E3S Web of Conferences **401**, 01010 (2023) https://doi.org/10.1051/e3sconf/202340101010
CONMECHYDRO - 2023

Capacity of second spillway of Boguchanskaya HPP

A. P. Gouriev, *N. V.* Khanov, *V. A.* Fartukov, *M. I.* Zborovskaya*, and *M. M.* Abidov

Russian State Agrarian University - Moscow Timiryazev Agricultural Academy, Moscow, Russia

Abstract. The main problem in designing the discharge of water flow from the upstream to the downstream is to ensure the capacity of spillway structures. This article discusses the design options for the design top of the spillway: with water diversion through a short nozzle in the pressure mode and energy dissipation in stilling basin, with the pressure-free and pressure-free passage of water through spillway No. 2 and damping of the flow energy by jet discharge into the lower stream by the inclined face of the threshold in the stilling basin; with no pressure passage and pressure passage of water through spillway No 2 and energy dissipation of the flow by throwing the flow discharge into the downstream by the inclined face of the threshold of a stilling basin; with the passage of building expenses through the sill in no pressure and pressure-type short-nozzle modes, with the spillway No. 2's horizontal water outlet at the level of 139.5 m, with a jet discharge by a toe-springboard at an angle of 35°, and with the outlet rib's mark of 143.8 m. This article aims to obtain the flow characteristics of the considered spillway option for the period of temporary operation of the hydrotechnical complex.

1 Introduction

The hydropower plant on the Angara River, near Kodinsk, Kezhemsky District, Krasnoyarsk Territory, is part of the Angara HPP Cascade, its fourth, lower stage. With an installed capacity of 2,997 MW, it is one of Russia's five largest hydropower plants.

The Boguchanskaya HPP technical project was approved on December 7, 1979, by decree of the Council of Ministers of the USSR ¹ 2699P. Its nominal capacity was 3 000 MW, normal reserve level of the reservoir was 208 meters.

In 1980 construction of the main structures of Boguchanskaya HPP started. But after the disintegration of the USSR, the pace of Boguchanskaya HPP construction slowed down considerably, and in 1994 the construction was suspended.

Resumed construction of the plant in the framework of public-private partnership since 2005. Institute "Gidroproekt" adjusted the technical project of Boguchanskaya HPP. In particular, the requirements to pass the flood discharges were tightened, which led to the necessity to design spillway number 2, which was placed in the station dam in place of hydroelectric units 10-12.

The design of spillway No. 2 required the use of non-standard technical solutions,

*Corresponding author: zborovskya@rgau-msha.ru

variants of which are given in [1-3] and this article.

The issues of designing spillway structures considered in papers [4-16] should also help in answering the questions about researching the need to improve the existing system of management and planning of hydropower plants within the framework of interacting power and water management systems, using the criteria of economic, social and ecological efficiency. And as indicated in [17], planning long-term power balances and increasing their feasibility should include forecast scenarios of water inflow into reservoirs for up to 1 year and their monthly adjustment, ensuring the safe operation of hydro-technical structures.

2 Material and Methods

The longitudinal section of the spillway No. 2 with combined options structures for the period of temporary operation of the hydroelectric complex is shown in Figure 1:

