stress-strain state (SSS) analysis.

Description of the structure under study

There was considered a 87 m high rockfill dam located on a 22.5 m thick layer of the gravel-pebble soil (Fig. 1). The dam body is made of rock mass. The adopted seepage-control element in the earth foundation is a clay-cement wall constructed by the bored pipe wall method. Besides, this wall is a seepage-control element of the upstream cofferdam being part of the dam body. The total depth of the wall is 51 m. In the upper part of the dam the seepage-control element is presented by a 0.5 m thick reinforced concrete face.

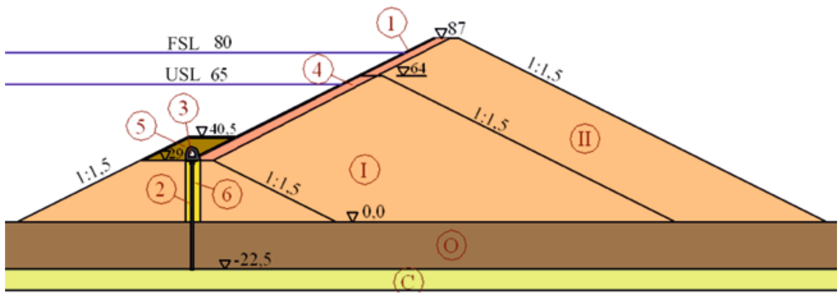

Figure1. Design of the earthfill dam under study
I, II – rockfill dam construction stages, O – earth foundation layer, C – rock foundation,
1 – reinforced concrete face, 2 – cut-off wall, 3 – reinforced concrete gallery, 4 – under-face zone,
5 – filling low-permeable soil, 6 – gravel-sand core in the upstream cofferdam body

There is a gallery arranged at the interface of the reinforced concrete face and the wall. The gallery is located above the top part of the wall, however, the wall does not abut the gallery to provide free settlements of the gallery.

Studies of the dam SSS were conducted by numerical modeling using the finite element method.

Quadrilateral finite elements were used to describe the continuous medium behavior. Contact elements of zero thickness were used to model the contact behavior of earth structures between each other. The numerical model consisted of 721 finite elements (out of which 66 are contact elements), and 746 corner nodes. All the finite elements had a cubic function of displacement approximation. The total number of degrees of freedom in the numerical model was 7222.

Analyses were conducted with the NDS-Ncomputer program developed by Sainov M.P. at the Chair of Hydraulic Engineering of Moscow State Civil Engineering University [18]. This permitted considering non-linearity of soil behavior, as well as non-linearity of interaction of earth structures with each other and with soil mass. The modified soil model proposed by Rasskazov L.N. was used to consider non-linear deformation of soils [19, 20]. Coulomb model was used to assess shear strength of contacts. The face reinforced concrete and the cut-off wall material in the analysis were assumed to be elastic. Deformation modulus of reinforced concrete was assumed equal to 29000 MPa (Poisson's ratio $\nu = 0.18$). Deformation modulus of clay-cement was assumed equal to 100 MPa (Poisson's ratio $\nu = 0.2$). The clay-cement had such a Deformation modulus that its composition should include 120…140 kg of bentonite and 120…160 kg of cement [15].

Analyses were conducted allowing for the staged dam construction. At the first stage the upstream cofferdam and the dam foundation should be constructed. Then a seepage-control wall is constructed in the gravel-sand core of the cofferdam and then a gallery above it. The first-stage CFRD is constructed under protection of the cofferdam (to ∇ 69 m). Afterwards the reservoir is filled to ∇ 65 m. Then the second-stage dam is constructed and the reservoir is filled to ∇ 80 m.

Technological features of constructing seepage-control structures were also taken into account. The arrangement of the cut-off wall was modeled by replacing one material (gravel-sand soil) in the structure model with another (clay-cement). However, it was taken into account that the wall clay-cement takes up the dead weight in the non-hardened state. Its Poisson's ratio is taken equal to 0.45, and the deformation modulus is 20 MPa. Clay-cement took up other loads after gaining strength. A reinforced concrete face was constructed fully for the whole height of the dam stage after the completion of its filling.

Totally 52 steps of the analysis were considered and each step was characterized by modeling parts of the structure or occurring a new load.

Analysis results

Figures 2–8 show the results of the earthfill dam SSS analysis with a cut-off wall and a reinforced concrete face for two moments of time. The first one is before the second-stage reservoir filling, the second after the completion of construction and the reservoir filling to ∇ 80 m. In the figures the red curves correspond to the first moment of time, the green ones – to the second.

