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Serviceability of rockfill dam with reinforced concrete face and grout curtain

Работоспособность каменно-набросной плотины с железобетонным экраном и инъекционной завесой

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Ключевые слова: каменно-набросная плотина; железобетонный экран; инъекционная завеса; напряжённо-деформированное состояние; численное моделирование

Abstract. The article deals with design validation of a new structural design of an embankment dam, i.e. rockfill dam with seepage-control element of composite design. It consists of a reinforced concrete face (in the dam upper part) and a wide grout curtain (in the dam lower part). Analyses of the dam stress-strain state (SSS) were conducted on the example of a 235 m high dam. Numerical modeling of the dam was performed with consideration of its construction and loading sequence, as well as non-linear deformation of rockfill. Impact of rockfill deformation and grout curtain material on the dam SSS was studied. It was revealed that conditions of the reinforced concrete face operation in the considered dam structural design differ from conditions of its operation in the dam of t classical design; it is subject to not tensile but compressive longitudinal force. This effect decreases the risk of cracking in the face. However, it should be taken into account that under certain conditions the compressive longitudinal stresses in the face may exceed the concrete compressive strength, therefore, the face thickness (in the lower part) is recommended to be taken equal more than 2 m. The least safe assembly of the considered dam structural design is interface of two seepage-control elements. It is arranged with the aid of a concrete gallery located under the grout curtain. SSS of the reinforced concrete face lower part greatly depends on the grout curtain deformations. To provide the face safety the deformation modulus of the grout curtain material should be not less than that of rockfill. The grout curtain strength and SSS are mainly determined by the material deformation. At high rigidity of the curtain material there is a danger of appearance in it of tensile stresses and separation of the curtain from the rock foundation. It is recommended to arrange the grout curtain of clay-cement grouts so that deformation modulus of the curtain material does not exceed 500 MPa.

Аннотация. Статья посвящена расчётному обоснованию новой конструкции грунтовой плотины – каменно-набросной плотины с противофильтрационным элементом составной конструкции. Он состоит из железобетонного экрана (в верхней части плотины) и широкой инъекционной завесы (в нижней части плотины). Расчёты напряжённо-деформированного состояния (НДС) плотины проводились на примере плотины высотой 235 м. Численное моделирование плотины проводилось с учётом последовательности её возведения и нагружения, а также с учётом нелинейности деформируемости каменной наброски. Было исследовано влияние на НДС плотины деформируемости каменной наброски и материала инъекционной завесы. Было получено, что условия работы железобетонного экрана в рассмотренной конструкции плотины отличаются от условий его работы в плотине классической конструкции – экран испытывает не растягивающее, а сжимающее продольной усилие. Этот эффект снижает риск образования трещин в экране. Однако следует иметь ввиду, что при определённых условиях сжимающие продольные напряжения в экране могут превышать прочность бетона на сжатие, поэтому толщину экрана (понизу) рекомендуется принимать больше 2 м. Наименее надёжным узлом рассмотренной

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конструкции плотины является сопряжение двух противофильтрационных элементов. Оно выполнено с помощью бетонной галереи, которая расположена над инъекционной завесой. НДС нижней части железобетонного экрана сильно зависит от деформаций инъекционной завесы. Для обеспечения прочности экрана необходимо, что модуль деформации материала инъекционной завесы был не меньше, чем у каменной наброски. Прочность и НДС инъекционной завесы в основном определяется деформируемостью её материала. При высокой жёсткости материала завесы возникает опасность появления в ней растягивающих напряжений и отрыва завесы от скального основания. Рекомендуется выполнять инъекционную завесу из глинисто-цементных растворов с тем, чтобы модуль деформации её материала не превышал 500 МПа. По результатам исследования можно сделать вывод, что рассмотренная конструкция каменно-набросной плотины с составным негрунтовым противофильтрационным элементом является хорошей альтернативой для классической конструкции грунтовой плотины с железобетонным экраном.