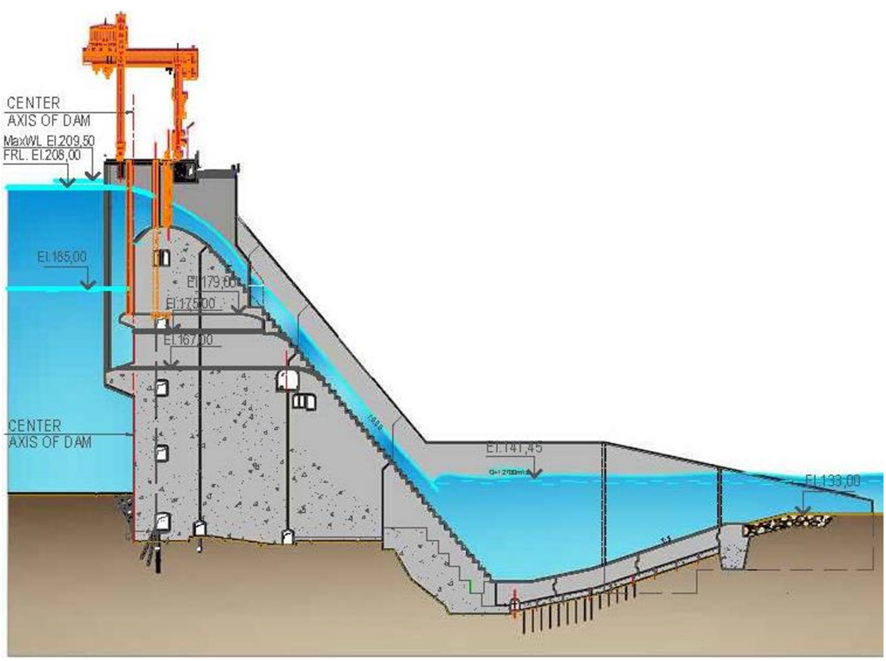


Fig. 1. Longitudinal section of spillway No.2 with combined options structures for period of temporary operation of hydroelectric complex

The layout of spillways structures of the Boguchansky hydroelectric complex is presented in Figure 2:

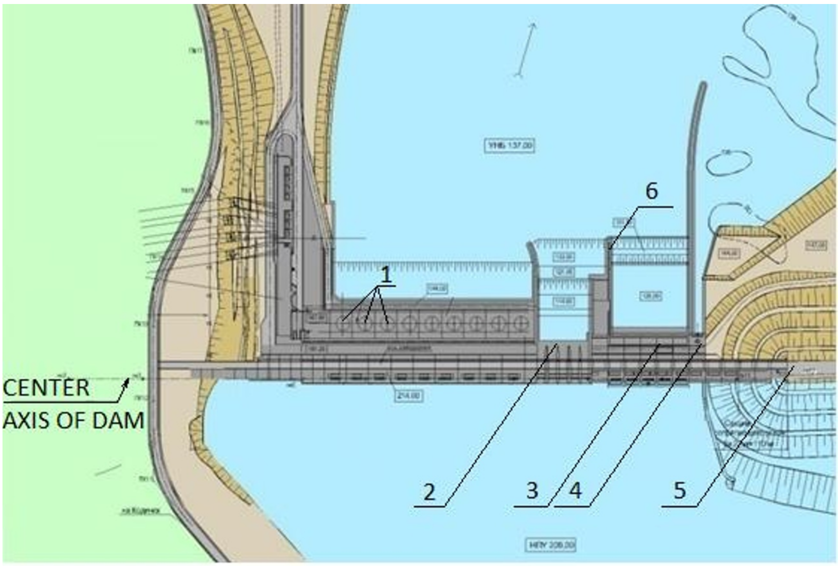


Fig.2. Layout of spillways structures of Boguchansky hydroelectric complex: 1 is hydroelectric power station; 2 is surface spillway No.2; 3 is deep spillway No.1; 4 is timber transmission facility; 5 is ground dam; 6 is separate wall between spillways No.1 and No.2

The calculated flood hydrograph with design probability P = 0.2% is presented graphically form in Figure 3:

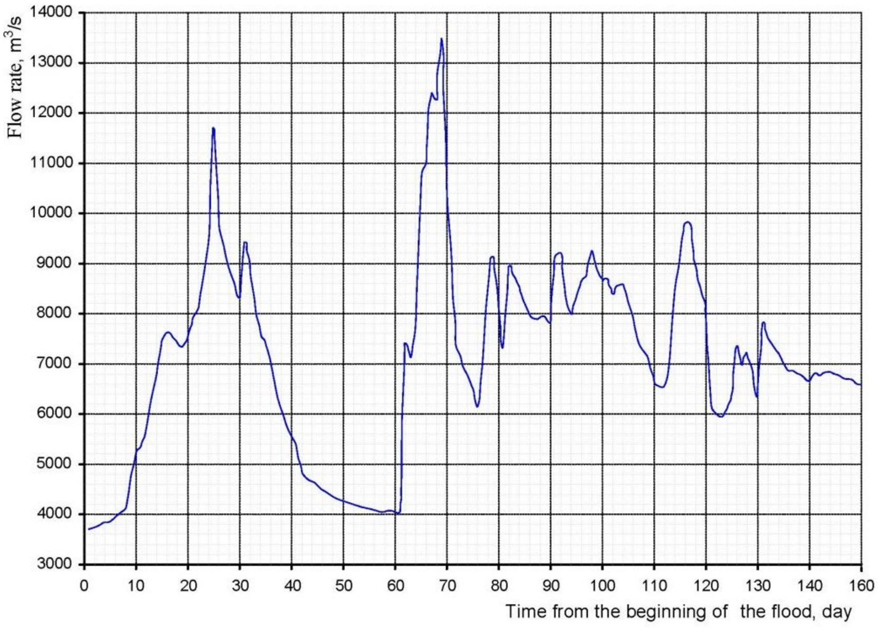


Fig. 3. Calculated flood hydrograph with design probability P = 0.2%

The dependence of the initial data on the capacity of spillway No.1 is well described by the following equation:

$$Q = 976.4 \cdot (\nabla RWL - 150.1)^{0.487} \quad (1)$$

Taking into account fluctuations in the water levels downstream when the upstream level changes and the number of working water discharge openings of spillways No.1 and No.2 and HPP units, the flow rate of one turbine can be described by the following equation

$$Q_{\text{т}} = 142.64 + 10.54 \cdot (\nabla RWL - \nabla TWL) - 0.0632 \cdot (\nabla RWL - \nabla TWL)^2 \quad (2)$$

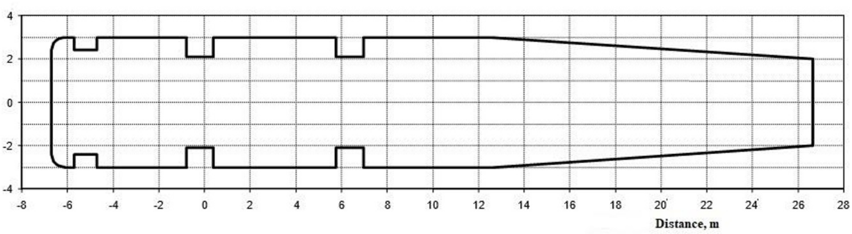
the dependence of flow rate on the water levels downstream of the Angara River Q = *f* (TWL) is well described by the equation:

$$Q = 524 \cdot (\nabla TWL - 135.5)^{1{,}786} \text{ m}^3/\text{s} \quad (3)$$

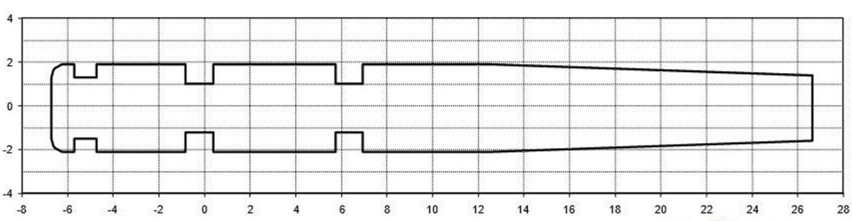
which allows us to obtain the dependence of the depth of water downstream of the h_{TWL} depending on the flow rate of water:

$$h_{TWL} = 0.03 \cdot Q^{0.56} \text{ m} \quad (4)$$

Drawings of the cross-section of the separation piers developed, taking into account the requirements of the operation of the hydroelectric complex during the construction period, are shown in Figure 4, a, b.



a)



b)

Fig. 4. Cross-section of separating piers: *a*) 6.0 m thickness; *b*) 4.0 m thickness

For the temporary exploitation of the hydrotechnical complex, the variant of skipping of building flow rates was considered through the operation of spillway No. 2 of the Boguchanskaya HPP as the main spillway facility. In this variant, the calculation of the spillway No.2 capacity was performed with free overflow through a flat threshold at the mark El.167.0 m at the Full Reservoir Level (FRL) El.175 m and water level forcing up to the Reservoir water level (RWL)

In developing options for skipping construction flow rates during the temporary operation of the Boguchansky hydroelectric power plant structures, a variant with the operation of spillway No 2 as the main spillway structure was considered.

