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# Results of computational studies of stress-strain state of locks No. 15 and No. 16 of Gorodetsky complex of hydraulic structures

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**Abstract.** The article provides the results of computational studies of the stress-strain state of reinforced concrete structures chambers of locks No. 15 and No. 16 of the Gorodetsky complex of hydraulic structures, performed on the basis of a finite element model taking into account the data of long-term monitoring and field surveys. Combinations of loads that are pleasant in computational studies correspond to the most unfavorable operating conditions. The applied finite element models take into account the structural features of the structure, including interlocking construction joints and the actual location of the reinforcement. In the computational studies, the load-bearing capacity of the actual anchoring of the new repair concrete and the concrete of the main array was checked; the distribution of vertical stresses in the chamber walls, in the reinforcement of the chamber wall, the values of the main stresses in the concrete block under the construction joint was obtained. Conclusions are drawn about the values of horizontal displacements of the upper parts of the chamber walls, cracking in the walls, and the need for reliable anchoring of new repair concrete to the concrete of the main array.

## 1 Introduction

The article presents the results of calculations of the stress-strain state of the lock chambers walls based on finite element models, taking into account the data of monitoring, field surveys (including underwater surveys), taking into account the actual condition of structures, as well as previously carried out repairs. Calculations were performed by numerical methods based on finite element models of the system "structures - surrounding soil array – foundation".

The results of the work have a high degree of importance since calculations of the existing stress-strain state of reinforced concrete structures allow us to make a conclusion about the condition of the structure that has been in operation for more than 50 years and

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make recommendations for improving the condition, carrying out the necessary repair work.

Gorodetsky complex of hydraulic structures is located on the Volga River near Gorodets, Nizhny Novgorod region, 850 km from the southern port of Moscow, according to the Atlas of the unified deep-water system of the European part of the Russian Federation. The lower complex of hydraulic structures is located in the left-bank floodplain of the Volga River, 442 km from the Rybinsk HPP, above Gorodets.

The shipping structures include two parallel single–chamber reinforced concrete locks of dock construction with a continuous bottom: the left is lock No. 15, and the right is lock No. 16. The upper and lower heads of locks No.15 and 16 are continuous reinforced concrete structures of the dock type. The chambers of both locks consist of 12 sections separated by a temperature-deformation joint. The upper and lower heads are separated by the same joints. The walls of the locks are cut with horizontal construction joints in height with a step of about 3 m.

On the side of the chamber, the walls are lined with shell plates 5-8 cm thick and 2.50 m high, in accordance with the initial project. The cladding consisted of four rows of plates. In 2012, a project was developed for the overhaul of locks, in which shell plates damaged during prolonged operation above 74.5 m are dismantled by cutting concrete to a depth of 250 mm and laying new concrete anchored to a depth of 220 mm. In the range of 64.50-67.55 m, the main concrete array was cut down to a depth of 500 mm, and new concrete was laid, anchored to a depth of 250 mm. The class of new concrete is B22.5 in accordance with the project documentation.

## 2 Materials and methods

Based on the analysis of load combinations during the operation and repair of the wall lock chambers, analysis of design materials, and the results of calculations performed earlier, two calculated combinations of loads corresponding to the most unfavorable condition of the chamber walls were adopted. At the same time, cases were considered when the load on the rear face of the wall is the greatest since the stresses in the reinforcement at the rear face and the displacement of the top of the walls towards the chamber is the most dangerous.

Taking into account the design schemes in the drawings of the project, for the case of a drained chamber, the highest groundwater level was taken, which is possible with rapid emptying of the chamber when the water level in the ground behind the wall drops much more slowly. Similarly, when the chamber was filled, which is possible with rapid filling, the lowest groundwater level was taken, which did not have time to rise after the water in the chamber.

When the chamber is filled, the water pressure on the front face of the wall largely compensates for the pressure of the ground outside. At a low groundwater level behind the wall, stretching in the reinforcement at the front face and displacement of the top of the wall towards backfill is possible. Therefore, the case of a filled chamber is not considered; however, consideration of this calculation case is advisable to take into account the transfer of water pressure through the filling ground of the inter-lock space to the wall of the adjacent drained chamber.

The first design case provides for a repair period in which both chambers are drained; the groundwater level is the highest. At the same time, the effect of winter temperature is assumed.

In the second calculation case, one of the chambers is drained at the highest groundwater level. The other chamber is filled with the transfer of water pressure on the first chamber through the filling ground of the inter-lock space. At the same time, it affects the lowest temperature, which corresponds to the calculated case.

The numerical modeling technique provides for finite element modeling of structures as part of the "structure - backfilling - foundation" system in a spatial formulation.

