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Stress-strain state of seepage-control walls in foundations of embankment dams

Напряжённо-деформированное состояние противофильтрационных стен в основании грунтовых плотин

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Ключевые слова: стена в грунте; напряжённо-деформированное состояние; глиноцементобетон; численное моделирование; прочность

Abstract. Results are considered of systematic study of stress-strain state (SSS) of a seepage-control wall arranged in the embankment dam foundation. The following factors affecting the wall SSS were studied: the wall depth, deformation modulus of the wall material, the pattern of the wall resting. Studies were conducted with the aid of numerical modeling. They revealed significant role of friction processes and slippage at the contact with soil on the wall SSS formation. It is friction through which soil transfers to the wall the compressive longitudinal forces which increase while the wall stiffness increases. It was also revealed that conditions of operation of suspended walls are more favorable that those of walls resting on rock. Empirical formulae were proposed which permit predicting the value of maximum compressive forces in the wall. Compressive strength of the wall was assessed. At that, it was taken into account that strength of plastic clay-cement concrete considerably increases if it is compressed from all sides as compared to uniaxial compression. A considerable role of accounting this effect was shown at selecting material for arrangement of the wall. It was obtained that to provide the wall strength it is necessary to have its material deformability exceeding the deformability of the surrounding soil not more than 5 fold. It was revealed that at perceiving by the wall the horizontal forces of seepage or hydrostatic pressure the longitudinal forces in it sharply decrease. It was obtained that then there is a danger of the wall tensile stress failure, because bend deformations in the wall cause irregular distribution of stresses in it. Especially it is hazardous for the walls made of rigid materials.

Аннотация. Рассматриваются результаты методического исследования напряжённо-деформированного состояния (НДС) противофильтрационной стены, устроенной в основании грунтовой плотины. Было исследовано влияние на НДС стены следующих факторов: глубина стены, модуль деформации материала стены, схема опирания стены. Исследования проводились путём численного моделирования. Они выявили значительную роль на формирование НДС стены процессов трения и проскальзывания на контакте с грунтом. Именно через трение грунт передаёт на стену значительные сжимающие продольные усилия, которые тем больше, чем больше жёсткость материала стены. Также выявлено, что условия работы висячих стен более благоприятные, чем у стен, опёртых на скалу. Предложены эмпирические формулы, которые позволяют спрогнозировать величину максимальных сжимающих напряжений в стене. Выполнена оценка прочности стены на сжатие. При этом учитывалось, что прочность пластичного глиноцементобетона существенно возрастает по сравнению с одноосным сжатием, если он сжат со всех сторон. Показана существенная роль учёта этого эффекта при выборе материала для устройства стены. Получено, что для обеспечения прочности стены необходимо, чтобы деформируемость её материала не более чем в 5 раз превышала деформируемость окружающего грунта. Обнаружено, что при восприятии стеной горизонтальных сил фильтрационного или гидростатического давления, продольные усилия в ней резко уменьшаются. Получено, что при этом возникает опасность нарушения прочности стены на растяжение, т.к. изгибные деформации стене вызывают в ней неравномерное распределение напряжений. Особенно это опасно для стен, которые выполнены из жёстких материалов.

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Introduction

As it is known, very often deep curtains are arranged by a cutoff wall method for seepage control in foundations of embankment dams. The advantage of this type of curtains as compared to injections is the fact that they reliably intersect water permeable layer. At that, arrangement of the walls is cost-saving, because they have small thickness (0.6÷1.2 m). If the wall is made of materials based on cement (concrete, clay-cement concrete), in compliance with the existing standards[1] it may withstand considerable head gradient (more than 100). Due to this seepage-control walls may perceive great heads. At use of modern technologies their depth may reach 135 m as at Peribonka dam in Canada [1]. Therefore, the cutoff wall methodology at present is widely used in hydraulic construction against seepage. The examples may be dams Karkhe [2], Xialongdi [3], Kureika [4], Yumaguza [5], Sangtuda [6], Peribonka [1], Merowe [7], Sylvenstein [8] and others.

In order to provide safe operation of seepage-control walls (SCW) it is necessary to provide their crack growth resistance and strength. Therefore, to assess strength at SCW designing it is required to assess their stress-strain state (SSS).

By present time the issue of SCW SSS has not been sufficiently studied. SSS analyses of wall made of non-soil materials were conducted in Moscow Civil Engineering University by a number of authors
[9–12]. However, in most cases they referred to walls arranged in dam bodies.

Recently there appeared papers about SSS of SCW arranged in dam foundations [13–16]. A number of papers devoted to SSS of SCW have been also prepared by us [17–21]. It was established that walls in embankment dam foundations may subject to considerable compressive and tensile longitudinal stresses. We revealed that these stresses are more the more is the ratio between deformation moduli of the wall material and foundation soil. The results of our studies showed that to provide the wall strength it is necessary to have its material deformability exceeding the deformability of the surrounding soil not more than 5 fold which complies with ICOLD recommendations [22].

However, these studies were not of systematic character; their results refer only to particular considered conditions. This paper describes the results of more full systematic study of SSS of seepage-control walls arranged in foundations of embankment dams. The purpose of this work is to reveal operation conditions and peculiar features of CSW SSS in foundation of an embankment dam as well as to verify recommendations for selection of material for SCW. These studies also permit assessing the effect on SSS of such factors as material rigidity of the wall, its depth, and conditions of rest.

In the studies there considered a seepage-control wall made in the uniform non-soil foundation of 100 m high embankment dam. It is a rock-earthfill dam with a core. The dam shells are filled with gravel-pebble soil; the core is made of loam. The wall is arranged along the core axis (Fig.1). The scheme of the wall and core conjugation (with the aid of a cantilever or a concrete gallery) was not considered not to complicate the analysis of SCW operation conditions. The wall does not propagate into the dam core.

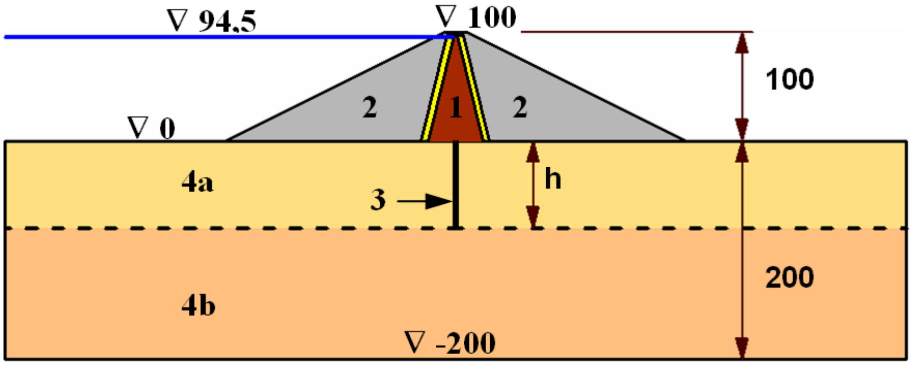

Figure 1. Design diagram of the wall in foundation of an embankment dam:
1 – loam core, 2 – shells of gravel-pebble, 3 – seepage-control wall, 4a, 4b – foundation layers

Two alternatives of foundation structure were considered. The first alternative is a homogenous structure of the earth foundation. In the second alternative the upper layer which is cut out by a wall, consists of earth and the lower layer refers to rock.