The analyses showed that main displacements and settlements occurred in the reinforced concrete face and the wall after the reservoir filling. The character of settlement distribution of the reinforced concrete face and that of the wall is different. Nearly the uniform distribution pattern during construction stages is peculiar for the reinforced concrete face. For example, horizontal displacements of the second-stage dam face are in the range of 22.2 ÷ 23.9 cm (Fig. 2), and settlements – 12.2…15.6 cm (Fig. 3).

Figure 4 shows the face displacements in the direction to normal. The shape of the curve (Fig. 4) evidences that the face bending deformations are not large. Bending occurs only in the zone of abutment to the concrete gallery and that of the first-stage dam crest. In the first case the bend occurs toward the downstream side, in the second – towards the upstream side.

Figure 5 shows the face longitudinal displacements, i.e. the displacements are directed along the slope. After the reservoir filling these displacements are directed from the face foot to the dam crest. They reach 15 cm. The occurrence of such displacements is caused by the fact that the face horizontal displacements have a larger value than their settlements. As in the lower part of the face the longitudinal displacements are somewhat larger than in the upper part, the face is subject to a pressing longitudinal force.

The wall displacement distribution pattern is determined by locking its lower end by the rock foundation. The wall foot displacements and settlements are actually equal to 0. Maximum wall displacements and settlements are observed in their top part. At the moment of the end of reservoir filling the top maximum displacement amounted to 21.5 cm (Fig. 2). The shape of the displacement curve is close to linear, which evidences about low development of bending deformations.

Maximum displacements of the wall top amounted to 41.2 cm (Fig. 3). A large value of settlements is explained by the fact that they are caused by the wall dead weight when clay-cement is in a semi-liquid state. Settlements of the top under the impact of other loads accounted for only 15.5 cm, i.e. nearly similar to the concrete gallery settlements (16 cm).

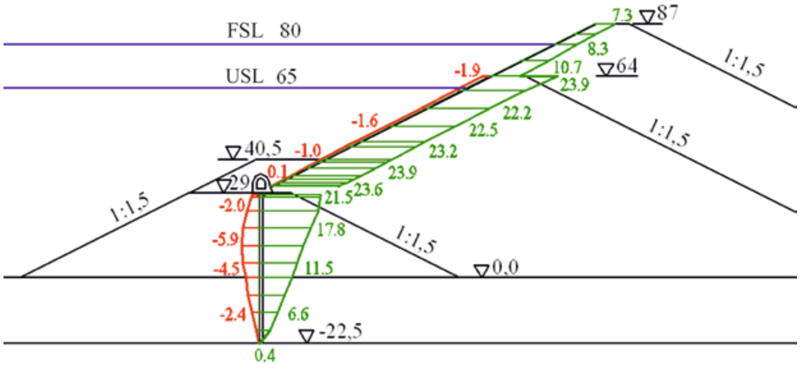

Figure 2. Horizontal displacements (cm) of dam seepage-control elements
Red color indicates displacements before the reservoir filling.
Green color indicates displacements at the reservoir filling to ∇ 80 m

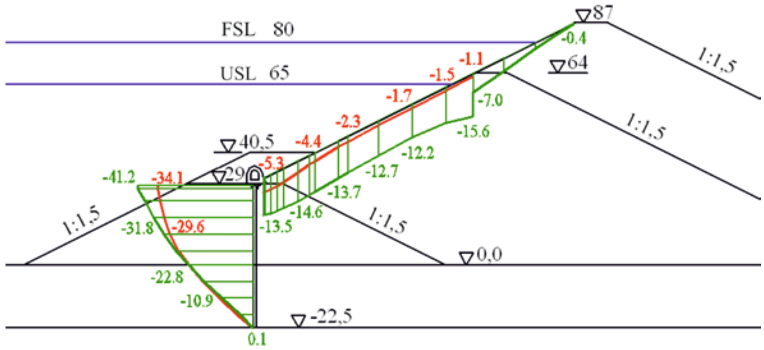

Figure 3. Settlements (cm) of dam seepage-control elements

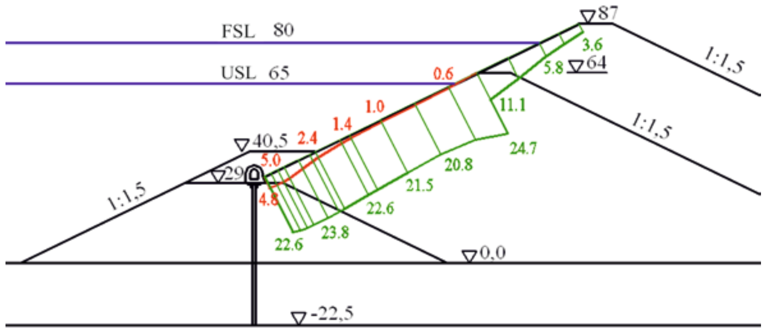

Figure 4. Reinforced concrete face displacements (cm) in the direction perpendicular to the slope