Finite element models take into account the structural features of structures, including the actual placement of reinforcement. In computational studies, horizontal and vertical construction joints are modeled, taking into account the decrease in strength under the combined action of shear with tension or compression compared to a monolithic structure. In the same way, a violation of the adhesion of reinforcement with concrete in the areas of opened joints and cracks is simulated. When the existing concrete of the walls is stripped, the working reinforcement of the front face is exposed.

Special approaches are used in modeling cracking at the joints and on monolithic concrete. At the same time, the conditions for the formation of cracks are the effect of tensile stresses exceeding the tensile strength of concrete and the strength of joints in a complex stressed state, that is, with the combined action of shear forces with tensile or compressive forces in the plane of the joints.

The design model reproduces both the design construction joints and the joints that are not presented in the working documentation caused by interruptions in the concreting of walls. The results of preliminary calculations showed that the presence of additional non-design concreting joints affects the stress values in the reinforcement of the rear face insignificantly.

In the model of the system "construction - backfilling - foundation", a fragment of the array of the foundation of the lock and the soils of backfilling behind the outer walls of the chambers, filling the inter-lock space between the adjacent inner walls of the two threads of the locks was reproduced.

The loads were set in accordance with the accepted design scheme: with the accepted levels of soil, groundwater, water in chambers, etc., as well as taking into account the physical and mechanical properties of structures materials, foundation soils, and backfills. At the same time, the software complexes used for computational studies make it possible to obtain loads on the lock structures from soil and water, taking into account the current level of the theory of soil mechanics.

The temperature effects were reproduced in the "structure - backfilling - foundation" models in accordance with the average monthly and average annual temperature for the area of hydraulic structures based on regulatory documents. The thermophysical properties of the materials were taken into account during the design studies.

The problem of temperature distribution within the computational area was previously solved. The initial calculated temperature were set on the contour of the calculated area, and then (taking into account the thermophysical properties of the elements of the area) the temperature values inside the area were calculated. In accordance with the obtained temperature distribution in each finite element, into which the calculated area is divided, the problem of determining the stress-strain state of the structure was solved.

One of the most important features of the approach to the modeling of structures, which distinguishes it from traditional approaches to ordinary engineering calculations taken in the design, is that it allows you to take into account various damages, defects, and other deviations from the prerequisites accepted in the design, by including them in the developed and corrected finite element models.

Figure 1 shows a view of the simulated design area, including gateways No. 15 and No. 16, the foundation, and the ground filling. Figure 2 shows a fragment of the lock chamber model.

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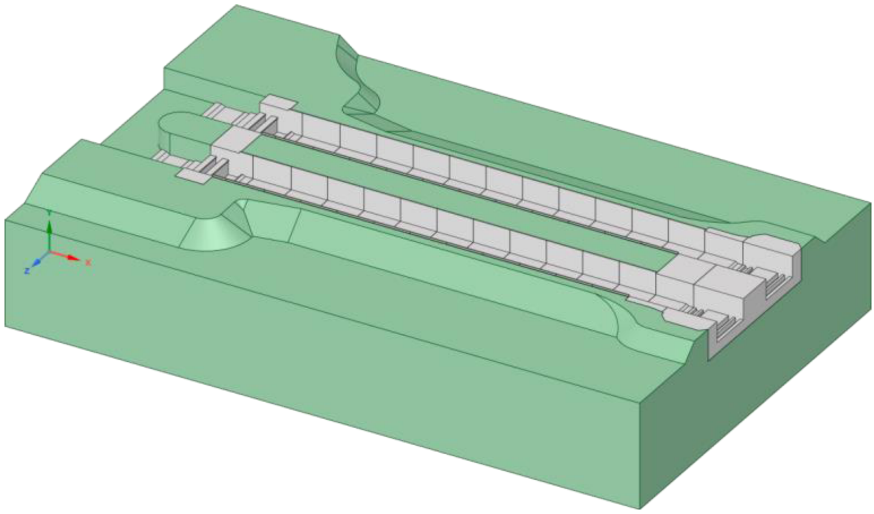