[1] Building Code SP 23.13330.2011. Foundations of hydraulic structures. Updated version of SNiP 2.02.02-85.

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Foundation soils were taken to be linearly deformable at active loading and unloading. Deformation modulus of the earth where the wall was arranged at active loading was taken equal 100 MPa and at unloading 500 MPa. This earth refers to gravel-pebble-sand soil. The earth Poisson's number was taken equal 0.35. Earth friction coefficient ($f = tg\ \varphi$) along the wall was taken equal 0.78 ($\varphi = 38°$); specific cohesion was absent.

Methods

Studies were conducted with the aid of numerical modeling by finite element method (FEM). Computations were conducted with use of software worked out by Dr. Ph. (Tech) M.P. Sainov [23].

Finite-element model of the structure covered the dam body and the foundation block. It was sufficiently detailed and comprised 1157 finite elements (Fig. 2). Width-wise the wall 3 rows of finite elements were distinguished. Among the finite elements 38 were contact elements; they simulated non-linear behavior at interaction of foundation soils with the wall. Besides, contact elements simulated possibility of occurrence of shear cracks in the soil mass above the wall top and the foot of the wall. At creation of a finite-element model the high-order finite elements were used with cubic approximation of displacements inside the element. The number of the model degrees of freedom comprised 11208.

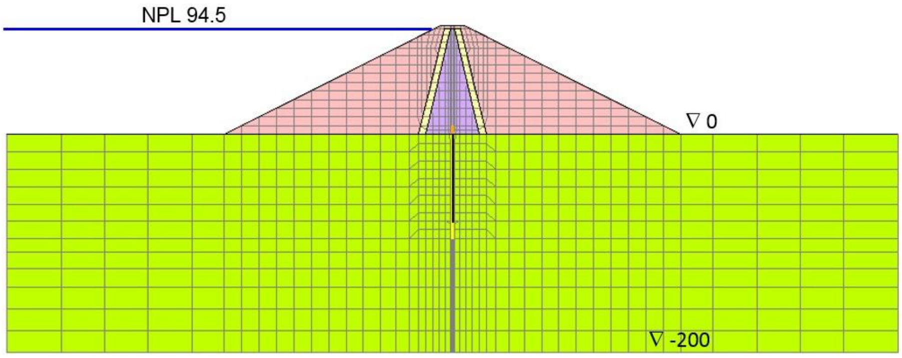


Figure 2. Diagram of FEM computational mesh

Studies were conducted for 6 design diagrams. The wall depth (20, 40 or 80 m) and conditions of its rest (suspended or resting). The resting wall was deepened into a rock layer with deformation modulus 5000 MPa; i.e. conditions of embedding were modeled. Parameters of design diagrams are given in Table 1.

Table 1. Parameters of design diagrams

No. of alternative	IA	IB	IC	IIA	IIB	IIC
Wall depth, m	80	40	20	80	40	20
Thickness of earth foundation under the wall, m	120	160	180	0	0	0
Conditions of rest	suspended			resting		

Rock on which the wall rested in the design was taken as waterproof. Therefore, in the alternatives of series II the resting wall took hydrostatic pressure of the upstream and downstream sides. In the alternatives of series I the suspended wall worked on taking loads from streamlining the wall by seepage flow. Therefore, to determine seepage loads in alternatives IA, IB, IC the seepage task was solved (Fig. 3). Analysis shows that suspended walls perceive horizontal forces 2–3 times as less as compared to resting walls.

For each of the design diagrams several alternatives of the wall materials were considered: from liquid plastic clay-cement concrete to reinforced concrete (Table 2).

Table 2. Parameters of wall material alternatives

Alternative	density, t/m^3	deformation modulus [MPa]	Poisson's ratio	Uniaxial compression strength [MPa]	Angle of internal friction
1	1.93	100	0.30	1.27	30°
2	1.98	500	0.30	2.13	32°
3	2.10	1000	0.30	2.66	35°
4	2.12	5000	0.25	4.45	40°
5	2.40	29000	0.18	11.5	

These parameters were determined by us based on the results of experimental studies of clay-cement concrete properties conducted by other authors [5, 24–26]. For concrete, uniaxial compression strength (11.5 MPa) was adopted in compliance with Building Code SP 23.13330.2011.

32 design steps were considered. The first and the second step modeled the foundation SSS before the dam construction. The next step simulated the process of the wall arrangement by the trench method: trench excavation for the wall was modeled and its filling by the wall material. It was assumed that the wall material was not in the hardened state at perceiving its own weight. Its deformation modulus was taken equal 20% of the final one and Poisson's ratio was 0.45. At this step it was taken into account that non-hardened wall material could freely slip with respect to the trench walls.

At the next 16 steps there was model the process of the embankment layered construction. The next 13 steps modeled the process of the reservoir filling and the process of forming seepage (or hydrostatic) forces in the foundation. The task on stabilized seepage regime was solved for determining seepage forces (Fig. 3).

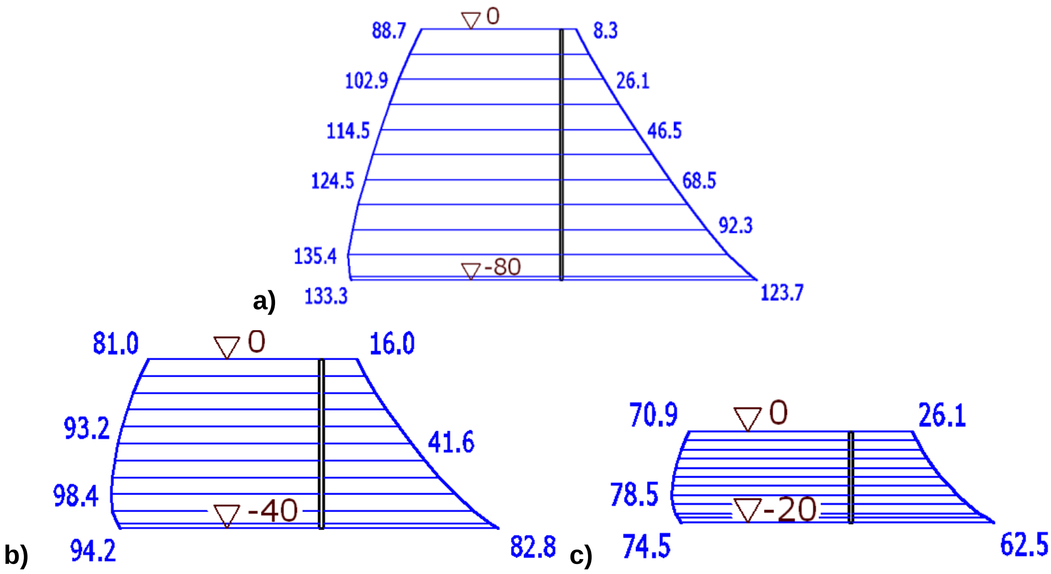


Figure 3. Seepage loads on the wall (in meters of water column):
a – at wall depth 80m, b – at wall depth 40m, c – at wall depth 20 m

Results and Discussion

Analysis of the results of SCW SSS computations was conducted for two most dangerous moments of time:

1) moment of the dam construction completion,

2) moment of the reservoir filling completion.