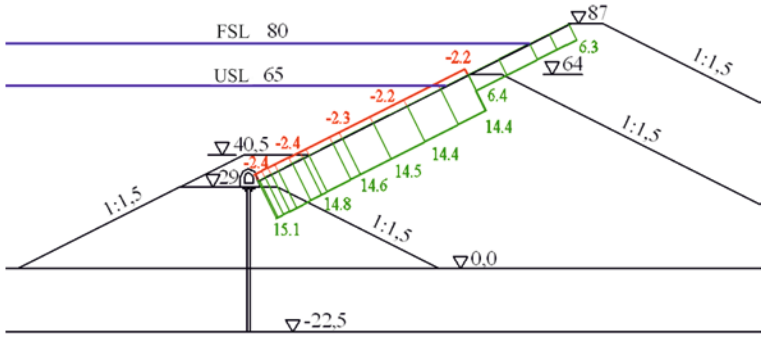

Figure 5. Reinforced concrete face displacements (cm) in the direction along the slope

It is interesting that displacements in the zone of the reinforced concrete face interface with the gallery: displacements in the perimeter joint. At the reservoir filling the perimeter joint opens for 0.4 mm. The value of shear displacements in the joint is considerably larger – 2.9 mm. Such deformations are not dangerous for the joint tightness; they may be perceived by water stops.

Figures 6–7 shows the distribution of stresses in the reinforced concrete face. Stresses in the direction across the slope are formed by water hydrostatic pressure; they are not large. Stresses in the direction along the slope are determined by a character of deformation distribution of the face and rock fill under it. Before the reservoir filling the zones of tensile longitudinal stresses are formed due to bending in the face. At the upstream face they reach 2.08 MPa (Fig. 6), at the downstream face they are 2.98 MPa (Fig. 7). These stresses may be taken up by reinforcement.

After the reservoir filling the compressive longitudinal deformations compensate tensile stresses in the face, therefore the face SSS becomes more favorable. The upstream face is compressed by longitudinal stresses actually for the whole length (Fig. 6). Compressive stresses reach 7.14 MPa. Not large tensile longitudinal stresses are connected with face local bending deformations. On the downstream face there are only compressive longitudinal stresses (Fig. 7); they do not exceed 4.3 MPa. However, in the zone where the face abuts the concrete gallery there formed the zone of compressive stress concentration equaling up to 12.8 MPa.

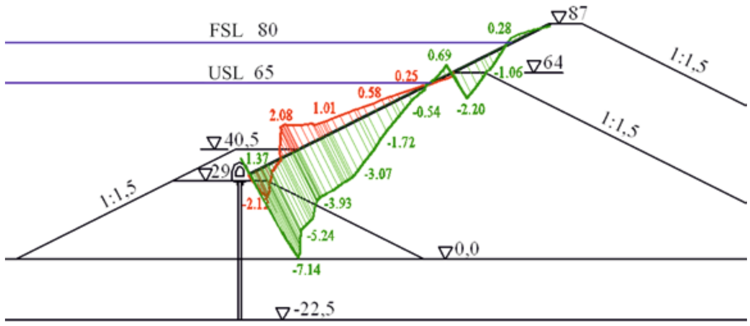

Figure 6. Stresses (MPa) on the upstream face of the reinforced concrete face in the direction along the face