**Fig. 1.** View of the simulated computational area

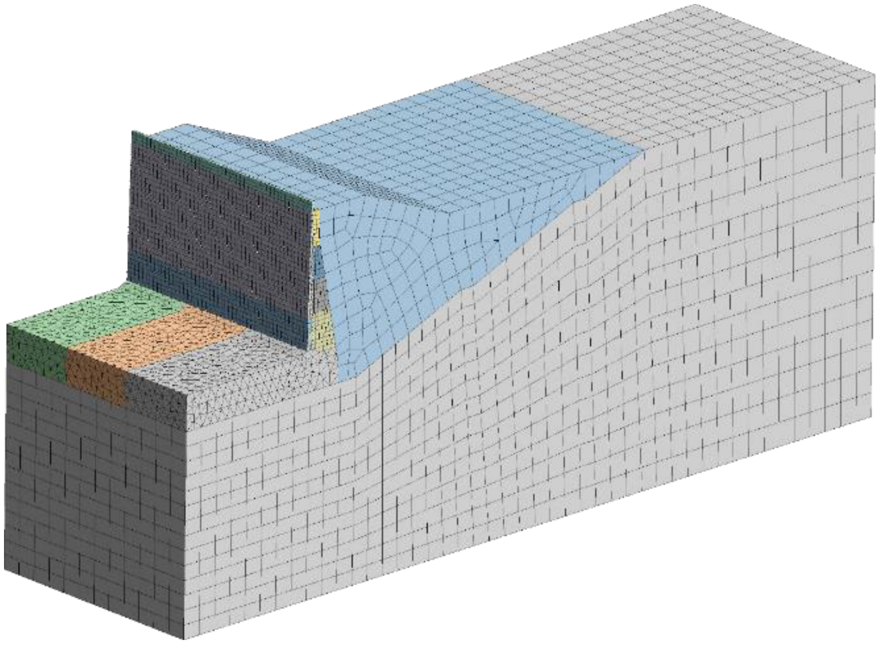

**Fig. 2.** Fragment of the finite element model of the section of the lock chamber No. 15

The finite element models take into account the actual condition of the lock chamber wall structures obtained as a result of field surveys, as well as during the analysis of monitoring.

# 3 RESULTS

In work, verification calculations of the stress-strain state of the lock chambers as part of the "construction - backfilling - foundation" system were performed, taking into account the actual state of the structures.

The computational studies were carried out by numerical methods based on finite element models in accordance with the computational schemes. As noted above, the problem of temperature distribution within the computational area was previously solved.

***Checking the bearing capacity of the actual anchoring***

The loads affecting the contact zone between the main concrete and the concrete of the repair zone are water back pressure in the construction joints and ice formed in winter, which also increases the width of the opening of the construction joint by increasing the volume in the crack by up to 10%. The results of checking the anchorage bearing capacity are presented in Table 1.

**Table 1.** Results of checking the anchoring bearing capacity

| Indicators | Calculation of anchoring above the 67.55 m | | Calculation of anchoring below the 67.55 m | |
|---|---|---|---|---|
| | The effect of ice expansion | Back pressure action | The effect of ice expansion | Back pressure action |
| 1. Expansion of the ice layer | 0.03 mm | - | 0.03 mm | - |
| 2. Relative elongation of the anchor reinforcement section taking into account slippage (ε) | 0.0012 | - | 0.0006 | - |
| 3. Maximum back pressure (p) | - | 8.95 t/m$^2$ | - | 12.00 t/m$^2$ |
| 4. Tensile stresses in anchor fittings ($\sigma_a$) | 252 MP<365 MP | 191 MP <365 MP | 126 MP <365 MP | 84 MP <365 MP |
| 5. Force in one anchor ($N_a$) | 0.0222 MN | 0.0216 MN | 0.1959 MN | 0.0412 MN |
| 6. Contact area in the well (Ø18 mm) "concrete of the main array" – "mortar on Portland cement around the anchor" | 0.01356 m$^2$ | 0.01356 m$^2$ | 0.0502 m$^2$ | 0.0502 m$^2$ |
| 7. Tangential stresses arising at the contact | 1.64 MP | 1.59 MP | 3.9 MP | 0.82 MP |
| 8. Strength of concrete during cutting ($R_{b,sh}$) | 0.71 MP | 0.71 MP | 0.71 MP | 0.71 MP |
| 9. Result: pulling strength of anchors | not provided | not provided | not provided | not provided |