Spillway No. 2 begins to work when the Spillway capacity of the working units of the hydroelectric power plant and spillway No. 1 is exhausted. The inclusion of spillway No. 2 is provided not only for cases of skipping the flood of a design probability p = 0.2% with $Qp_{0.2\%}$ = 11 700 m^3/s (capacity of spillway No. 2 Q_{max} = 5 500 m^3/s) but also for the whole range of flow rates exceeding the total one.

The maximum flow rate of the hydroelectric complex $Q_{p=0.2\%}$=11 700 m^3/s, taking into account the transformation of the flood with probability P=0.2% at which 5500 m3/s is supposed to pass through the spillway No. 2.

2.1 Selecting the moment when spillway no. 2 is switched on the work

The preliminary hydraulic calculations have established that to pass the estimated construction flow rate with probability P = 0.2% head is H = 18 m is required at the threshold of the temporary water discharge opening of the spillway No. 2, at which the flow rate Q_{max} = 5 500 m^3/s shall be.

As mentioned above, the inclusion of spillway No 2 should be provided not only during the passage of the flood of the design probability but also in the entire range of flow rates exceeding the total throughput of the operating HPP units and the throughput of spillway No. 1.

Quite severe restrictions are imposed on the operation of the spillway structures of the Boguchanskaya HPP [18-24].

To ensure maximum power generation, it is necessary to maintain the upstream level of at least 185.0 m, corresponding to the calculated Full Reservoir Level (FRL) during the temporary operation of the hydroelectric complex.

On the other hand, to prevent flooding and flooding of territories located in the zone of influence of the reservoir at the temporary water reservoir level WRL = 185.0 m, there are restrictions on exceeding this mark.

However, these limitations are purely theoretical for the following reasons.

1. The control of the upstream level is carried out using instrumental observations performed using the tools for measuring water levels. Each instrument has inherent measurement accuracy corresponding to the measurement class. The level gauges used at HPP have an accuracy class of 2.0 and 2.5, with an error of ±1% of the measured head. With a head equal to 18 m, this error will be ± 0.18 m.

2. At hydroelectric power plants, water intakes are protected grilles against debris ingress to prevent failure of guide vanes and impellers. Due to the grilles clogging with floating debris, the level difference on the grilles constantly exceeds the design values. Cleaning of grates from garbage is carried out periodically. Therefore, to achieve the actual parameters of hydroelectric units at HPP, the level upstream reservoir is often measured behind the grilles against the ingress of the debris before entering the water intake openings, extending these measurements to the entire upper water area reservoir. This fact was registered by us during the calibration of the spillways of the Volgograd HPP in 1972.

3. On large reservoirs with a long length, significant fluctuations in the water level are caused up- and down- by surge phenomena.

The magnitude of the increase in the water level before the pressure front of the hydroelectric complex from surges (or decline level during a surge) is determined by the following dependence:

$$h_{up} = 2 \times 10^{-6} \times \frac{W^2 \times D}{g \times H} \times cos \propto \quad (5)$$

where W is the calculated wind speed, m/s; D is the wave acceleration length, m; g = 9.81

m/s^2 is free-fall acceleration due to gravity; H is the depth of the reservoir, m; α is the angle between the direction of the wind and the direction of the wave acceleration, °.
Following Russian State standard SP 38.13330.2018 "Loads and impacts on hydraulic structures (from wave, ice and ships). SNiP 2.06.04-82" when determining the mark of the crest of a dam of the 1st and 2nd class of capital, the surge is determined for the wind probability P = 2% under normal operating conditions. However, winds with a lower speed can cause surges of considerable height.