1. Introduction

At present at construction of ultrahigh dams (more than 200 m high) 2 types of embankment dams are used: a rockfill dam with a central core and a rockfill dam with a reinforced concrete face (CFRD). The highest rockfill dam is Nurek dam (Tajikistan) 300 m high [1], and the height of the highest Shuibuya CFRD in China is 233 m [2].

As construction and operation of a rockfill dam in severe climatic conditions is complicated at elaboration of designs for construction of ultrahigh embankment dams, only one structural design alternative was considered – CFRD [3–6]. However, it is known that CFRDs are insufficiently safe: cracks appeared in reinforced concrete faces of several ultrahigh dams [7–9]. In order to provide safety of CFRDs and to extend the area of their application the studies are conducted of their stress-strain state (SSS) [4, 10–12]. Based on studies there are improved structural designs of ultrahigh dams and measures are worked out on enhancing their safety. Namely, it is proposed to arrange transversal joints in the reinforced concrete faces, optimize the dam construction sequence [10, 12] and even make the under-face zone of soil-cement-concrete [11].

However, according to our studies [11, 12], these improvements of classical structural designs of CFRD does not allow guaranteeing their safety. This is connected with peculiar feature of such dam performance: at dam deformations in the lower part of the reinforced concrete face there appears a longitudinal tensile force, which results in formation of joints in the face. Therefore, it is necessary to search for other ways of improving structural designs of ultrahigh CFRD. Namely, one of such ways is use of a seepage-control element of composite (combined) design.

One of the alternatives of rockfill dam with a combined seepage-control element is design where a reinforced concrete face (in the dam upper part) is combined with the grout curtain (in the dam lower part) (Figure 1).

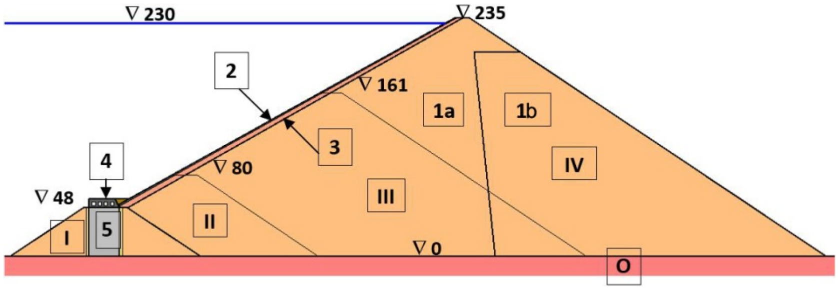


Figure 1. Dam design with combination of seepage-control elements: a reinforced concrete face and a grout curtain. 1a, 1b – rockfill, 2 – reinforced concrete face, 3 – under-face zone, 4 – concrete gallery, 5 – grout curtain, I, II, III, IV – dam construction stages

This structural design was proposed by VNIIG as an alternative for construction of ultrahigh (235 m) dam of Kankun HPP in South Yakutia [13]. The advantage of this structural design is possible increase of dam construction rates due to refusal from pit excavation for construction of a seepage-control element. It is envisaged that the 1st stage dam is filled as a rock-earthfill dam with a wide central core of sand-gravel soil. Then this core is injected with cement mortar (or cement-clay) and is turned into a grout curtain. The 1st stage dam 48 m high can take up the head required for passing water through a bypass along the closed river channel. The dam main part is arranged with a seepage-control element (SCE) in the form of a reinforced concrete face**.** The reinforced concrete face is interfaced with the grout curtain by arrangement of a concrete gallery in the 1st stage dam upper part. The movement joint is arranged between the reinforced concrete face and the gallery.

The thickness of the reinforced concrete face in the lower part was taken equal 2 m, and in the upper part 1m. The thickness of the grout curtain was taken equal 30 m.

The purpose of our study is assessment of workability, safety of this structural design alternative of an ultrahigh embankment dam.

2. Methods

The study was conducted with the aid of numerical modeling of the dam stress-strain state (SSS) with use of the computer program developed by Ph.Dr.(Tech.Sc) M.P. Sainov [14]. Analyses were conducted with consideration of the schedule of dam construction sequence and external forces applied to it. Besides, it was taken into account that the grout curtain in the 1st stage dam appears only after completion of its construction.

