IOP Conf. Series: Materials Science and Engineering **365** (2018) 042074 doi:10.1088/1757-899X/365/4/042074

Stress-strain state of earth dams with a clay cement-concrete diaphragm

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Abstract. The purpose of this research is to calculate stress-strain state of an earth dam with a "diaphragm wall" along the axis of the dam and coupling "diaphragm wall" with a rock bed. "Diaphragm wall" is a vertical clay cement-concrete diaphragm made of secant piles, which elastic modulus varied during the calculation 60 ÷ 120 MPa. The numeric modeling of the stress-strain state was carried out at the elaboration of structure variants for the earth dam of the Nizhne-Bureyskaya Hydro Power Plant (HPP) height about 40 m. Evaluation and analysis of working capacity of the cement-clay concrete diaphragm were carried out on the basis of computational modeling results, which were obtained using the finite element method (FEM). A specialized computer program was used for this calculation, developed at the Hydraulics and Hydraulic Engineering Department of the Moscow State University of Civil Engineering (MGSU). The modeling was carried out in flat setting. In the modeling were included a staged constructing of the earth dam and staged applying loads. There was considered also two variants of presence of friction between ground and liquid clay cement concrete. At the first variant, friction was taken close to zero at the beginning of the calculation and was increased at following stages, and at the second variant, maximum possible friction was taken at all stages of the calculation. For these two variants of the calculation were modeled two variants of embedding the clay cement-concrete diaphragm into the rock bed - rigid and compliant. In this research are shown all results of the modeling of the stress-strain state of the earth dam: the distribution of stresses in the earth dam and in the "diaphragm wall" and also vertical and horizontal displacements in the diaphragm for all described variants. Conclusions were done as a result of this research about effect of construction variants at results of the calculation. All modeling results were analyzed. Recommendations were given for the structure of impermeable diaphragms.
Keywords: earth dam, impermeable diaphragm, stress-strain state (SSS), clay cement-concrete (CCC), finite element method (FEM).

1. Introduction

The proposed modeling was carried out at the Department of Hydraulic Engineering of the Moscow State University of Civil Engineering. The purpose of this modeling was to assess deformability and stability of earth dams under static loads.

Clay cement-concrete diaphragm walls were used in dam body as anti-filtration elements only for small dams. There are no horizontal displacements, and pressure gradients and filtration rates are sufficiently small [1].

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Introduction of domestic practice of hydraulic engineering CCC diaphragm walls as anti-filtration elements of high dams requires implementation of researches and experimental and technological works aimed at determining standards for critical gradients of CCC pressure, improving their deformation-strength characteristics, development of calculation methods [2].

A work on the numeric modeling of earth dams was carried out and described in scientific works. Finite elements were used also for these calculations [3]. For example, in the work [4] author considered the model which included geometric factors, depending on SSS of earth dams at the process of filling it with water.

The Nizhne-Bureyskaya earth dam is projected maximum height of the central section of about 40 m. The dam's project level is 141.75 m; the width along the coping is 14.5 m. The normal water level (NWL) is 138.00 m. Dam's persistent prisms consist of gravel and pebble, the central prism - of sandy soil. The slope of the pressure face is 1:2; of the downstream side - 1:1.75; of rocky prisms - 1:1.5.

The vertical clay cement-concrete diaphragm was accepted as the main anti-filtration element with constant thickness. It is made of secant piles and located along the axis of the dam. The diaphragm will be made after the dam is built to the level 138.5 m, i.e. almost the entire height. It must cross dam body in the central section and come in the rock bottom. The diaphragm will also cross gravel-pebble soils in floodplain areas. A two-row cementing curtain is the anti-filtration element in the rock soil with a distance between rows of wells 4.0 m and a step of wells in the row 3.0 m.

Secant piles are made with a diameter 1.2 m and a step of piles 0.85 m. Piles coupling with concrete facing of sides is carried out with clay cement-concrete «pillows». The embedment is 1.0 m into the rock soil.

The dam is built in an open channel in an uncoated way. The basement of the dam is made by the method of pouring soil into the water below the level 113.50 m, above the level 113.50 m - by the dry filling of soft and rocky soils, layered with a compaction by vibrating and pneumatic rolls and by passing transport.