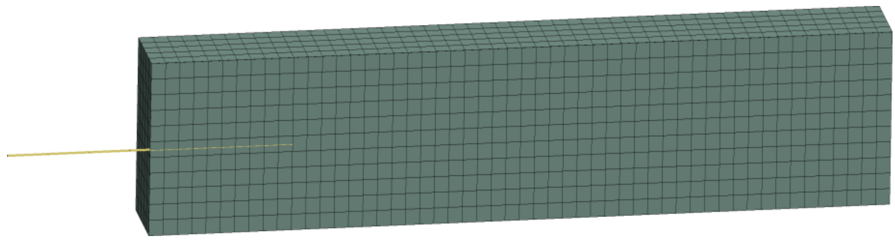

**Fig. 3.** Finite element anchoring model

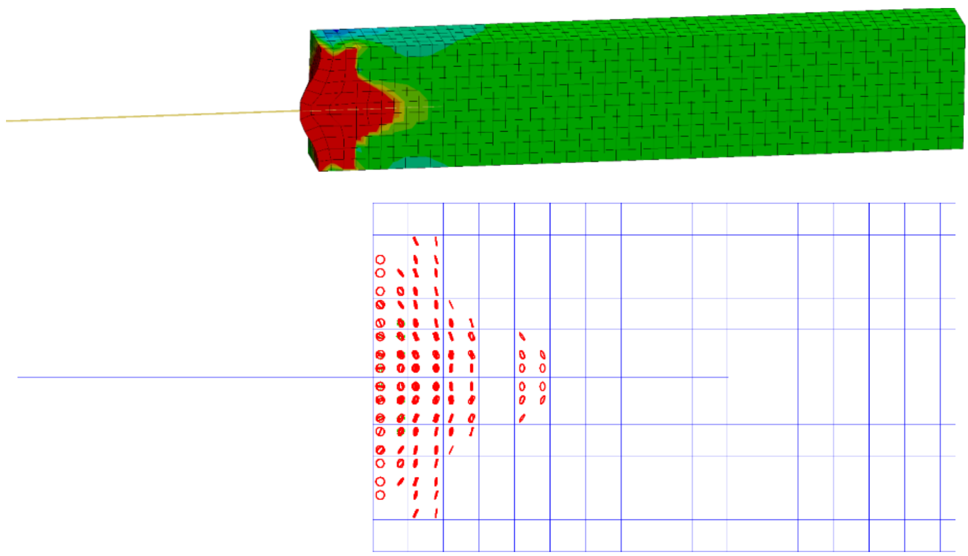

**Fig. 4.** Results of the calculation of the crack formation zone from the pulling load

A pulling load was applied to the anchor step by step. As can be seen from Figure 4, when the pulling load is below the bearing capacity of the anchor, which is made in the form of a 12 mm diameter reinforcement of class A-III, a network of cracks forms on the front face of the concrete of the main array.

Ice expansion in the vertical construction joint between the new concrete and the concrete of the main array leads to further development of cracks along the contact along the construction joint.

***Calculation results for combined loads 1 - both chambers are drained***

Figure 5 shows the distribution of horizontal displacements. As can be seen from Figure 5, the maximum horizontal displacements of the chamber top are directed inside the chamber and amount to 37.5 mm.