The structure finite element model included 1074 finite elements with cubic approximation of displacements inside the element. Contact finite elements were used for modeling non-linear effects of rigid structures contacts behavior among each other, as well as with soils. The total number of degrees of freedom in finite element model comprised 9979.

At analyses consideration was taken of non-linear character of rockfill deformation. For this purpose the used soil model was proposed by Dr. L.N. Rasskazov [15]. At determining parameters of the non-linear model the use was made of the data of experimental studies conducted by Marsal, Marachi and Gupta [16–18]. As the data analysis of field observations over construction settlements of real dams entails large studies of rockfill deformation properties [19-20], our investigations were carried out for a wide range of rockfill deformability. Three alternatives of rockfill deformation properties were considered in the dam upper part: A, B, C. In alternative B the rockfill deformation was adopted to be 2 times as less as in alternative A, and in alternative C – 4 times as less. The averaged rockfill deformation modulus in each alternative was approximately 45, 90 and 180 MPa. At the completion construction stage averaged values of rockfill deformation modulus of the upstream shell reached the following values: for alternative A – 90 MPa, for alternative B – 200 MPa and 300 MPa for alternative C. From the downstream part of the dam body the rockfill deformation in all calculations was taken as in alternative A.

For the grout curtain the elastic material was taken, because deformation and strength properties of soils strengthened by injecting cement-containing grouts, actually have not been studied. Evidently, they may vary in wide ranges, because injection may be accomplished with grouts of different composition and properties. Pure cement or cement-clay grouts may be used [21]. For tentative assessment of the grouted soil properties we used the data on properties of clay-cement-concretes, which greatly vary depending on the content of cement and bentonite.

Our analyses were conducted for three alternatives of grouted soil properties. In alternative 1 the grouted soil deformation modulus was taken equal 5000 MPa, in alternative 2 – 1000 MPa, in alternative 3–200 MPa.

Accordingly, 9 alternatives were analyzed. They are designated by a combination of figures and letters, for example, 3B. The figure indicates the alternative of the grouted soil properties and a letter indicated the alternative of rockfill properties.

3. Results and Discussion

The results of analyses are given in Figures 2–8 and in Table 1, 2 for the most critical moment of time – the moment of the reservoir impoundment to FSL 230 m. Let us see how the selected factors affect the workability of each component of the dam seepage control element.

SSS of the grout curtain is characterized by bend deformations occurring due to the dam body displacement towards the downstream side. Maximum displacement of the curtain towards the downstream side for alternatives A is equal from 47 to 64 cm, for alternatives B – from 33 to 39 cm, for alternatives C – 20 cm (Table 1). The curtain bend is accompanied by decrease of compression on its upstream face and increase of compression on the downstream face.

Table 1. Typical parameters of the grout curtain SSS for different alternatives

SSS parameter	**alternatives**								
	1A	1B	1C	2A	2B	2C	3A	3B	3C
max u_x, cm	47	33	20	55	37	20	64	39	20
max u_y, cm	14	11	10	29	21	16	56	37	28
max σ_y, MPa	1.7	1.3	1.0	1.2	0.8	0.6	1.0	0.6	0.2
min σ_y, MPa	-31.1	-23.1	-16.8	-15.3	-11.0	-7.4	-4.9	-3.4	-2.6
L, m	18.9	15.7	4.8	11.5	5.7	0.6	1.2	1.2	0.6
P, mm	63.6	58.2	18.9	93.6	57.7	14.0	95.0	80.0	32.3

Designations:

L, P – the length of opening and maximum opening of the contact between the curtain and the foundation respectively,

u_x, u_y – horizontal displacement and settlement of the grout curtain downstream face respectively.

At grout curtain bending there is a danger of crack formation on its upstream face, especially in the near-contact zone. This evidences about the existing zone of tensile vertical normal stresses σ_y (Figure 2, Table 1), as well as about opening of the grout contact with rock foundation (Table 1). These manifestations are typical for all the considered alternatives in different degrees.