The main stages of the dam's building:

1. Filling the dam body with soil between the mating base and the interim dam to the level 115.00 m;
2. Channel overlap and pouring of soils into the water to the level 113.50 m;
3. Dry filling the dam body with soil to the level 138.50 m;
4. "Diaphragm wall" building from the level 138.50 m;
5. Filling the reservoir with water to the level 128.00 m and beginning of work the hydroelectric power station on low pressure;
6. Filling the dam body with soil to the level 141.75 m;
7. Filling the reservoir to the level 138.00 m and beginning of work the hydroelectric power station on the full time.

2. Methods

The analysis of work capacity of dam's variants was carried out based on a numeric modeling of their stress-strain state (SSS) under static loads.

Numeric simulation was performed by the finite element method (FEM). Calculations were carried out with a flat setting [5]. We used for calculations the computational program, developed at the Department of Hydraulic Engineering of the Moscow State University of Civil Engineering [6]. A special feature of the program is the ability to take into calculations some large discontinuities in the computational domain (contacts, cracks, seams, etc.) in an explicit form, using a modified contact-element in the computational scheme.

The non-linear model of a contact interaction (physical model) allows to take into calculations opening and closing of contacts between solid blocks, elastoplastic work of the contact by shear load (including possible softening, using the Mohr–Coulomb model), and also dilatancy properties of cracks by shear deformations [7].

For the numeric modeling of all elements are used finite elements with quadratic approximation of displacements within an element [8]. We can consider an elastic or elastoplastic work of a solid material. The general procedure for solving nonlinear tasks allows us to consider formally and solid, and contact elements with elastoplastic models of the materials behavior within the framework of same unconditionally stable algorithm of the initial stress method. Divergence means that the system has exhausted the ability to withstand a load.

The FEM grid is shown in Figure 1 for the calculated incision. In the calculation area were included: the body of the dam, the "diaphragm wall", the alluvium at the base of the dam and the fragment of the rock soil. The FEM grid consists of 1330 finite elements of solid materials and 66 contact elements, united by 1481 nodal points. The interaction between the diaphragm and the dam body or the rock soil was modeled with contact elements.