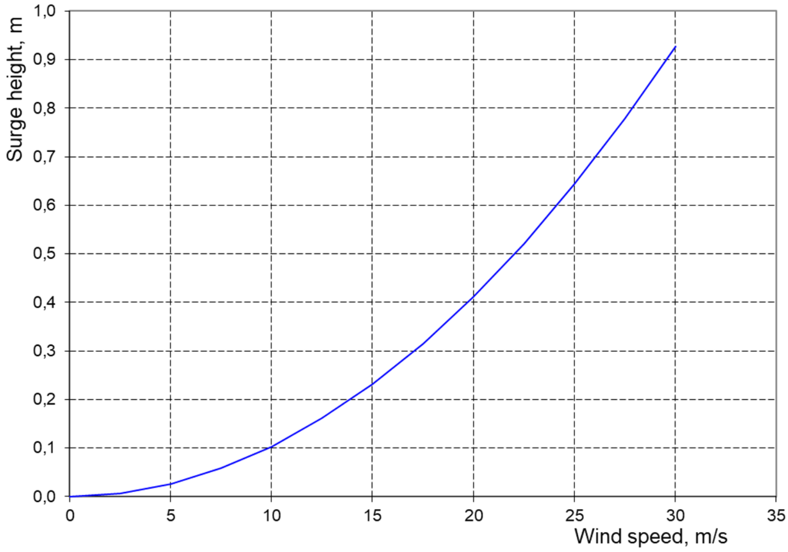


Fig. 5. Dependence of water surged height on wind speed upstream

When calculating the surge magnitude for the dam location, the water depth in the reservoir was taken to be $H_{average} \approx 49.5$ m and the length of the wave acceleration in the wind direction was 250 km.

As can be seen from Figure 5, moderate wind with a speed of up to 10 m/s can cause wave surges up to 0.10 m.

Under these conditions, choosing criteria for determining the reasons for the rise in the water level in front of the structures becomes important: the rise occurs due to an increase in water flow or a surge.

According to Russian State Standard SP 38.13330.2018, the time period t = 6 hours = 21 600 seconds is taken in the absence of relevant observations. This is the minimum time required for the complete development of the wave process. It follows that if, 6 hours after the beginning of the rise in the level of the head pool, its growth continues, then this fact can serve as a basis for the need to open the gates.

With a large area of the reservoir, as the case is for the Boguchany HPP, the drawdown of the reservoir occurs rather slowly.

The dependence of the water reservoir surface area on the headwater reservoir level WRL within elevations from 175.0 to 195.0 miss well described by the following equation:

$$F = 0.27037 \cdot (WRL - 136)^{2.144} \text{ km}^2 \qquad (6)$$

and water reservoir volume V, in this case, is described by the following equation

$$V = 85996 \cdot H^{3.144} + C \text{ m}^3 \qquad (7)$$

In the interval between headwater reservoir level WRL elevations 185.1 to 185.0 m, the water reservoir volume will be

$$\Delta V = 85996 \cdot (49.1^{3.144} - 49.0^{3.144}) = 113.94 \text{ million m}^3$$

The volume of 113.94 million m^3 can be filled in 6 hours with a flow rate $Q = {113.9 \times 10^6}/{(6 \times 3600)} = 5275\, {m^3}/{s}$, which practically corresponds to the design capacity of spillway No.2 for the option of spillway No. 2 used as the main spillway structure.

The rate of rise of the headwaters reservoir level WRL at this flow rate, as in the case of an up-level during wind surge, will be 100/6=17 mm/hour.

Therefore, if after 6 hours, the level rise (or fall) stops, it will indicate a surge of water due to wind surge; if the level rise continues, then this will indicate an increase in flood.

For example, at a flood flow rate approximately equal to the throughput of two operating HPP units Q =1100 m^3/s, the rate of rise of the headwaters reservoir level WRL will be:

$$V_z = \frac{1100 \cdot 3600 \cdot 1000}{1130 \cdot 10^6} = 3.5 \frac{mm}{hour}$$

So, the rate of the headwaters reservoir level WRL rise in front of the structures can be used to determine the cause of both the water reservoir level rise and its drawdown.

3 Results and Discussion

Calculations of the variant of the culvert an opening of spillway No. 2, operating according to the scheme of "outflow from an opening in a thick wall" during the period of temporary operation, were performed for a structure in which the ceiling of the resulting outlet is mated with the pressure face of the dam by an elliptical insert outlined by the following equation.

$$\frac{x^2}{(A-x)^2} + \frac{z^2}{(B-z)^2} = 1 \qquad (8)$$

where from we obtain

$$z = B \cdot (1 - \sqrt{1 - \left(1 - \frac{x}{A}\right)^2} \qquad (9)$$

where: x is the distance from the inlet section; z is the distance from the top of the exit section.