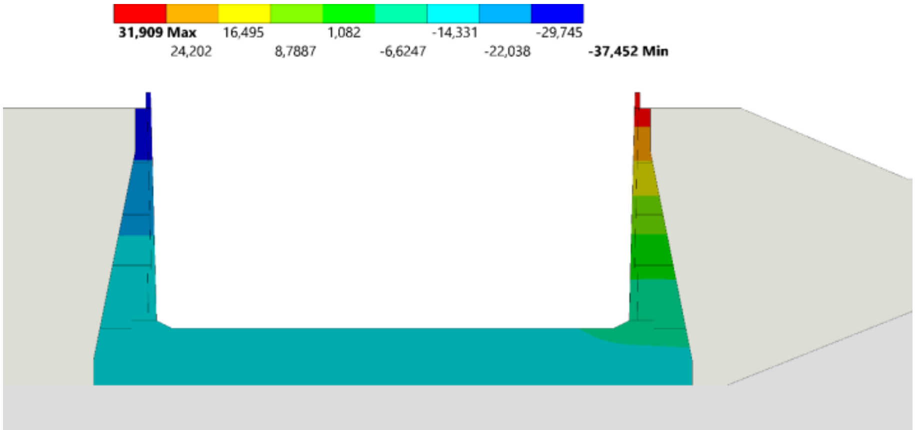


**Fig. 5.** Both chambers are drained. Horizontal displacements, mm

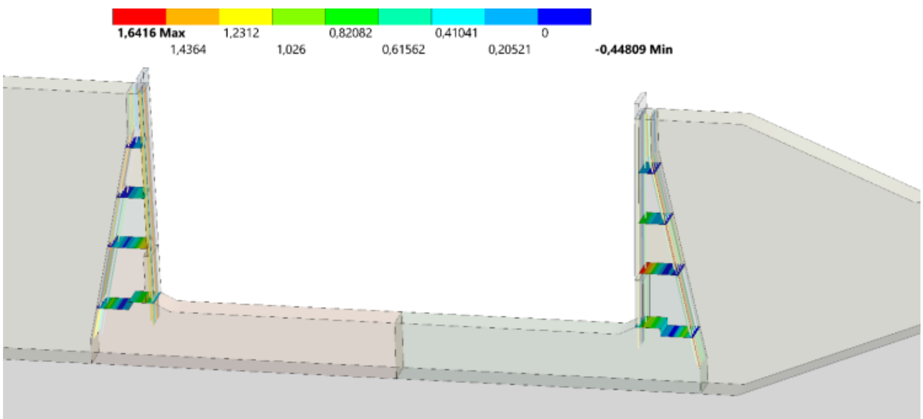


**Fig. 6.** Both chambers are drained. Distribution of vertical stresses in construction joints, MP

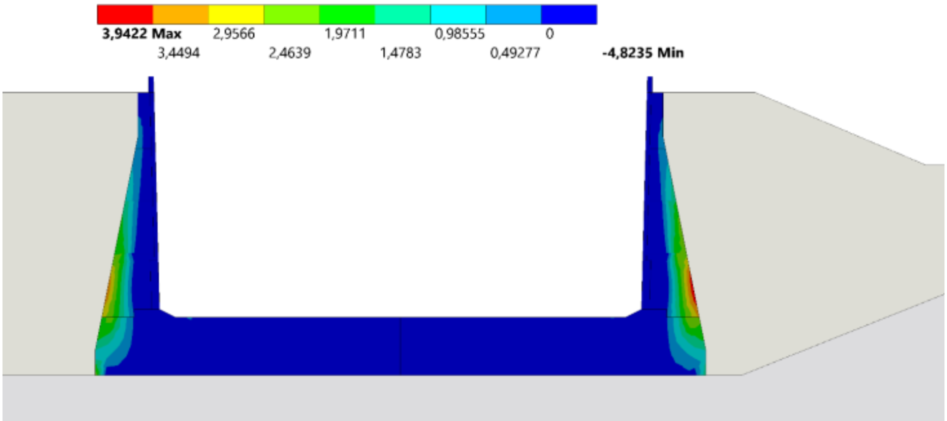


**Fig. 7.** Both chambers are drained. Vertical stresses in the wall of the lock chamber, MP

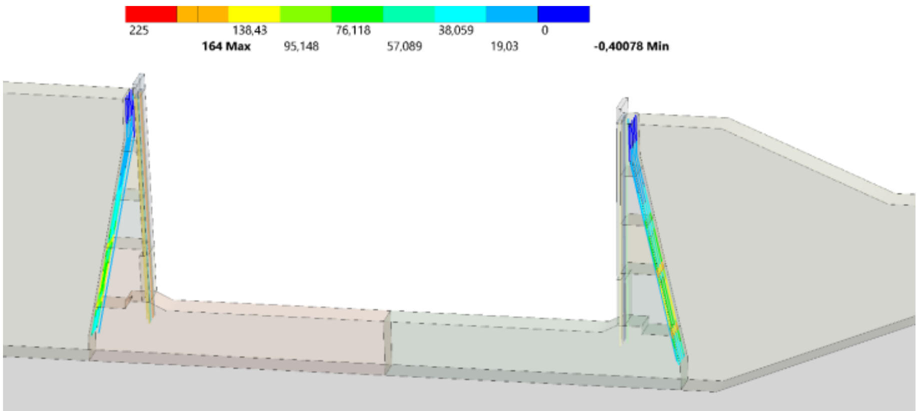


**Fig. 8.** Both chambers are drained. Stresses in the walls reinforcement of the lock chamber, MP

It can be seen from Figure 6 that the opening of the construction joints actually occurs to the full depth of the compressed cross-section zone. The vertical stresses in the concrete of the chamber walls without taking into account the work on strengthening the walls of the lock chamber are shown in Figure 7. As can be seen from Figure 7, the maximum compressive stresses in the concrete of the main array are 4.82 MP, while the horizontal construction joints are opened from the rear side.

The corresponding stresses in the reinforcement are shown in Figure 8. As can be seen from Figure 8, the maximum stresses in the rear face reinforcement were 164 MP, which does not exceed the design resistance for Class A-II reinforcement (225 MP).

***Calculation results for a combination of loads 2 - one of the chambers is drained***

Figure 9 shows the distribution of horizontal displacements.