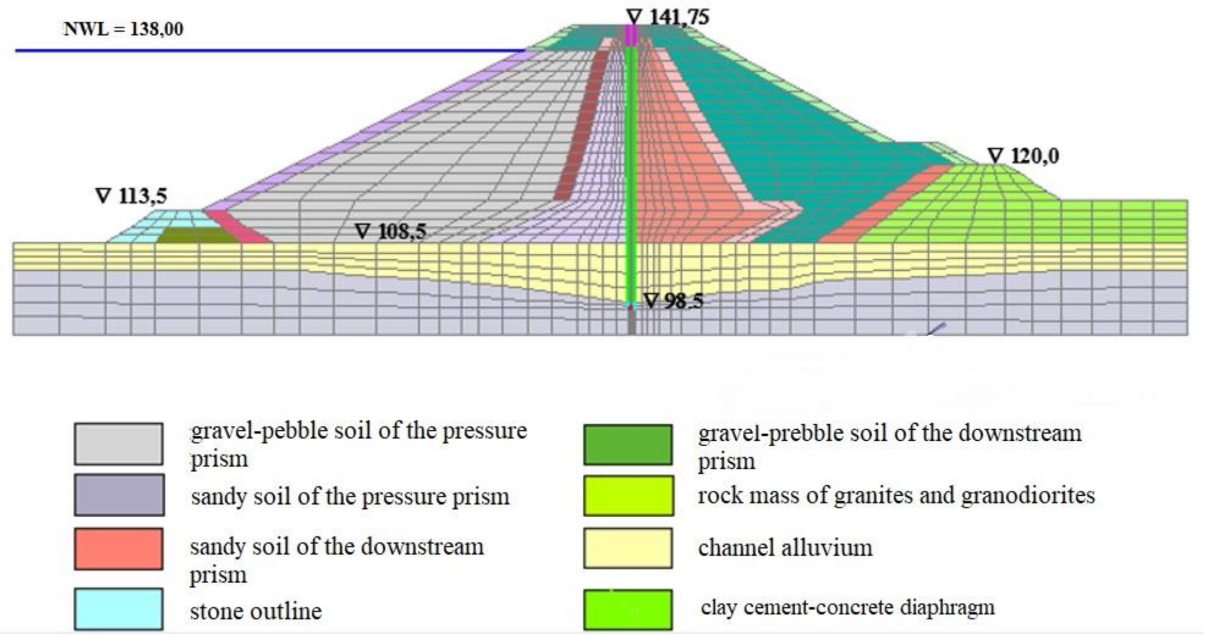

Figure 1. FEM grid for the calculated incision

As boundary conditions was taken absence of displacements along the bottom of the calculations area and absence of horizontal displacements along side faces of the calculations area. Total number of degrees of freedom (equations), including into calculations out-of-node degrees in the design scheme, was 5501.

Calculations were carried out for the main combination of next loads: own weight and hydrostatic water pressure on the "diaphragm wall". By calculations were taken into consideration the sequence of the dam building and loads application to it.

At first, we calculated the stress-strain state by loads from ground weight - the weight of rock massif and alluvium. The resulting stressed state in ground was saved and used in further calculations, and displacements were removed.

The building begins with the filling of pressure and downstream stone banquets. Then the basement part of the dam is built (to the level 113,50 m) by soils pouring into water. After its completion, building is carried out to the level 138,50 m in almost horizontal layers. By the calculation was modeled the embankment to levels 120.00 m, 129.00 m and 138.50 m. After this, the "diaphragm wall" is arranged in the dam body.

After the completion of the wall construction made of secant piles is simulated rise of water to the level 128.00 meters. Then the dam is filled by sandy and gravel-pebble soils to the dam's project level

(141.75 m), and water rises to the level of the NWL - 138.00 meters. The downstream level was taken at 109.16 meters.

Calculations of the earth dam SSS were carried out with next values of materials properties in the calculations area (Table 1). The angle of internal friction of the clay cement-concrete (30 deg.) and the ultimate uniaxial compression strength soil of the clay cement-concrete (0.65 MPa) were used during the calculation in addition to values given in the table.

Table 1. Materials properties

No	Name of soil	Density, g/cm^3				Modulus of deformation MPa	Poisson's ratio
		soils particles	dry soil	normal humidity soil	water-saturated soil		
1	2	3	4	5	6	7	8
1	Gravel-pebble soil (prisms)	2.65	2.10	2.20	2.30	70	0.27
2	Sandy soil (prisms)	2.63	1.70	1.79	2.05	40	0.30
3	Gravel-pebble soil (interjacent areas)	2.65	1.70	1.82	2.06	35	0.30
4	Rock mass of granites (fastening of slopes, stone banquets)	2.70	1.90	2.00	2.19	70	0.27
5	Channel alluvium	2.68	1.95	2.22	2.22	50	0.28
6	Granodiorites of the weathering area	2.72	2.66	-	2.68	2000	0.30
7	Clay cement-concrete «diaphragm wall»	1.8	-	-	-	60 80 120	0.30

Since it is very difficult to include into the numeric model a "deep folding" peculiarity of the diaphragm material into a cavity formed in dry filtering material with water outflow from the mixture. Besides the pressure on the material is changing during pile erection, following variants of boundary conditions were considered by the "diaphragm wall" building:

Variant No 1. In calculations was included a "liquid state" of the clay cement-concrete at the initial moment of time after filling the well. At that moment, friction was assumed to be close to zero between ground and the liquid clay cement-concrete, and the soil of the diaphragm wall could slip relative to the surrounding ground mass. Actually, friction will be observed between ground and the clay cement-concrete in reality, but the calculation without friction allows us to estimate the largest compressive stress in the diaphragm wall and compare them with the ultimate uniaxial compression strength soil of the clay cement-concrete. At next stages, the clay cement-concrete hardened and became the deformation modulus (3 variants - 60 MPa, 90 MPa, 120 MPa). Tangential stiffness and friction were restored during interaction with the soil. Tensile strength was assumed to be close to zero.

Variant No 2. Maximum possible friction was observed at the time of filling the well with the clay cement-concrete, assuming slippage with the coefficient of friction equal to the shear strength of the surrounding soil. This calculation was carried out with allowance for friction and correspondingly the diaphragm material hanging when it is built. These conditions reduce compression level in the diaphragm and, by further loading with water pressure, makes it possible to estimate the possibility of the appearance of tensile stresses and cracking.