Let us denote the height of the opening at the outlet as a. Let us take the hole height at the outlet equal to the thickness of the compressed jet when it flows out from under the shutter. With perfect jet compression, we can take the coefficient of vertical compression of the ε = 0.61. In this case, the height of the opening at the inlet is $A = {a}/{0/61} = 1/64a$, and the top of the inlet opening will be higher than the top of the outlet by $B = (1 - \varepsilon)A$.

$$z = (1-\varepsilon)\cdot A\cdot(1-\sqrt{1-\left(1-\frac{x}{A}\right)^2} = \frac{1-\varepsilon}{\varepsilon}\cdot a\cdot(1-\sqrt{1-\left(1-\frac{\varepsilon\cdot x}{a}\right)^2} =$$
$$= 0.64\cdot a\cdot(1-\sqrt{1-\left(1-\frac{0.61\cdot x}{a}\right)^2} \quad (10)$$

For such a design of the inlet section of the water discharge opening, the flow rate can be determined according to the dependence for determining the flow rate when flowing out from under the gate:

$$Q = \mu\cdot A\cdot b\cdot\sqrt{2g\cdot H} \quad (11)$$

The flow rate coefficient μ can be determined according to [5]:

$$\mu = 0.615 - 0.203\cdot\frac{A}{H} = 0.615 - 0.333\cdot\frac{a}{H} \quad (12)$$

For the adopted design of the inlet A=1.64a, we obtain:

$$Q = (0.615 - 0.333\frac{a}{H})\cdot 1.64\cdot a\cdot b\cdot\sqrt{2g\cdot H} = (1.01 - 0.546\frac{a}{H})\cdot a\cdot b\cdot\sqrt{2g\cdot H} \quad (13)$$

The effect of gate slot structures and energy losses on friction can be estimated by the flow rate coefficient $\varphi = 0.95$; with this in mind, we obtain the final formula to determine the flow rate of one span depending on the height of the water discharge opening and head H:

$$Q = 22.98\cdot(1.85 - \frac{a}{H})\cdot a\cdot\sqrt{H} \quad (14)$$

Using dependence (14), the total flow rate was calculated for all water discharge openings of spillway No. 2 in the head range from 19 to 10.5 m. The results of these calculations are graphically provided in Figure 6 as curves for spillway No. 2 capacity operating in "the expiration from under the gate" mode.