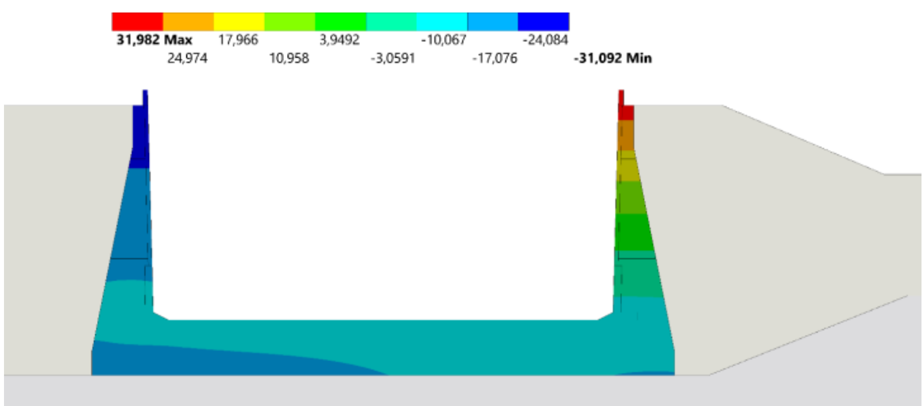


**Fig. 9.** One of the chambers is drained. Horizontal displacements, mm

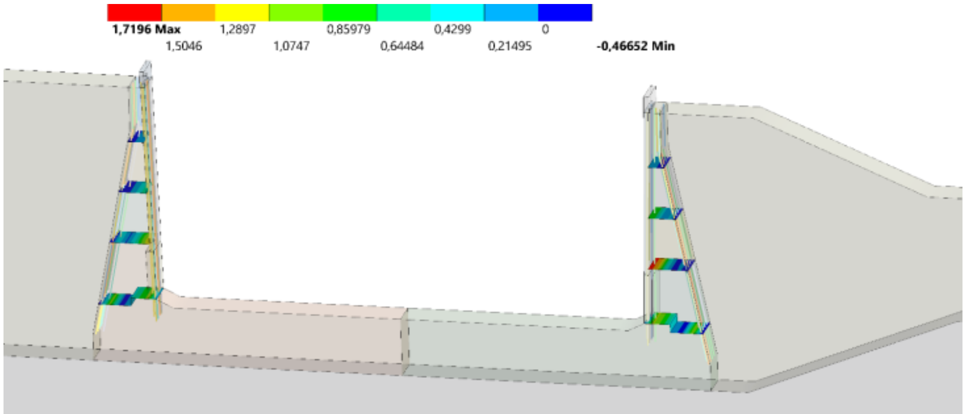


**Fig. 10.** One of the chambers is drained. Distribution of vertical stresses in construction joints, MP

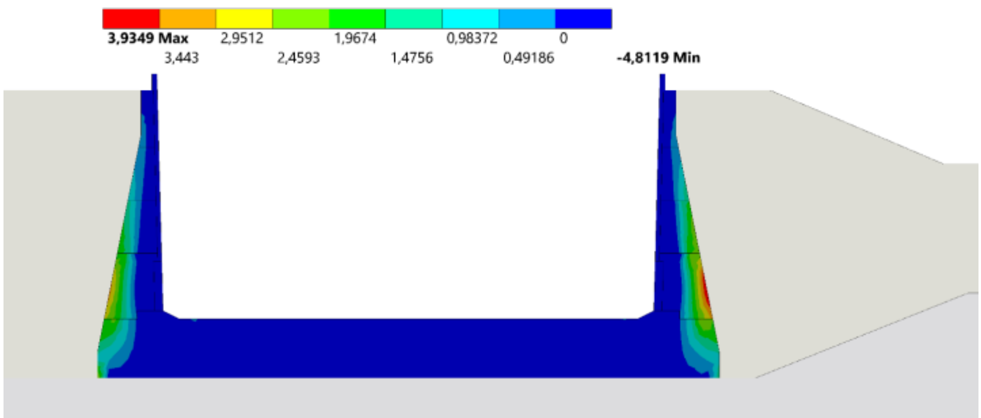


**Fig. 11.** One of the chambers is drained. Vertical stresses in the walls of the lock chamber, MP

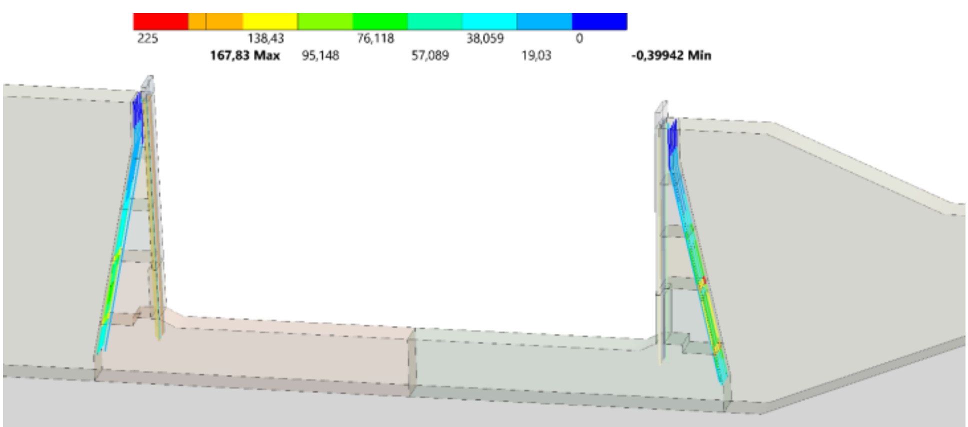


**Fig. 12.** One of the chambers is drained. Stresses in the walls reinforcement of the lock chamber, MP

As can be seen from Figure 9, the maximum horizontal displacements of the chamber top are directed inside the chamber and amount to 32.0 mm. It can be seen from Figure 10 that the opening of the construction joints actually occurs to the full depth to the compressed cross-section zone.

Vertical stresses in the chamber wall without taking into account the work on strengthening the walls of the lock chamber are shown in Figure 11. As can be seen from Figure 11, the maximum compressive stresses in the concrete of the main array are 4.81 MP, while horizontal construction joints are opened from the rear side.