There were modeled two variants of embedment the wall into the rock - rigid and compliant for both variants of the diaphragm building and further loading. Stresses and displacements in diaphragm elements (which modeled the core sand), which were obtained by the diaphragm modeling at previous stages of building, were reset.

3. Results and discussion

In Figure 2 is shown normal stresses in the dam body when it is built to the level 138.50 meters. The stress-strain state is represented in the form of colored fillings in finite elements. Color depends on the value of stresses.

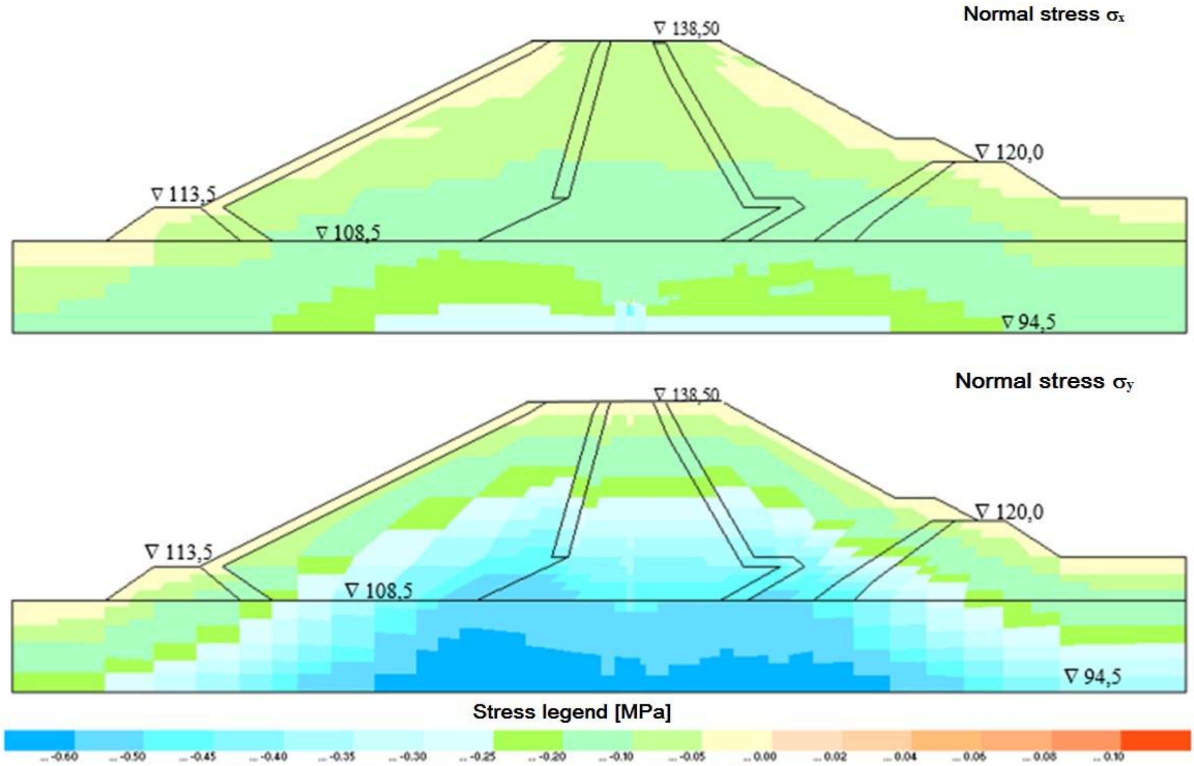

Figure 2. Normal stresses in the calculated incision of the dam at the building to the level 138,50 meters

Horizontal stresses σ_x in the dam body do not exceed -0.2 MPa, and in ground under dam -0.5 MPa (ground weight was included in calculations). Vertical stresses σ_y reach - (0.5 ÷ 0.6) MPa in the dam body in the base of the pressure prism. Stress distribution pattern is close to the symmetric with the axis of the dam. In horizontal incisions of the earth dam, stresses in lateral prisms are about 15 ÷ 20% higher than in the body. This can be explained by the greater density and rigidity of the gravel-pebble soil of prisms and interjacent areas as compared to the sand in the dam body. In the rock soil under the dam, stresses σ_y reach -0.8 MPa. Displacements on the coping level 138.50 m were accumulated at all building's stages: sinking - 20.7 cm; horizontal displacement along the axis of the dam - 2.7 mm in the direction of the downstream side.