As it was shown earlier [21], the first moment is dangerous from the point of view of hazardous failure of compression strength. At this moment the wall perceives maximum vertical load and has maximum compressive stresses. The wall vertical load appears due to settlements of the surrounding earth mass under the action of the dam weight. This load is transferred to the wall by the foundation soil through friction on the contact “soil-wall”.

The second moment of time is dangerous from the point of view of possible appearance of tensile stresses in the wall. Tension in the wall may appear due to bend deformations, which the wall acquires at perceiving water pressure horizontal forces.

The results of analyses are given in Figures 4–15, 18–29 for some design alternatives in the form of curves of stresses and displacements. The curves of stresses given in the figures do not take into account stresses in the wall from its own weight.

Analysis of the results of SCW SSS computations for the moment of the dam construction completion shows the following:

- If the wall is made of rigid material, at the foundation settlements the considerable compressive longitudinal stresses are transferred to the wall. Due to this compressive stresses are concentrated in it.
- The more the wall material differs by deformability from the foundation soil, the higher are vertical compressive stresses y in it.
- For the alternatives where SCW is made of rigid materials (No. 3, No. 4, No. 5), there develops the typical process of the soil slip with respect to the wall due to the contact shear strength failure. The slip processes occur mainly in the upper part of the contact “wall-soil”.
- Due to this maximum compression is observed in the wall lower part.

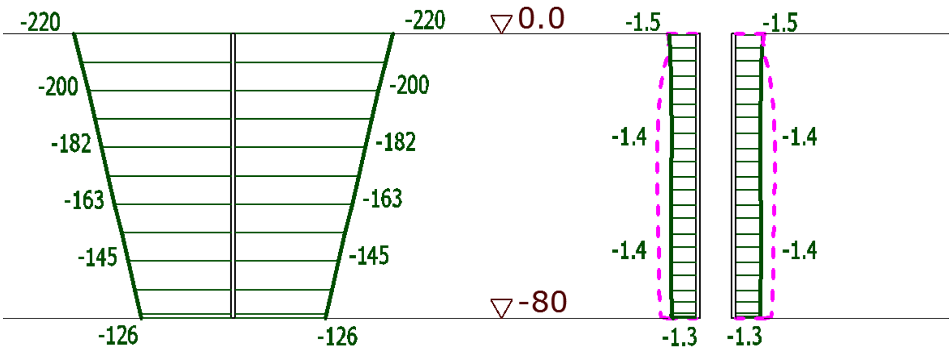


Figure 4. Wall SSS of alternative IA-1 for the moment of the dam construction completion: a – settlements (cm), b – vertical stresses (MPa) on the upstream and downstream faces

Green lines correspond to the curves for the wall. Pink-violet lines correspond to soil settlements. The dotted line indicates the wall material compressive strength.

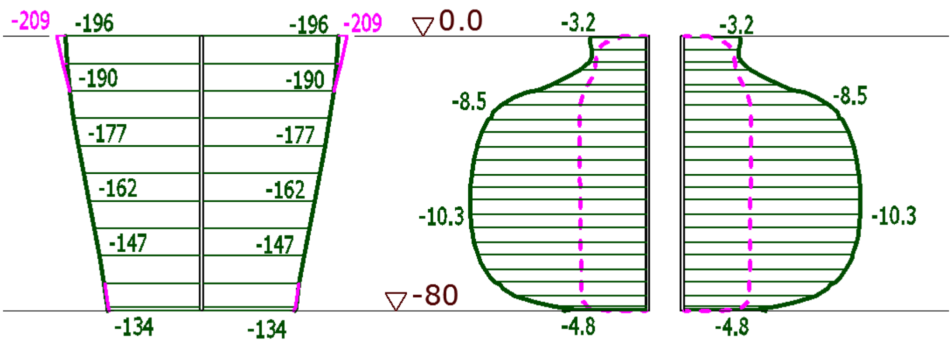
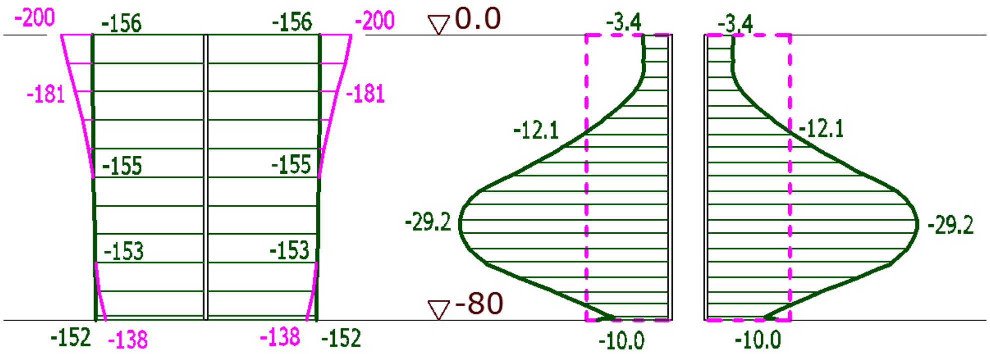
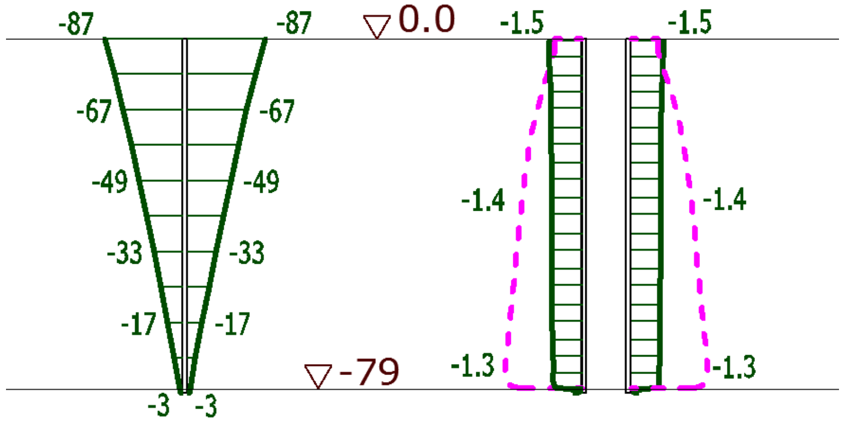


Figure 5. Wall SSS in alternative IA-3 for the moment of the dam construction completion. Legend see at Figure 4



a) b)