If the grout material is comparable by its deformability with rockfill (alternatives of series 3, Figures 2a, b, c), the zone of tension is small by dimensions, and the values of tensile stresses σ_y are not large. The length of the curtain opening contact with rock does not exceed 2 m (Figure 4).

If the grout material is rigid (alternatives of series 1, Figures 2g, h, i), tensile stresses σ_y cover large volume and will inevitably result in formation of cracks in the grout curtain. At that, on the curtain downstream face (Figures 2 g, h, i) high compressive stresses σ_y are concentrated (from 17 to 30 MPa), therefore, compressive strength failure may be expected. Besides, the length of the curtain opening contact with rock will amount from 15 to 60 % of its width (Figure 4).

Judging by Figures 3a, b, the grout curtain may have the acceptable level of compressive and tensile stresses σ_y only in case when the deformation modulus of the curtain material does not exceed 500 MPa, and averaged value of deformation modulus of rockfill (at perception of hydrostatic forces) is not less than 200 MPa. We came to the similar conclusion earlier for the case of the dam with massive grout curtain [22].

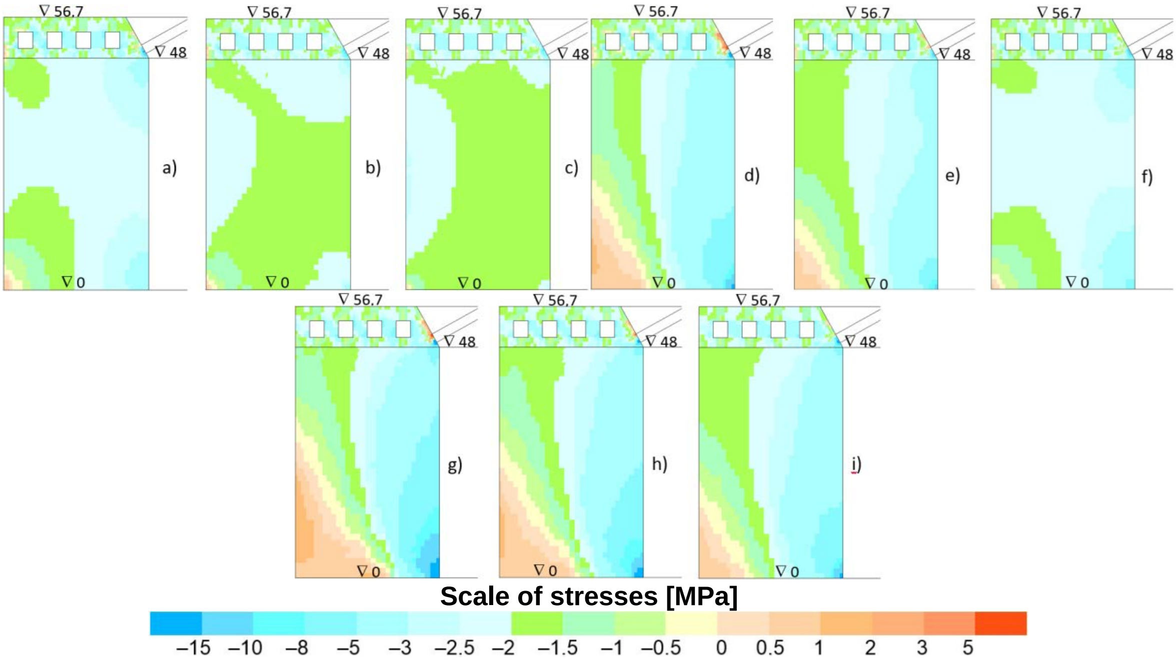


Figure 2. Distribution of stresses σ_y in grout curtain for different alternatives
a – 3A, b – 3B, c – 3C, d – 2A, e – 2B, f – 2C, g – 1A, h – 1B, i –1C