Results of the SSS calculation are shown in Figure 3. There is the variant of the calculation without friction on surfaces sides during the building of the diaphragm wall and with the pliable embedding in the rock. It shows normal stresses in the dam after the diaphragm building, after the dam’s filling to the level 147.75 m and after the lifting water level first to the level 128.0 m and then to the level 138.00 m.

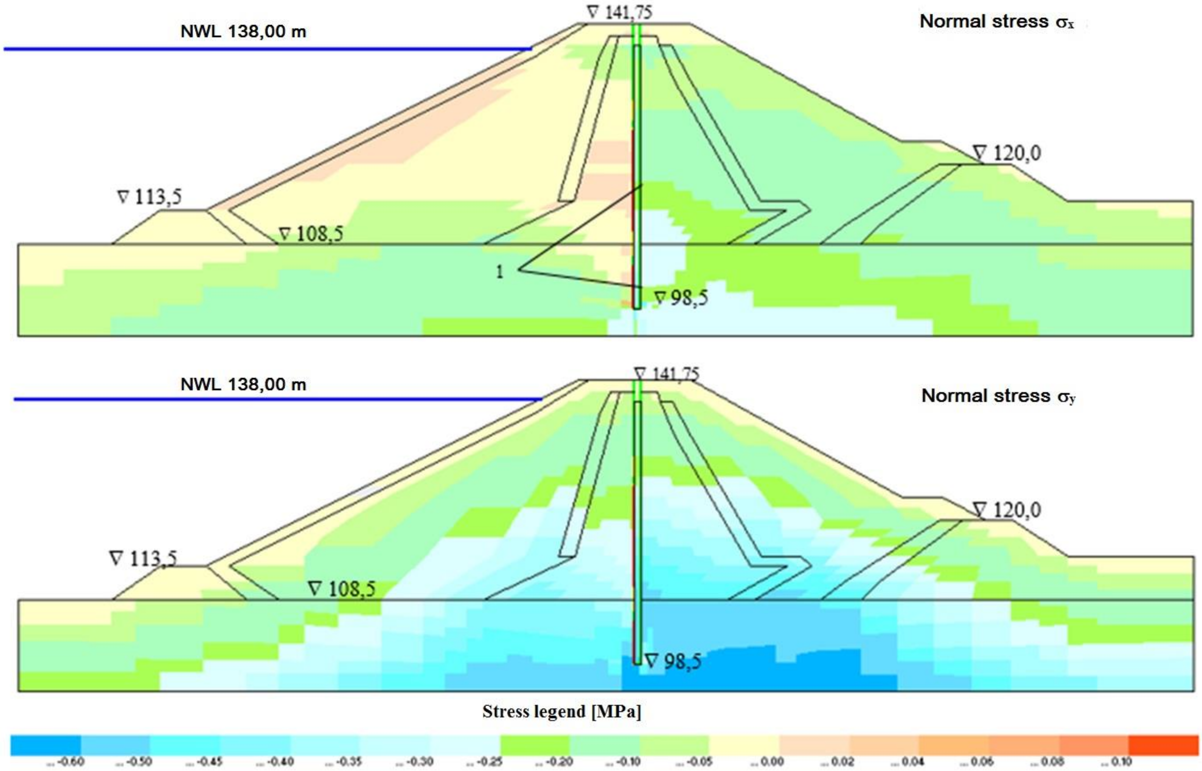

Figure 3. Normal stresses in the dam

Horizontal stresses σ_x decreased almost to zero in the prism of the pressure side in contiguity to the diaphragm and in the slope area and to σ_x = -0.1 MPa in the dam's coping area. In the downstream prism stresses σ_x increased, but did not exceed the value -0.3 MPa. Stresses redistribution is a reaction to the hydrostatic load. The hydrostatic effect explains also the areas detachment formation between the diaphragm wall and the body of the pressure prism in levels 110.00 ÷ 125.00 m in the dam and 99.00 ÷ 105.00 m under the dam. The size of the contact opening does not exceed 1 mm, and it will not cause decompaction of the ground. Vertical stresses σ_y in the pressure prism decrease also to values not exceeding -0.4 MPa. Stresses σ_y increased a little in the downstream prism, but did not exceed -0.6 MPa. The maximum compression in the ground is σ_y = -0.85 MPa in the rock below the embedment of the diaphragm. Tangential stresses are small and do not exceed 0.1 MPa in the dam body.