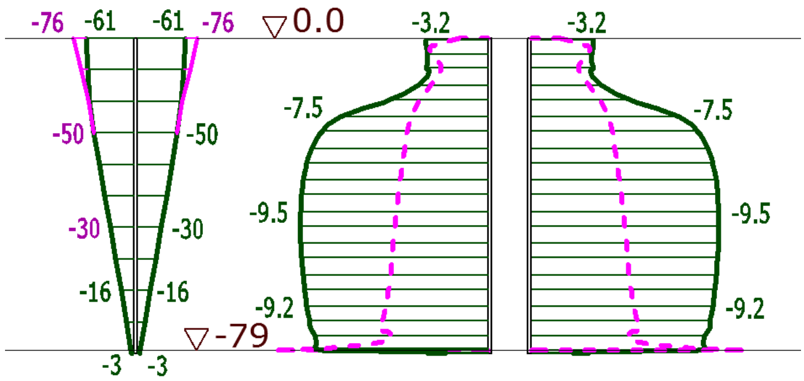
Figure 6. Wall SSS in alternative IA-5 for the moment of the dam construction completion. Legend see at Figure 4



a) b)

Fig.7. Wall SSS in alternative IIA-1 for the moment of the dam construction completion: a – settlements (cm), b – vertical stresses (MPa) on the upstream and downstream faces

Green lines correspond to the curves for the wall. Pink-violet lines correspond to soil settlements. The dotted line indicates the wall material compressive strength.



a) b)

Figure 8. Wall SSS in alternative IIA-3 for the moment of the dam construction completion. Legend see at Figure 7

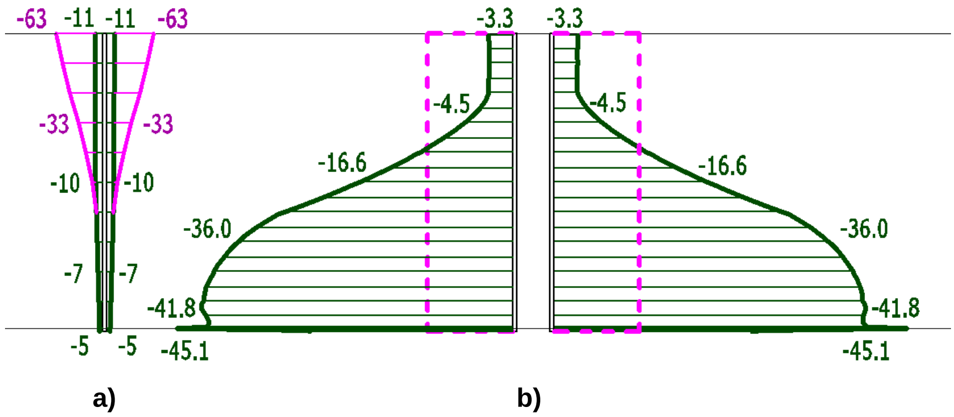


Figure 9. Wall SSS in alternative IIA-5 for the moment of the dam construction completion. Legend see at Figure 7

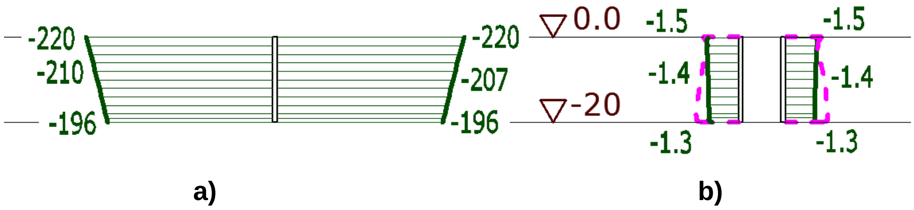


Figure 10. Wall SSS in alternative IC-1 for the moment of the dam construction completion: a – settlements (cm), b – vertical stresses (MPa) on the upstream and downstream faces

Green lines correspond to the curves for the wall. Pink-violet lines correspond to soil settlements. The dotted line indicates the wall material compressive strength.

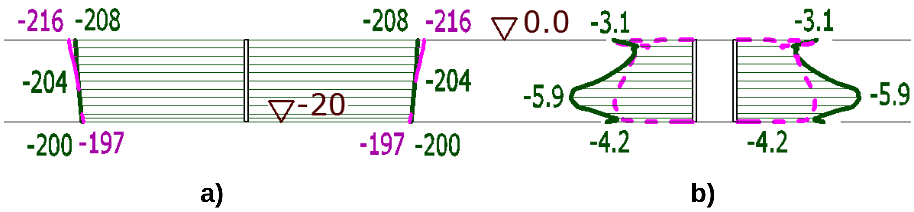


Figure 11. Wall SSS in alternative IC-3 for the moment of the dam construction completion. Legend see at Figure 10

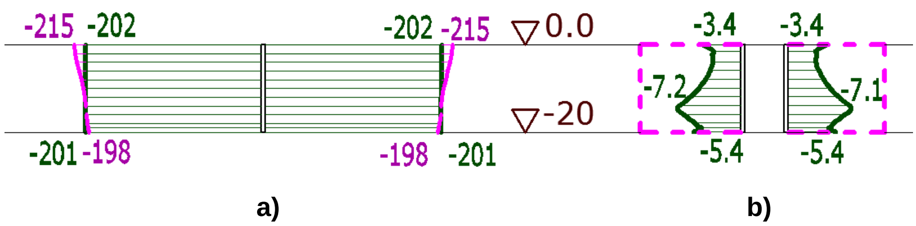


Figure 12. Wall SSS in alternative IC-5 for the moment of the dam construction completion. Legend see at Figure 10

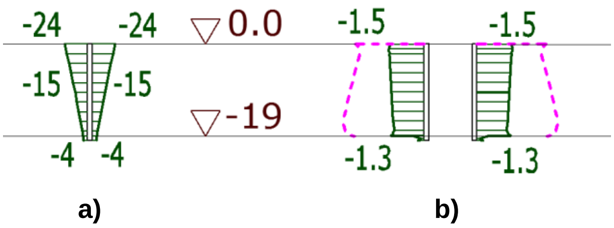


Figure 13. Wall SSS in alternative IIC-1 for the moment of the dam construction completion. Legend see at Figure 10

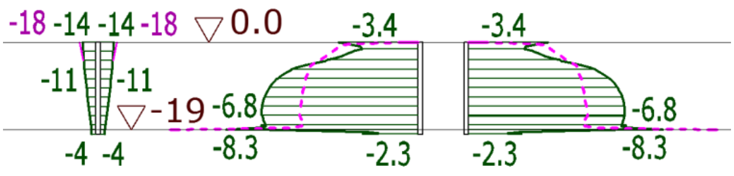


Figure 14. Wall SSS in alternative IIC-3 for the moment of the dam construction completion. Legend see at Figure 10

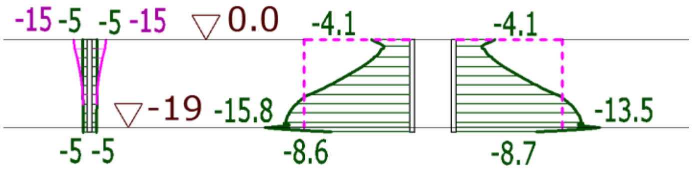


Figure 15. Wall SSS in alternative IIC-5 for the moment of the dam construction completion. Legend see at Figure 10

For each design diagram there were drawn variation curves of maximum stresses σ_y^{max} depending on the ratio between deformation modulus E_{wall} and foundation deformation modulus E_{found} (Fig. 16).