The corresponding stresses in the reinforcement are shown in Figure 12. As can be seen from Figure 12, the maximum stresses in the rear face reinforcement were 168 MP, which does not exceed the design resistance for Class A-II reinforcement (225 MP).

As a result of the concrete stripping of the front face of the wall during the strengthening, that is, the reduction of the design section of the wall, horizontal displacements directed inward increase to 38.5 mm, while the stresses in the reinforcement of the rear face increased to 193.3 MP, compression stresses in the concrete of the main array increase to 5.21 MP.

Figure 13 shows the stresses in the concreting block under the horizontal construction joint in the wall of the lock chamber.

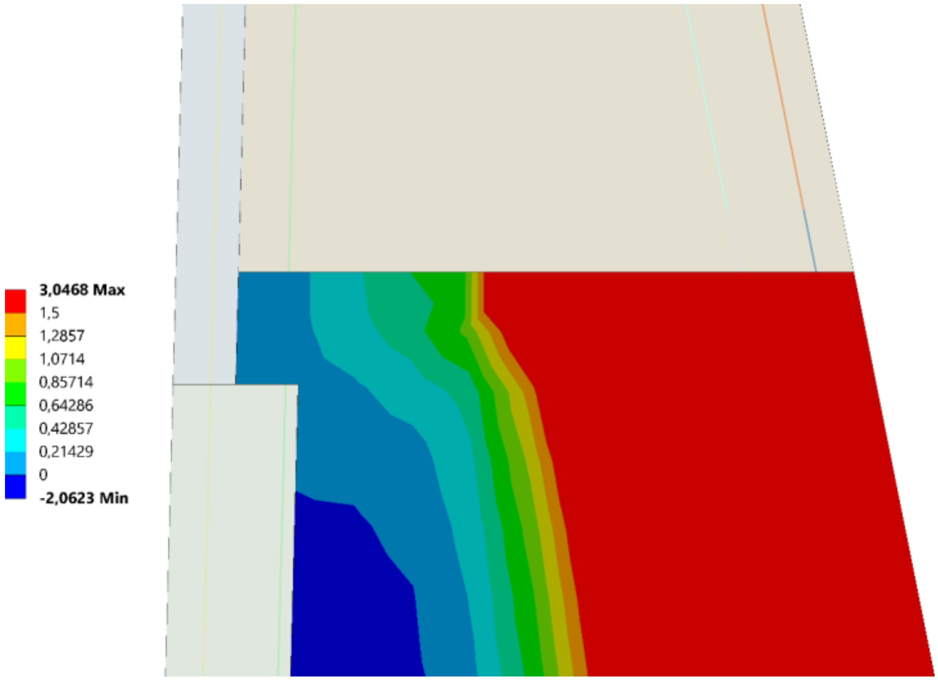


**Fig. 13.** Maximum main stresses in the concreting block under the construction joint

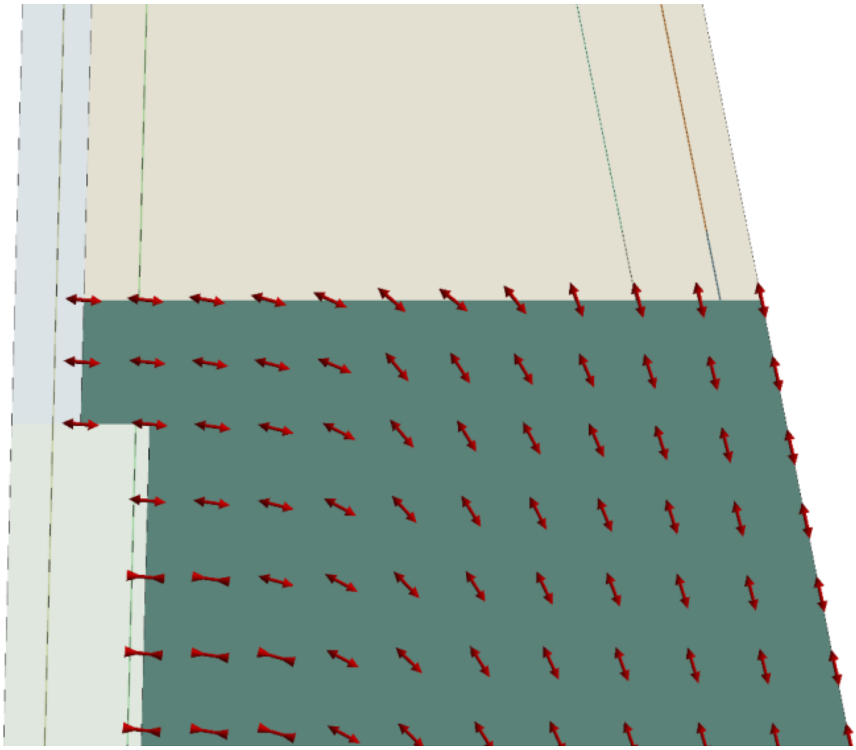

**Fig. 14.** Vectors of maximum main stresses in the concreting block under the construction joint

Figures 13 and 14 show the zones of transverse tensile stresses in the concreting block under the horizontal joint between blocks. Horizontal transverse reinforcement is required to perceive the revealed tensile stresses.