Displacements and normal stresses in the diaphragm are shown in the Figure 4. There is the variant without friction along surface sides of the diaphragm and with the pliable embedding.

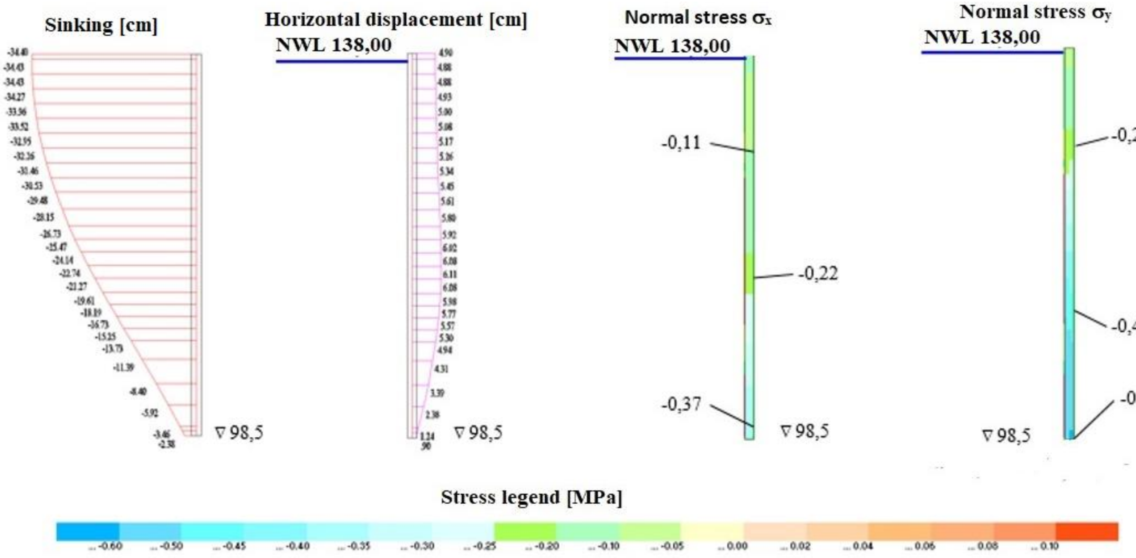

Figure 4. Displacements and stresses in the diaphragm wall

We can see on the diagram of the diagram wall sinking, that most of its sinking was formed due to pressure from its own weight at the moment when the clay cement-concrete was in the liquid (or semi-liquid) state. The magnitude of CCC slippage is quite high and amounts to approximately (10-13) cm in the absence of the contact friction with the ground. The accumulated sum of the diaphragm top displacement was about 34 cm due to weight of overlying soils. Horizontal displacements of the diaphragm wall are small. They are directed toward the downstream pool and are caused by the hydrostatic pressure from the reservoir. The maximum displacements value is 6.1 cm at the time of completion of the reservoir filling. The maximum of horizontal displacements is observed near the level 118.00 m, i. e. approximately in the middle of the diaphragm wall. However, the embedment in the rock does not allow the diaphragm to move freely (displacement is limited to 10 mm in the rock), so it is deflected. The deflection is approximately 0.15% of the diaphragm wall height. Thus, the diaphragm of clay cement-concrete in the dam have flexural deformations although, but they are not significant.

The analysis of vertical stresses σ_y shows, that the main factor in the SSS formation in the diaphragm wall is the pressure from its own weight. Other factors (sinking of surrounding soil due to weight of overlying layers, as well as flexural deformations of the diaphragm) are not so important. The diaphragm wall has compressive stresses σ_y in all incisions formed mainly by its own weight. Tensile stresses do not arise. The maximum stress value reaches -0.64 MPa at the time of the reservoir filling to the NWL. They did not exceed -0.61 MPa before filling. Differences in the distribution of stresses are very small on pressure and downstream faces of the diaphragm, because the flexion of the "diaphragm wall" is small. The contact diaphragm-rock is compressed by σ_y = -0.58 MPa on the pressure side of the earth dam.