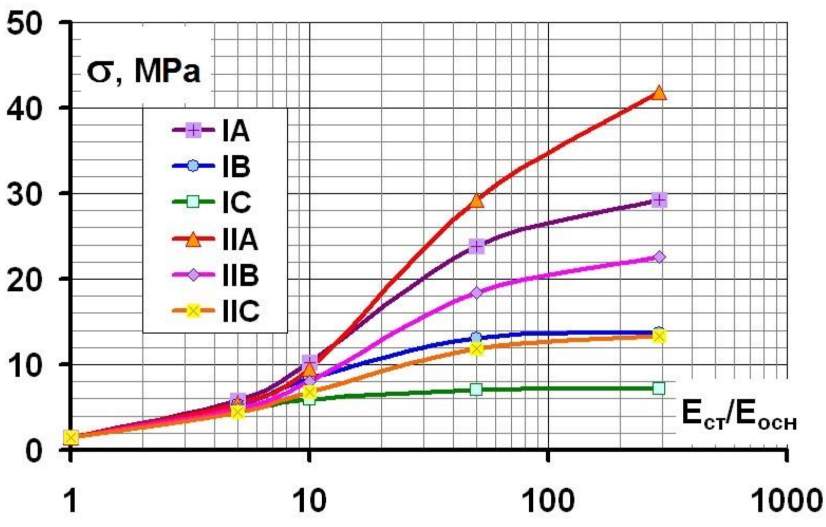


Figure 16. Variation of maximum values of longitudinal stresses in the wall depending on ratio E_{wall}/E_{found}

It is possible to distinguish two typical sections in this relationship. The first section is characterized by smooth increase of compressive stresses in the wall as the ratio E_{wall}/E_{found} increase. It is realized at E_{wall}/E_{found}. not exceeding 20. These conditions are characterized by weak development of slip processes at contact «wall-soil». Slip occurs only in the upper part of the contact; its shear strength is minimum.

At the first section the relationship between E_{wall}/E_{found} may be rather strictly described by the following relationship:

$$\sigma_y^{max} = A \cdot p \cdot \left(\frac{E_{ст}}{E_{осн}} \right)^n, \tag{1}$$

where p – pressure transferred from the dam to the foundation,

A, n – empirical values.

By the results of analysis the pressure p amounted to approximately 1.5 MPa.

One of the empirical relationship parameters is always A≈1. The obtained values of index n are shown in Table 3.

Table 3. Values of index n

Design diagram	IA	IB	IC	IIA	IIB	IIC
n	0.84	0.78	0.70	0.76	0.73	0.66

Analysis of the obtained graphs (Fig. 16) shows that at the first section of relationship E_{wall}/E_{found} is characteristic the following:

- In the resting walls the maximum stress values y grow slightly less intensively than in suspended walls. This is explained by increase of contact “wall-soil” length where slip processes develop;
- Stress maximum values y have small dependence on the wall depth, though less deep walls have slightly less values of stress.

The study permitted revealing that the value of index n depends on the wall depth and conditions of the wall performance (suspended or resting). Dependence of n from the wall depth H may be described by relationship:

$$n = B\,H^{m} \qquad (2)$$

Empirical index B is within limits $0.46 \div 0.48$, and $m \approx 0.12 \div 0.13$ depending on boundary conditions of the wall operation conditions.

At the second section (at $E_{wall}/E_{found} > 10 \div 20$) the possibility of further increase of stresses in the wall is limited by intensive development of slip processes at contact “wall-foundation”. The more is the length of the contact where shear strength fails, the higher are the values of stresses σ_y^{max} in the wall. Failure of the contact shear strength develops mainly from the wall upper end downward. Suspended walls have the contact strength failure near the lower end also.

Due to presence of slip there is limit value of maximum compressive stresses which may be transferred to the wall.

Analysis of the results shows that:

- The deeper is the wall, the higher is limiting compressive stress in it. For example, in diagram IA-5 (H=80 m) they comprised 29.2 MPa (Fig. 6b), and in diagram IC-5 (H = 20 m) – 7.2 MPa (Fig. 12b);
- In the walls resting on rock foundation the limit value of compressive stress is higher that in suspended walls. For example, in diagram IA-5 limit stresses amounted to 29.2 MPa (Fig. 6b), and in diagram IIA-5 – 41.8 MPa (Fig. 9b).

Thus, resting walls of large depth have more favorable SSS.

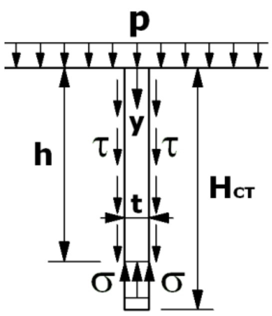


Figure 17. Diagram for the wall analysis

In [27] for the simplified design diagram (Fig. 17) we obtained an analytical dependence for determining the limit value:

$$\sigma_y^{пред} = p + \frac{h}{t}\left[2p\lambda\,\mathrm{tg}\,\varphi + 2c + \gamma\lambda\,\mathrm{tg}\,\varphi\,h\right], \qquad (3)$$

where h – length of the contact section, where shear strength failed,

t – wall thickness,

φ, c – angle of internal friction and specific cohesion at contact “wall-soil” respectively,

γ, λ – specific weight and coefficient of soil lateral pressure respectively.

Formula (3) shows that strength parameters of contact “soil-wall” considerably affect the limiting state of compressive stresses. The stronger is the contact the more are compressive stresses in the wall.

Dependence (3) also permits assessing the effect of depth on values of compressive stresses. Theoretically it is quadratic, however, the results of SSS analysis for the considered case show that the role of the quadratic term is small and stresses are actually increased linearly with growth of the wall depth.

This dependence also indicates on the method of decreasing compressive stresses in the wall, i.e. increase of the wall thickness.

However, analytical dependence (3) is approximate; it does not allow considering the effect of the foundation SSS peculiar features. The true values of compressive stresses in the wall may be obtained only by numerical modeling.

The obtained SCW SSS permitted us to assess the wall strength and reveal what material is the best for arranging SCW.

At assessing strength of the wall made of plastic material (for example, clay-cement concretes) we proposed to take into consideration the fact that SCW is in a complicated stress state. Experimental studies with clay-cement concrete show that its compressive strength increases when we have lateral compression [24–25]. This effect may be considered based on the theory of strength of Coulomb–Mohr. In compliance with this theory the material compressive strength proportionally increases with lateral compression growth:

$$R = R_1 + \sigma_1 \frac{1+\sin\varphi}{1-\sin\varphi}, \tag{4}$$

where R_1 – uniaxial compression strength,

σ_1 – compressive stress (maximum principal stress),

φ – angle of soil internal friction.

The wall compression is achieved by lateral pressure of the surrounding soil.