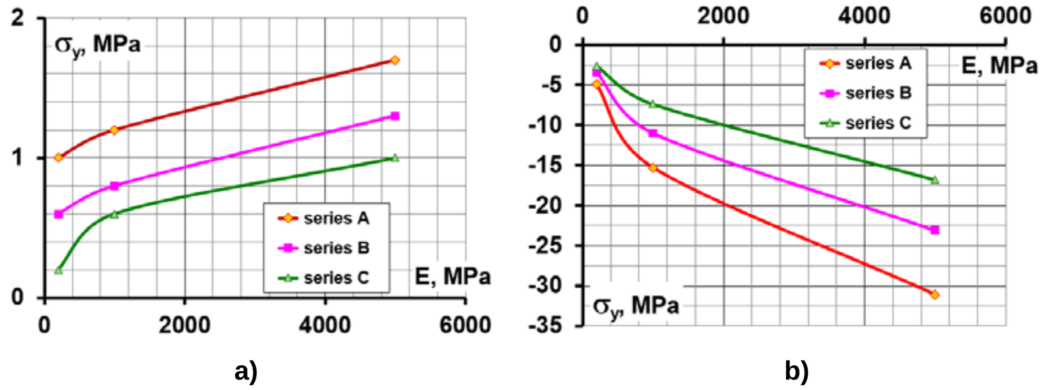


Figure 3. Variation of maximum stresses σ_y in the grout curtain depending on deformation modulus of its material a – tensile, b – compressive

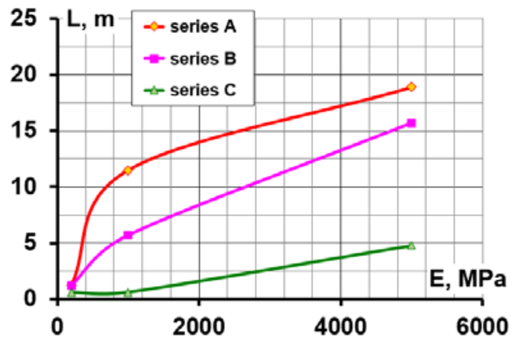


Figure 4. Variation of length of the contact "curtain-rock" depending on deformation modulus of the grout curtain material

Now let us consider SSS of the reinforced concrete face shown in Figures 5–7 and Table 2. Maximum displacement of the reinforced concrete face in direction across the slope is observed on the 2[nd] stage dam crest. Accordingly the reinforced concrete face is subject to bend deformations towards the downstream side (Figure 5).

But in spite of bend deformations the most part of the reinforced concrete face is subject to compression along the slope (Figures 6, 7). By this the SSS of the reinforced concrete face in the considered dam principally differs from that of rockfill dam classical design for which according to our studies [12] the characteristic feature is longitudinal tensile force existing in the reinforced concrete face. The same effect is characteristic for the dam alternative where the reinforced concrete face is combined with clay-cement-concrete diaphragm [23].