Stresses σ_x are less than stresses σ_y, but the diaphragm undergoes compression in all incisions. Stresses σ_x increase with depth and reach the maximum near the contact with the rocky soil. When the reservoir is filled to the NWL, the maximum stress value σ_x is -0.4 MPa. The ratio σ_y/σ_x is about 1.6. Stresses σ_x does not exceed -0.3 MPa before the reservoir filling. Apparently the time point before the reservoir filling is more dangerous from this point of view, because role of the diaphragm compression is reduced.

Similar calculations were carried out and analyzed for all calculation variants described in the chapter *Methods*. All obtained values of normal stresses and displacements are summarized in Table 2 for all considered variants, so to compare all results of their calculation.

Table 2. Final results of the calculation of the clay cement-concrete diaphragm

Design parameters	Deformation modulus of the CCC E=**60** MPa				Deformation modulus of the CCC E=**80** MPa				Deformation modulus of the CCC E=**120** MPa			
	Without friction on the side surfaces		With friction on the side surfaces		Without friction on the side surfaces		With friction on the side surfaces		Without friction on the side surfaces		With friction on the side surfaces	
	Rigid embed.	Compliant embed.	Rigid embed.	Compliant embed.	Rigid embed.	Compliant embed.	Rigid embed.	Compliant embed.	Rigid embed.	Compliant embed.	Rigid embed.	Compliant embed.
Sinking of the diaphragm, [cm]	38.48	39.0	22.1	22.34	30.31	34.4	21.94	22.18	30.21	30.3	21.8	21.99
Horizontal displ. of the diaphragm, [cm]	6.11	6.12	6.07	6.08	6.09	6.11	6.04	6.08	6.04	6.1	5.99	6.05
Contact between the diaphragm and the rocky soil	Comp. 0.17 MPa	Comp. 0.58 MPa	Opening contact L=0.37 m δ=0.52 mm	Comp. 0.14 MPa	Comp. 0.13 MPa	Comp. 0.58 MPa	Opening contact L=0.5 m δ=0.9 mm	Comp. 0.14 MPa	Comp. 0.04 MPa	Comp. 0.55 MPa	Opening contact L=0.63 m; δ=1.54 mm	Comp. 0.15 MPa
The max. of the compression in the diaphragm, [MPa]	0.67	0.64	0.23	0.20	0.69	0.64	0.26	0.22	0.73	0.64	0.31	0.24
Possible cracking in the diaphragm	No	No	Yes 0.25×1.0 m	No	No	No	Yes 0.25×1.0 m	No	No	No	Yes 0.5×1.0 m	No

4. Conclusions

1. Compliance of the contact between the diaphragm and ground has the greatest influence on the diaphragm stress-strain state on the lower part of the diaphragm (0.5-1.0 m). It should be noted, that compliance will be in reality, because contact with the rocky soil will not be rigid after removal of the casing pipes. This is due to a large aggregate of the CCC forms point contacts, and between them - clay-cement milk, which is substantially more deformable than accepted diaphragm deformation modules.

2. When the compliance is included in calculations, the diaphragm contact with the rock is not open by any boundary conditions along side surfaces of the diaphragm and by all of deformation modulus varied in the range 60 ÷ 120 MPa. There is an opening contact, when the rigid rock embedment is included in the numeric modeling. There is a possible hanging of the diaphragm material in this case due to friction along side surfaces at building time. The maximum size of the opening can be 0.6 m along the length of the horizontal contact section with the deformation modulus of the CCC 120 MPa by the contact opening to 1.5 mm.

3. Sinkings shown in previously figures include actually all vertical displacements, which is approximately twice large as actual strains of layers. Thus, we can expect sinkings of about 15 ÷ 20 centimeters.

4. Conditions in the contact between the diaphragm and the rocky soil deteriorate somewhat with a hardness increase of the CCC, because opening area of the contact increases, when friction is included along side surfaces of the diaphragm at building time. It can be recommended in this regard to strive for a more compliant material of the diaphragm wall (E = 60 MPa).

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