Analysis of meeting compressive strength conditions conducted for various alternatives showed the following:

- Consideration of lateral compression considerably improves the wall condition from the point of view of assessing compressive strength. Compression effect is especially noticed in the alternatives with liquid clay-cement concrete (alternative No. 1);
- In most alternatives compressive strength is not provided, even with consideration of strength growth at lateral compression;
- In most alternatives with liquid clay-cement concrete (Alternative No. 1) the most dangerous section is the wall top. In this section compressive stresses have maximum values and compressive strength values are minimum. It should be noted that SSS of foundation itself significantly affects SCW performance. By the results of analyses the foundation under the action of dam weight «sprawls». This results in the fact that the wall upper part turns to be weakly compressed by soil lateral pressure and the wall operation conditions are close to uniaxial compression state;
- In the alternatives of making SCW of rigid materials (alternatives Nos. 3–5) the most hazardous section is located in the wall lower part where compressive stresses are maximum;
- Operation conditions of suspended walls are more favorable, because compressive stresses in them are less and therefore, it is easier to provide the material strength;
- The only wall material whose strength was provided in any patterns and operation conditions is liquid clay-cement concrete (alternative No. 1). At that, safety factor in the wall upper part as a rule is close to 0;
- In operation mode of wall IC (suspended wall 20 m deep) (alternatives C) compressive strength was provided not only in alternative No.1 but also at arrangement of a concrete wall (alternative No. 5). This was achieved due to actually full slip of the foundation soil against a not deep wall.

This analysis shows that SCW operation conditions are very complicated; they depend on many factors, that is why it is not possible to formulate general recommendations for selection of material and providing SCW strength, which could be applicable to all possible SCW operation conditions. Depending on loads transferred by the dam to the foundation, conditions of resting, the wall thickness and depth, foundation structures the SCW operation conditions may differ greatly. In each particular case it is necessary to conduct numerical studies of SCW SSS.

Tentative recommendations may be as follows:

- For arrangement of the wall it is desirable to use the material whose deformability differs from that of the surrounding soil by not more than 2 times. This recommendation differs from ICOLD recommendations [22] and makes the requirements tougher;
- For improvement of SSS and strength condition of the SCW upper part it is desirable to exclude direct transfer of the dam weight loads to it. For this purpose a concrete gallery may be provided above the wall separated from it by a gap (cavity);
- One more way of improving the strength state of SCW made of rigid materials is increase of its thickness. Increasing thickness of the wall will permit decreasing concentration of compressive stresses in it;
- In some cases the use of more rigid and strong material, reinforced concrete may be allowed for SCW arrangement and the required strength will be provided. This is possible in conditions if compressive forces transferred to the wall are not great. For example, this is possible due to slip at contact “soil-wall”. Therefore, of great importance are strength indices at contact “soil-wall”. With this respect it is necessary to note that at the wall construction by a trench method a so-called bentonite “casing” is formed with low strength indices. “Oiling” may greatly affect the rigid wall SSS.

The wall SSS analysis on the moment of perception by the wall of the seepage flow pressure shows cardinal change of SSS. At reservoir filling the wall shifted toward the downstream side and acquired bending deformations (Figs. 18–29b). Maximum shift is observed in the wall head.

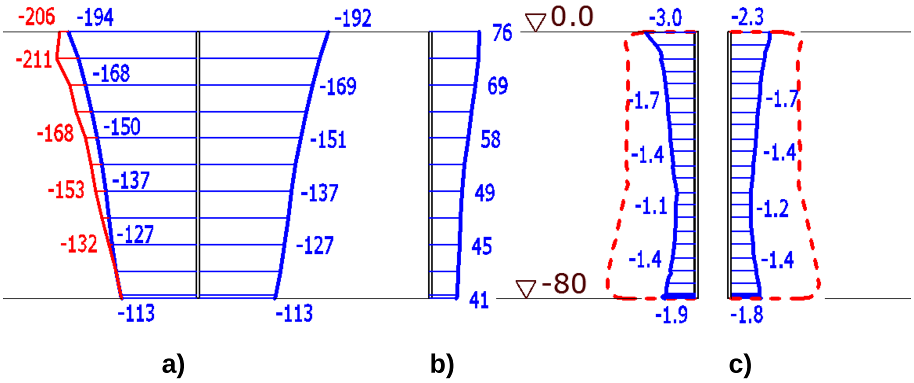


**Figure 18. Wall SSS of alternative IA-1 as of the moment of reservoir filling completion:
a – settlements (cm), b – displacements (cm),
c– vertical stresses (MPa) on the upstream and downstream faces**

Blue lines correspond to the curves for the wall. Red lines correspond to soil settlements. The dotted line indicates the wall material compressive strength.

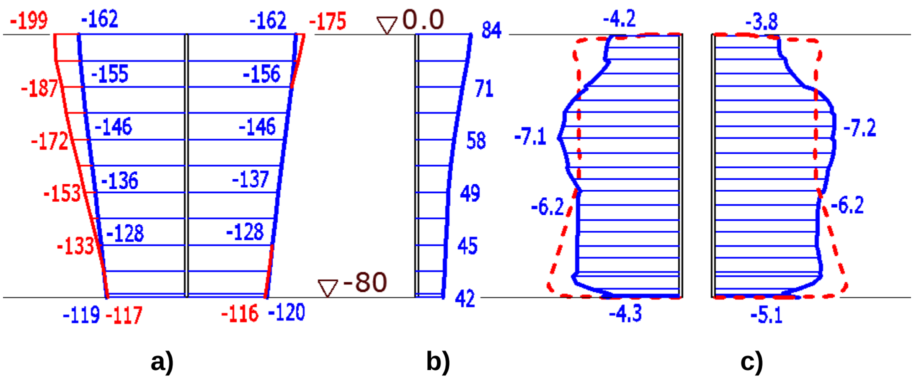


**Figure 19. Wall SSS of alternative IA-3 as of the moment of reservoir filling completion.
Legend see in Figure 18**

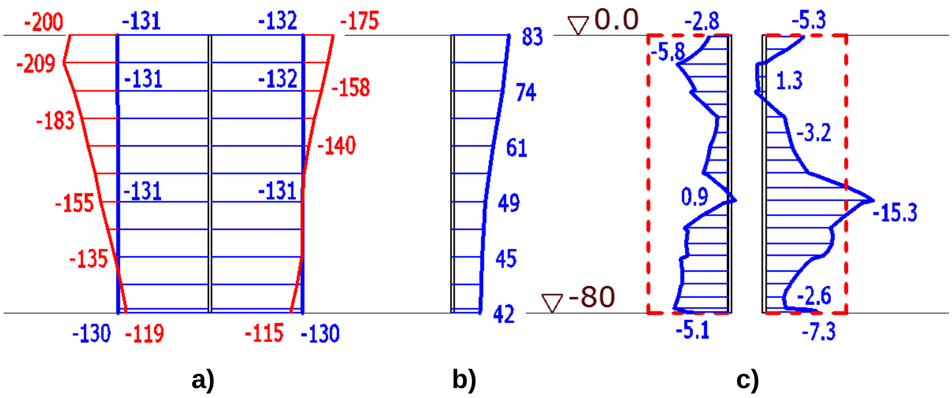


Figure 20. Wall SSS of alternative IA-5 as of the moment of reservoir filling completion. Legend see in Figure 18