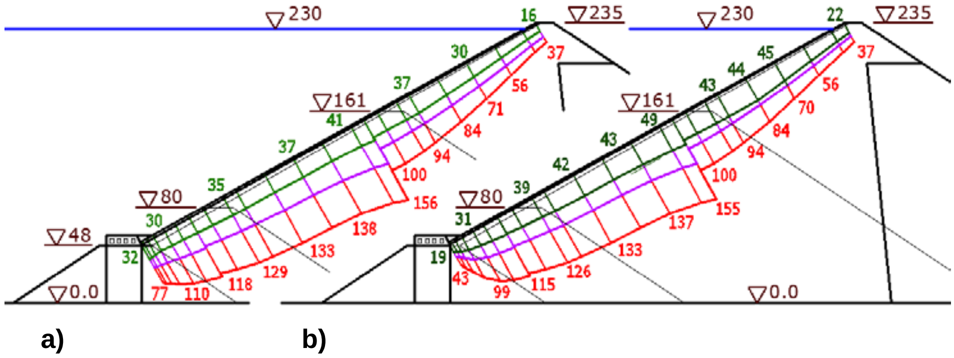
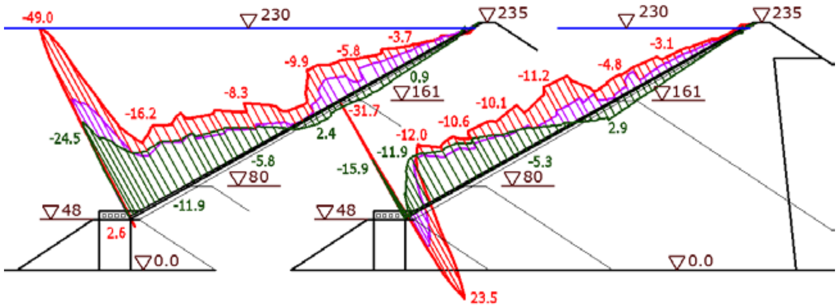
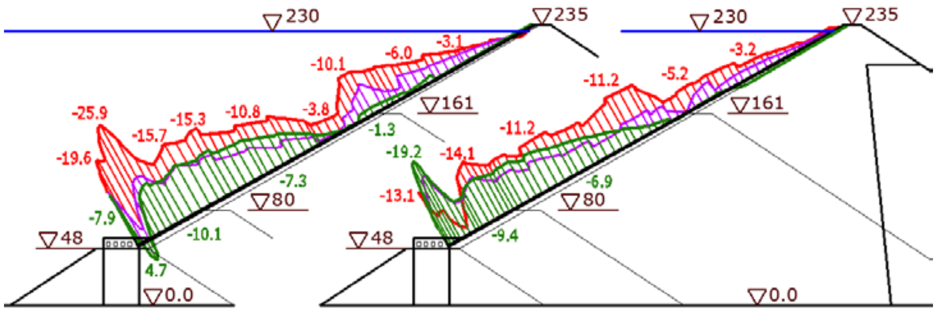


Figure 5. Deflections of the reinforced concrete face (cm) in various alternatives. a – alternatives of series 3, b – alternatives of series 1. Red color indicates the curve corresponding to alternatives of group A, violet color– group B, green – group C.



a) b)

Figure 6. Longitudinal stresses in the reinforced concrete face (MPa) in alternatives of group 1. a – on the upstream face, b – on the downstream face. Red color indicates the curve corresponding to alternative 1A, violet color– 1B, green – 1C.



a) b)

Figure 7. Longitudinal stresses in the reinforced concrete face (MPa) in alternatives of group 3. a – on the upstream face, b – on the downstream face. Red color indicates the curve corresponding to alternative 3A, violet color– 3B, green – 3C.

SSS of the reinforced concrete face upper part is different for all the alternatives and is determined by rockfill deformability. At high rockfill deformability (alternatives of series A) concrete strength of the reinforced concrete face upper part is not provided. Compressive longitudinal stresses σ_E in the reinforced concrete face exceed concrete design compressive strength of class B25 which according to Building code[1] amounts to 14.5 MPa.

SSS of the reinforced concrete face lower part is to a greater extent determined by the grout curtain deformation. If the grout curtain is made of rigid material (alternatives of series 1), the reinforced concrete face lower part is subject to great bend deformations towards the downstream side (Figure 5b). At that on the reinforced concrete face upstream part the compressive stresses are concentrated (Figure 6a), and the downstream part – tensile stresses (Figure 6a). In all the alternatives of series 1 the reinforced concrete compressive strength is not provided (Figure 8).

The reinforced concrete face lower part has more favorable SSS in the alternatives of series 2 and 3.

In alternatives of series 3, when the grout curtain material has low deformation modulus (200 MPa), the reinforced concrete face lower part is subject to bend towards the downstream side (Figure 5a). This results in increase of compression on the face downstream part (Figure 7a) and its decrease on the upstream part (Figure 7b). In alternative 3C SSS of the reinforced concrete face is unfavorable. On the upstream face the longitudinal stress σ_E is tensile. It reaches 4.7 MPa and cannot be perceived by reinforcement. Maximum value of compressive strength σ_E on the downstream face reaches 19.2 MPa and exceeds concrete compressive strength. Thus, deformability of the grout curtain material should not be lower than rockfill deformability.