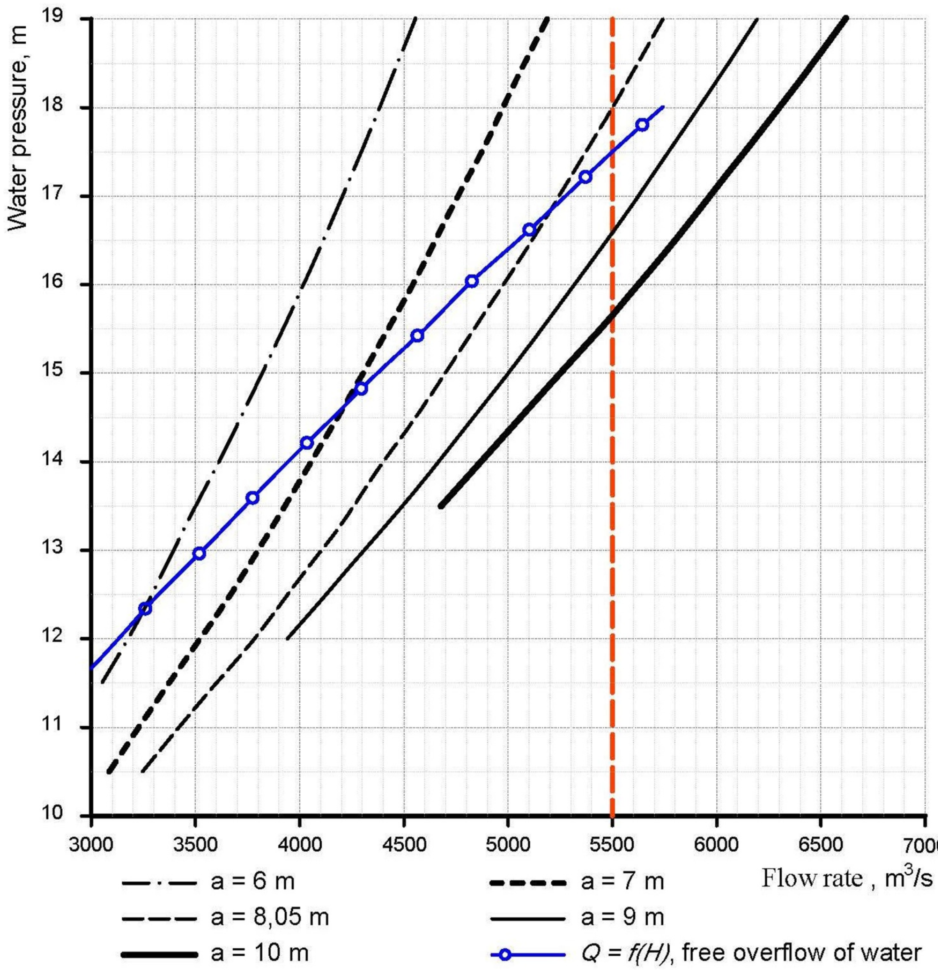


Fig. 6. Dependence of spillway No. 2 capacity on head and height of water discharge opening

The graphs of Figure 6 show the line of the capacity of the culverts for the design option with a free overflow of water at the elevation of the bottom of the culvert of 167.0 m, as well as the line corresponding to the design capacity of Q=5500 m^3/s.

As the curves of Figure 6 show, the required flow rate in the free water overflow mode will be ensured with the design head Hfree=17.5 m and in the pressure mode with a minimum opening height a=8.05 m with the head relative to the threshold of the dam Hforced=18.0 m. The head of 18.0 m is the maximum head that can be ensured with the design Reservoir water level (RWL) El.185.0 m during the temporary water reservoir operation.

Figure 7 shows the dependence of the required head on the water discharge opening height *H=f(a)* and dependence of the water discharge opening height on the head *a=f(H)* to pass the estimated flow rate Q_{calcul} =5500 m^3/s.

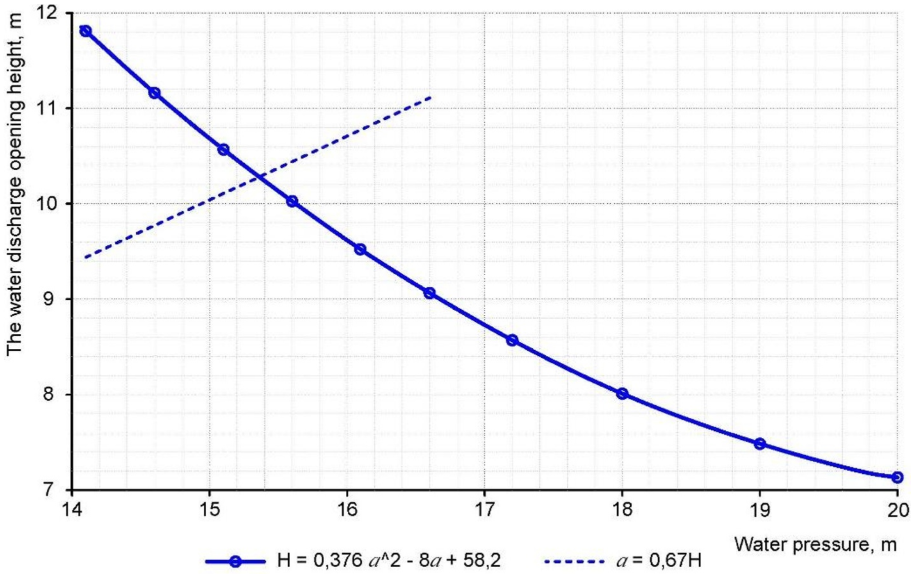


Fig.7. Dependence of required water discharge opening height $a = f(H)$ to skip flow rate of 5500 m^3/s in pressure mode

Figure 7 shows the line of the maximum height of the water discharge opening a = 0.67·H, at which the water discharge mode is maintained under pressure.