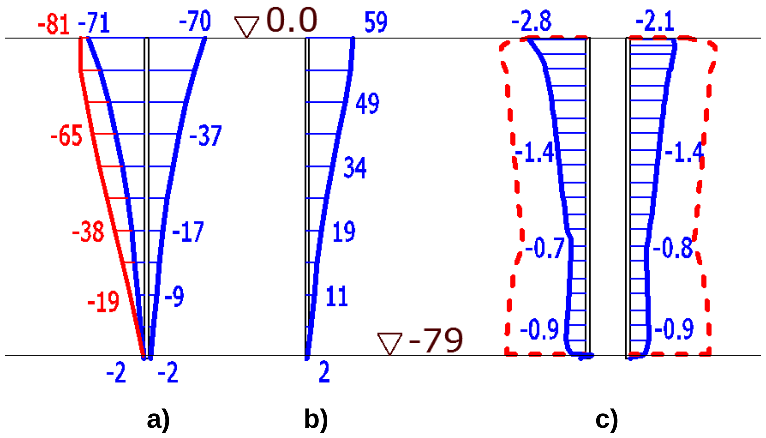


Figure 21. Wall SSS of alternative IIA-1 as of the moment of reservoir filling completion: a – settlements (cm), b – displacements (cm), c– vertical stresses (MPa) on the upstream and downstream faces

Blue lines correspond to the curves for the wall. Red lines correspond to soil settlements. The dotted line indicates the wall material compressive strength.

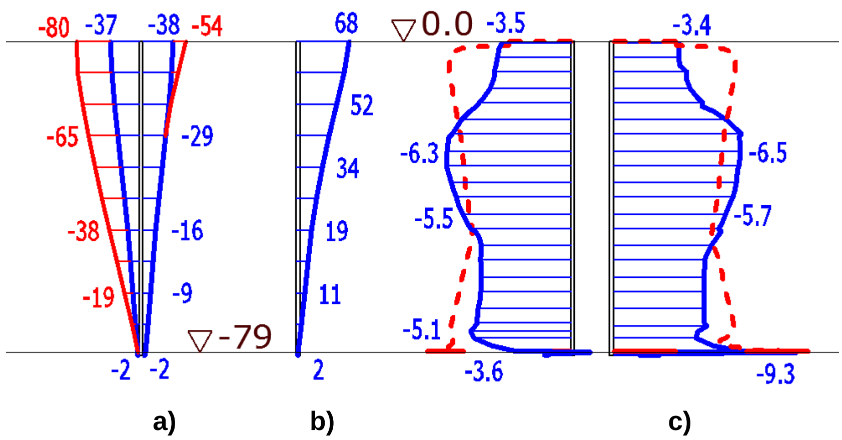
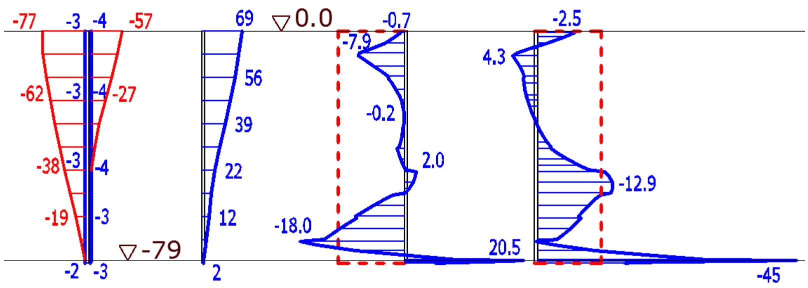
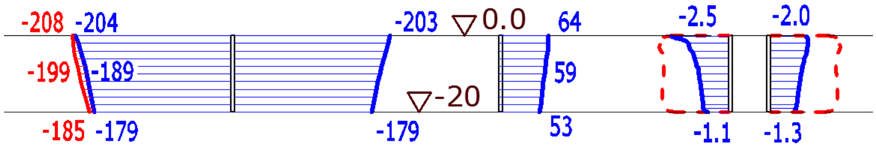


Figure 22. Wall SSS of alternative IIA-3 as of the moment of reservoir filling completion. Legend see in Figure 21



a) b) c)

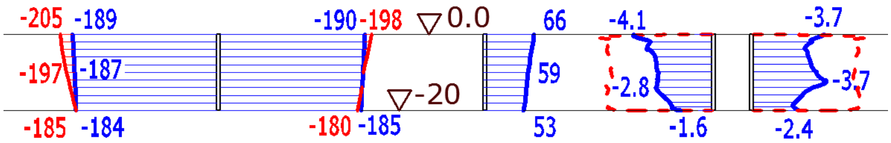
Figure 23. Wall SSS of alternative IIA-5 as of the moment of reservoir filling completion. Legend see in Figure 21



a) b) c)

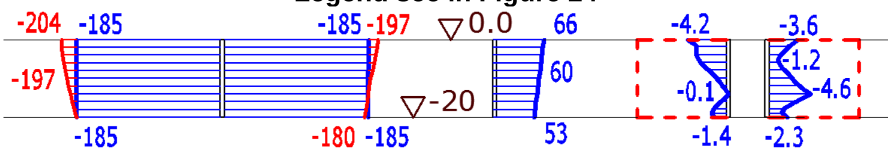
Figure 24. Wall SSS of alternative IC-1 as of the moment of reservoir filling completion: a – settlements (cm), b – displacements (cm), c– vertical stresses (MPa) on the upstream and downstream faces

Blue lines correspond to the curves for the wall. Red lines correspond to soil settlements. The dotted line indicates the wall material compressive strength.



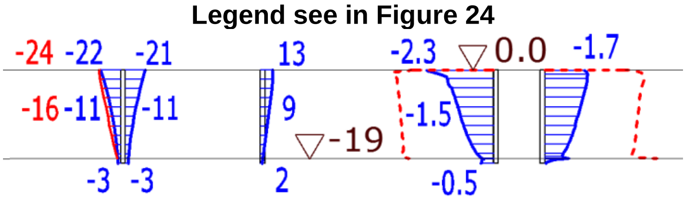
a) b) c)

Figure 25. Wall SSS of alternative IC-3 as of the moment of reservoir filling completion. Legend see in Figure 24



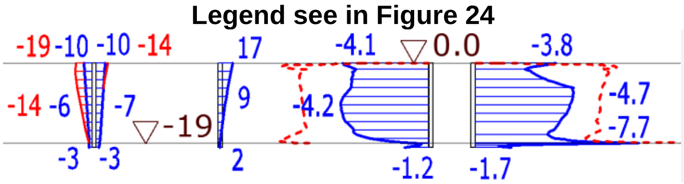
a) b) c)

Figure 26. Wall SSS of alternative IC-5 as of the moment of reservoir filling completion. Legend see in Figure 24



a) b) c)

Figure 27. Wall SSS of alternative IIC-1 as of the moment of reservoir filling completion. Legend see in Figure 24



a) b) c)

Figure 28. Wall SSS of alternative IIC-3 as of the moment of reservoir filling completion. Legend see in Figure 24