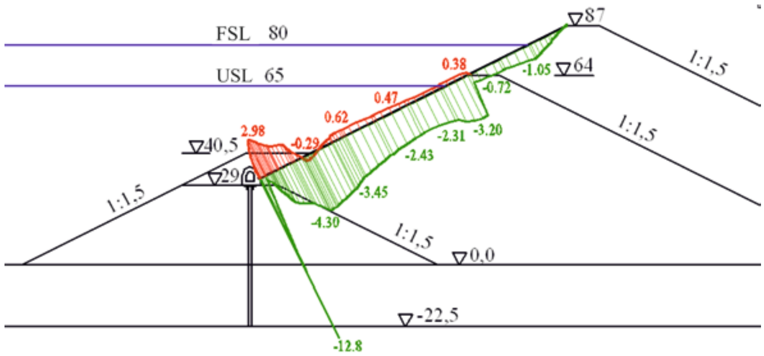

Figure 7. Stresses (MPa) on the downstream face of the reinforced concrete face in the direction along the face

The seepage-control wall has favorable SSS: it is compressed from all sides. The maximum level of compression is observed in a vertical direction by stresses σ_y (Fig. 8). This is explained by the fact that the soil surrounding the wall settles under the weight of the earthfill dam and also compresses the wall. Stresses σ_y are distributed heightwise uniformly. Compression deformations prevail over bending deformations, therefore, stresses vary slightly within the thickness of the wall. The maximum value of compressive stresses σ_y in the wall amounts to 1.1 MPa. It is less than the clay-cement uniaxial compression strength, and it is equal to 1.27 MPa at E = 100 MPa [15, 21].

Opening the wall contact with the rock foundation occurs for the length of 20 cm, which comprises only 1/6 of the wall thickness.

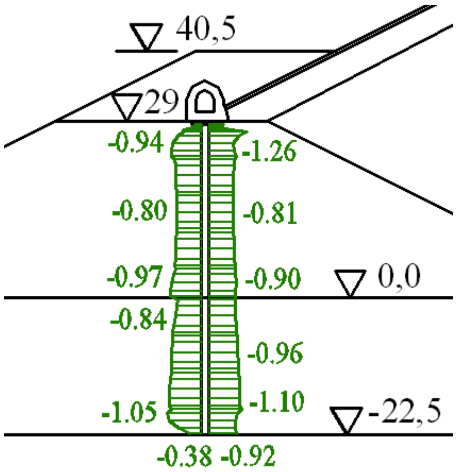

Figure 8. Stresses σ_y (MPa) on the faces of the seepage-control wall

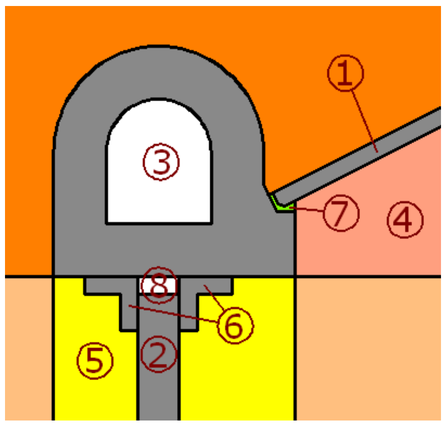

Figure 9. Scheme of conjugation for seepage-control elements 1 – reinforced concrete face, 2 – wall, 3 – gallery, 4 – under-face zone, 5 – sand core, 6 – foreshaft, 7 – filling the perimeter joint, 8 – air cavity

Thus, in the considered dam design both seepage-control elements are in a favorable stress state: they are compressed, but the compression level is not large. Compressive strengths may be disturbed only in the interface zone of the reinforced concrete face and the gallery, where stresses are concentrated up to 12.8 MPa. There is a danger of cleavage of the thin face corner, which can lead to the seal failure of the perimeter joint.

To prevent the concentration of compressive stresses in the reinforced concrete face, the interface of the reinforced concrete face with the concrete gallery may be provided in the form of soft hinged connection (Fig. 9).

Conclusion

1. On the whole, the design of the high earthfill dam with two combined seepage-control elements (a reinforced concrete face and a clay-cement cut-off wall) is favorable and safe. In dam displacements under the impact of hydrostatic pressure the seepage-control elements have free displacements not subjected to strong bending deformations. The connecting gallery plays the role of a hinge. On the one hand, it preserves connection of seepage-control elements; on the other hand, it compensates possible bending deformations.

In the considered structural design both seepage-control elements are in a favorable stress state: they are compressed actually in all the sections. The connection points of the seepage-control elements with the connecting gallery work reliably: displacements in the joints are not large; their tightness may be usually provided by the used seals.

2. The only disadvantage of the considered design is the concentration of compressive stresses on the downstream surface of the reinforced concrete face in the interface zone with the gallery. To cope with this disadvantage, it is proposed to make a soft connection between the face and the gallery filling the perimeter joint with a soft polymer material.

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