## 4 Discussion

The results of the performed computational studies complement the information base of calculations of reinforced concrete structures of locks. Calculations of the stress-strain state of reinforced concrete structures of the lock chambers are based on data from long-term monitoring of the structure, and the results of field surveys, including diving surveys of hydraulic structures.

As a result of this work, a spatial finite element model of the "structure - backfilling - foundation" system for locks No. 15 and No. 16 was developed. The spatial finite element model takes into account horizontal and vertical interlocking construction and temperature-shrinkage joints, as well as the actual reinforcement of the walls of the lock chambers.

The calculated cases were considered when the load on the rear face of the wall is the greatest since the stresses in the reinforcement at the rear face and the displacement of the top of the walls into the chamber are the most dangerous, that is, in fact, the repair period during which both chambers are drained, the groundwater level is the greatest while taking into account the impact of the average daily coldest five–day period (winter temperature -15.4°C).

The calculated studies take into account the results of surveys, including the determination of the physical and mechanical characteristics of concrete. The verification calculations of the lock were performed in a modern computing software package based on finite element models of the "construction - backfilling - foundation" system.

Calculations have shown that water back pressure and ice expansion in the vertical construction joint between the new repair concrete and the concrete of the main array leads to the development of cracking along the vertical construction joints.

In the course of computational studies, cracking in the walls along the contact surfaces of horizontal construction interblock joints in the concrete of the main array was revealed. It also revealed the presence of tensile stresses in the concreting blocks under the interblock joints.

In accordance with the requirements of pp. 8.20-8.26 of SP 41.13330.2012, horizontal transverse reinforcement is required in the structures of the walls of the lock chambers in the areas of horizontal interlocking joints, which was confirmed by calculations. Preliminary calculations of anchoring for reinforcement with a diameter of 12 mm showed that the depth of the anchoring should be at least 640 mm, while the analysis of the actual anchoring showed that it was not enough.

Computational studies have identified that the maximum displacement of the top of the walls inside the lock chambers was 38.5 mm with stress in the stretched reinforcement of the rear face of the walls – 193.3 MP, which is close enough (86%) to the design resistance for class A-II reinforcement (225 MP), compression stresses in the concrete of the main array increase to 5.21 MP, which close enough (87%) to the design resistance for concrete of class B10 (6 MP).

## 5 Conclusions

1. Computational studies have shown that the maximum displacement of the top of the walls into the lock chambers was 38.5 mm at stress in the stretched reinforcement of the rear face of the walls – 193.3 MP, which is close (86%) to the design resistance for class A-II reinforcement (225 MP), compression stresses in the concrete of the main array increase to 5.21 MP, which is close (87%) to the design resistance for concrete of class B10 (6 MP).

2. In the course of computational studies, cracking in the walls along the contact surfaces of horizontal construction interblock joints in the concrete of the main array was revealed. Computational studies have shown the presence of tensile stresses in the concreting blocks under the interblock joints. In accordance with the requirements of pp. 8.20-8.26 of SP 41.13330.2012, horizontal transverse reinforcement is required in the structures of the walls of the lock chambers in the areas of horizontal interlocking joints, which was confirmed by calculations.

3. As a result of the repair, the working reinforcement of the front face of the walls of the lock chambers above the 64.50 m is located in new concrete in accordance with clause 10.324 of SP 63.13330.2018 "Concrete and reinforced concrete structures. Basic provisions", as well as paragraph 8.107 "Manuals for the design of concrete and reinforced concrete structures of hydraulic structures (without prestressing)" (P46-89) to SNiP 2.06.08-87 (formerly SNiP II-56-77) it is necessary to ensure reliable anchoring of new (repair) concrete in the array of the main (old) concrete lock chamber walls. Analysis of the actual anchoring showed that it is not enough.

Calculations performed in accordance with the requirements of SP 63.13330.2018 and "Manuals for the design of concrete and reinforced concrete structures of hydraulic structures (without prestressing)" show that anchoring is required at least 640-760 mm, while the existing anchoring is 240-500 mm.

4. The results of computational studies have shown that the chambers of locks No. 15 and No. 16, located side by side, at a distance of 18 m from each other, interact with each

other. Therefore, in further calculations of the stress-strain state, it is necessary to take into account this interaction.

5. When constructing an additional chamber (No. 15A) of gateway No. 15, it is necessary to take into account the actual stress-strain state of the walls presented in this article in order to ensure the safe condition of existing lock chamber structures.

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