Thus, effect of deformability of rockfill and the grout curtain material on SSS is of complicated character and they are interconnected. The most favorable SSS of the reinforced concrete face is in alternatives 2C and 3B. These are alternatives where the grout curtain material is close by deformability to that of rockfill, but is slightly more rigid as compared to it. However, even in these alternatives the maximum

[1] Building Code SP 41.13330.2012. Concrete and reinforced concrete constructions of hydraulic structures. Updated version of SNiP 2.06.08-87.

value of compressive stresses σ_E in the reinforced concrete face slightly exceeds concrete design strength of class B25 (Table 2). It is necessary either to use concrete with higher strength or increase the reinforced concrete face thickness.

Table 2. Typical parameters of the reinforced concrete face SSS for various alternatives

parameter	alternatives								
	1A	1B	1C	2A	2B	2C	3A	3B	3C
U_{max}, cm	155	87	49	156	87	42	156	87	41
U_1, cm	43	27	19	53	36	24	77	49	32
min σ_E, MPa	-49.2	-31.0	-25.0	-42.6	-21.8	-16.4	-25.8	-15.6	-19.2
max σ_E, MPa	23.6	8.5	0.4	14.4	-	-	-	-	4.7

Symbols: U_{max} – maximum deflection of the reinforced concrete face,

U_1 – deflection of the reinforced concrete face at the contact with the concrete gallery,

min σ_E – most significant by value longitudinal compressive stresses,

max σ_E – most significant by value longitudinal tensile stresses.

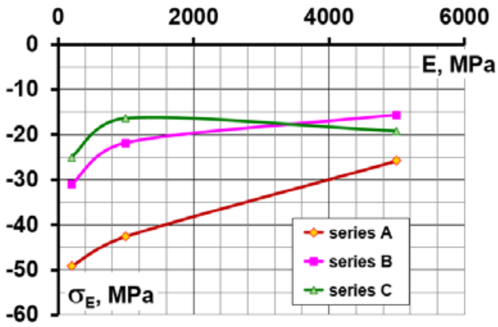


Figure 8. Variation of maximum values of reinforced concrete face depending on deformation modulus of the grout curtain material

4. Conclusions

1. The structural alternative of a rockfill dam where the seepage-control element is composed of a reinforced concrete face and a grout curtain is a good alternative to a classical structural design of a rockfill dam with a reinforced concrete face. The considered dam structural design is potentially efficient: at meeting certain conditions it may operate safely.

2. In the considered dam structural design the reinforced concrete face operates in more favorable conditions then in classical structural design of a rockfill dam with reinforced concrete face (CFRD). The face is subject to not tensile but compressive longitudinal force. To provide compressive strength of the reinforced concrete face it is necessary either to decrease rockfill deformability (rockfill deformation modulus should be tentatively at least 150 MPa), or increase its thickness. It is recommended to adopt the reinforced concrete face thickness in the lower part to be no less than 2 m.

3. To prevent crack formation in the grout curtain body and at the contact “curtain-rock” it is necessary that deformation modulus of the injected soil should not exceed 500 MPa. For grouting it is recommended to use bentonite-cement but not cement mortars.

4. SSS of the reinforced concrete face lower part greatly depends on deformations of the grout curtain located under the face, because the curtain high rigidity constricts the face movements and leads to its bending. To avoid considerable bend deformations of the reinforced concrete face lower part the deformation modulus of the grout curtain material should be not lower than that of rockfill.

5. Complexity in predicting workability of the considered dam structural design is attributed to insufficient knowledge of deformability of rockfill and soils strengthened by injecting grouts containing cement. Due to non-linearity of these material deformations the character of the reinforced concrete face deformations may change in the nature during dam construction and operation causing alternately compressive and tensile stresses.

6. To provide safe operation of combined seepage-control element it is necessary to think through the design of the contact between the reinforced concrete face and the concrete gallery located under the

grout curtain. Structural design of this interface should provide possibility of compensation of the reinforced concrete face displacements in direction along the slope, but considerable displacements in the perimeter joint are not allowed.

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