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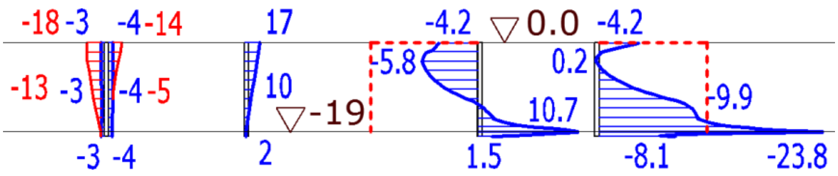

Figure 29. Wall SSS of alternative IC-5 as of the moment of reservoir filling completion. Legend see in Figure 24

Analysis of SCW SSS modeling shows the following:

- At perception of horizontal forces not only displacements occur but also the wall bend. At that, suspended SCW and resting walls displace and bend in different ways. Foot displacement of the walls resting on rock foundation is close to 0, but the wall top displacement is more than that of suspended walls. For example, the top displacement of the resting wall in alternative IIA-1 comprises 59 cm (Fig. 20b), and of the suspended wall in alternative IA-1 it is 76 cm (Fig. 20b). At that, bend of the suspended wall in alternative IIA-1 (57 cm) is much less than that of the resting wall in alternative IA-1 (35 cm). This difference in bends is explained by the fact that the resting walls take greater horizontal loads than the suspended ones;
- At perception of horizontal loads by the wall the vertical compressive forces perceived by it also considerably decreased. This is explained not only by the fact that weighing action contributes to decrease of pressure transferred to the foundation from the dam weight. Of great importance is development of slip processes along the upstream face of SCW (Figs. 17–28a). At the wall displacements toward the upstream side the normal stresses in the upstream contact "wall-soil" decrease, and consequently, its shear strength decreases also. Due to development of slip processes in the upstream contact "wall-soil" the vertical compressive forces transferred to the wall sharply decrease.
- Bend of the suspended wall is complicated. The upper part bends toward the downstream side and the lower part bends toward the upstream side. Due to the bend on one of the faces compressive stresses y decrease and on the other face they increase.
- For the walls made of clay-cement concrete there is hardly noticed non-uniformity in stress distribution. When the wall is made of reinforced concrete (alternative No. 5) compressive stresses pass to tensile stresses. For example, in alternative IA-5 tensile stresses reach 1.3 MPa (Fig. 19c) and exceed the concrete tensile strength.
- In the walls on rock foundations due to more displacements the bend deformations are expressed stronger. Due to the fact that in alternative IIA-5 the tensile stresses in the wall upper part reach 4.3 MPa (Fig. 22c). Besides, the resting wall character of bend deformations is complicated in the zone of embedment into the rock foundation. In the embedment zone the wall considerably bends toward the upstream side. In alternative IA-5 tensile stresses on the upstream face exceed 20 MPa (Fig. 22c).
- SSS of less deep walls is somewhat better. In alternative IIC-5 tensile stresses in the wall upper part do not exceed 0.2 MPa, and in the zone of embedment they amount to 10.7 MPa (Fig. 28c). Thus, for the walls embedded into a rigid rock foundation the most hazardous are tensile stresses in the embedment zone. We may arrive to the conclusion that the interface of SCW with rock should be provided not by its embedment into rock, but by arranging a more flexible connection. For example, it may be an earlier prepared zone of grouted impervious soil.

From the point of view of providing the dam compressive strength the wall strength state in the second design moment is more favorable than in the first one. This is explained by two reasons. The first reason refers to general decrease of compressive longitudinal forces in the wall. The second reason is attributed to the fact that at reservoir filling the wall is compressed under water pressure and due to this its material compressive strength increases.

Wall compressive strength is not provided only in use of rigid materials (alternatives Nos. 4–5, sometimes alternative No. 3).

Conclusion

1. SCW SSS analysis should be performed at least for two moments of time: The first is the moment of dam construction completion; the second refers to the moment of reservoir filling completion and formation of the seepage regime in the foundation. The first moment is dangerous from the point of

view of possible compressive strength failure, the second from the point of view of possible tensile strength failure.

SCW SSS in the foundation of a high embankment dam as of the moment of dam construction completion is characterized by concentration of considerable compressive longitudinal stresses. These stresses are greater the greater is the ratio between deformation moduli of the wall material and the foundation soil. After the reservoir filling the compressive stresses decrease, but this leads to possible appearance of tensile stresses in the wall. In the moment of reservoir filling completion the most hazardous for the wall are bend deformations.

2. Rigidity of the wall itself and conditions of interaction with surrounding soil mass have the greatest impact on formation of SCW SSS. The degree of developed slip processes at contact “soil-wall” plays a great role.

Besides, SCW SSS is affected by conditions of its rest (suspended or resting), foundation setting, the wall depth and thickness.

3. SSS of walls embedded into a rock foundation (resting walls) is less favorable than that of the suspended walls. At perception of vertical forces the resting wall maximum values of stresses turn to be slightly higher than those in suspended walls. Walls reaching water-tight stratum are subject to greater horizontal water pressure and therefore, greater displacements. They are characterized by greater bend deformations, especially in the zone of embedment into rock foundation. The zone of walls embedment into rock foundation is the most dangerous and unsafe section. At horizontal displacements of walls in the embedment, cracks may appear as a result of tensile strength failure as well as shear.

4. The empirical dependences proposed by us may be applied at the preliminary design stage for determining maximum values of compressive longitudinal stresses in SCW. However, these dependences are approximate. More reliable results may be obtained only by numerical modeling, because this is the only way of modeling slip processes at the contact of the wall with soil.

5. Deep walls operate in more complicated conditions than the wall of small deepness. The greater is the wall depth the higher are compressive stresses in it and the higher is the probability of compressive strength failure.

6. At assessing compressive strength of the walls made of plastic material, for example clay-cement concrete, it is necessary to take into account the effect of strength growth when we have lateral compression. As SCW are usually in the state of triaxial compression, account of this effect is of great importance.

7. For SCW constructed in a homogenous foundation it may be recommended to use the material whose deformation modulus is not more than 2 fold as compared to the deformation modulus of the surrounding soil. This meets ICOLD recommendations, but the difference is that it is tougher.

8. It is not recommended to use concrete and reinforced concrete for SCW arrangement. At foundation settlements considerable compressive stresses are concentrated in rigid walls, and at perception of horizontal forces the tensile stresses appear. It is especially dangerous to use concrete (reinforced concrete) for the walls arranged in soil and embedded into rock foundation.

However, there are cases when use of reinforced concrete may be allowed and justified. For example, reinforced concrete may be used for arranging suspended walls of small deepness.

9. At designing seepage-control walls there are several ways of their SSS regulation to provide strength. The first is using the material by deformability close to the enclosing soil. The second way is increasing the wall thickness. The third way is decrease of friction at contact “soil-wall” to decrease forces transferred to the wall by foundation soil, however, this way is not possible in practice. Besides, tangible effect may be reached by arranging a concrete gallery above the wall top, from which the wall will be separated by a gap. This will permit slight decrease of vertical compressive forces transferred to the wall.

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