As seen from the graphs in Figure. 7, setting the water discharge opening height and the pressure at the threshold of spillway No 2 weir has not a single-value solution. Gate parameters are the determining factors in assigning the parameters of the water discharge openings.

3.1 Skipping through the spillway No. 2 of the flow rate less than the calculated

Skipping the flow rates less than the calculated can be done using two modes of operation of the water discharge openings.

The first mode has periodic activation of the required number of spans for maximum throughput. At the same time, it is assumed that the capacity of the operating spillways is greater than the inflow discharge, which predetermines the decreased volume of the reservoir and a fall and a drop in the water level upstream. The operating time of the water discharge openings, in this case, is determined by the permissible value of the decrease in the water level upstream. Working in this mode significantly simplifies the operation of spillway No 2; however, it has certain difficulties downstream of the hydroelectric complex. The capacity of one span of spillway No. 2 exceeds 1000 m^3/s, and change in the Angara River flow rates by this value will occur during lifting or lowering the gate within tens of minutes.

The second mode, intermediate flow rate skipping mode, is the flow rate skipping in the "flowing out from under the gate" mode. In this case, it is necessary to constantly adjust the opening of the gates to ensure that the set water level is maintained upstream. In this case, the opening of the gates should be constantly adjusted to ensure that the specified water level in the upstream water is maintained. Considering the impact on the downstream flow regime, this method of flow regulation is the most preferred. However, from the point of view of the operation of cranes and hydro-mechanical equipment, this mode is much more complicated.

In this mode, the required value of the gate opening height is determined by the formula (14) to determine the flow rate; it is flowing from under the gate.

Comparing these options for maneuvering the gates, the option with the full opening of the gates has undeniable advantages in terms of the operation of mechanical equipment. There are the following considerations regarding the danger of creating unsteady regimes downstream of the hydroelectric complex. The turbine's regulation time does not exceed 5 to 8 seconds or 0.1 minutes, with an increase in flow rate from the idling of the units to the full opening of the turbine guide apparatus. This flow rate is approximately $0.9 \cdot Q_{calcul} = 495$ m^3/s. In this case, the intensity of the flow rate increase downstream is approximately 495/0.1=4950 m^3/min. The time for lifting the gate, required to ensure the full throughput of the span, is in the range of 8 to 10 minutes. At this gate lifting speed, the intensity of the flow rate increase downstream will be approximately 1000/10=100 m^3/min, which is almost 50 times less than the intensity when regulating the turbines.

Therefore, from this point of view, there are no restrictions on the isolated work of the culvert spans of spillway No. 2.

3.2 Study of free-falling stream jets parameters

The main purpose for studying the stream parameters at the downstream apron is to determine the depth and speed of the outlet flow, which ultimately determines the degree of river bed deformation downstream and its danger to the spillway end part stability.

Further calculations are made for skipping the maximum construction flow rate of 5500 m^3/s.

The flow parameters in the compressed cross-section in the area where the jet falls on the spillway surface of the spillway determine the nature of the conjugation of flows and the hydraulic situation downstream of spillway No. 2. In the constructions variant of the spillway with stilling basin, a hydraulic jump will occur, the state of which will be determined by the ratio of the value of the second conjugate depth and the depth of the stilling basin. In the construction variant of the spillway with water in the throw jet, a backwater curve will be formed at the water break, which will determine the distance of the jet discharge downstream.

3.3 Study of the spillway option no. 2 with pressure mode of skipping construction flow rates:

The flow depth at the outlet of the water discharge opening depends on the length of the free flow section and the opening height. Taking into account the relatively small distance from the outlet section of the opening to the section of the dam threshold conjugation with the inclined spillway edge, the energy losses can be calculated according to the steady motion formulas:

In this case, we have:

$$h_l = \lambda \cdot \frac{l}{4R} \cdot \frac{V^2}{2g} \tag{15}$$

where h_l is the losses of friction along the length behind the outlet section of the hole; $\lambda = 0.11 \cdot \sqrt[4]{\frac{\Delta}{4R}}$ is the Darcy coefficient; $R = \frac{a \cdot b}{a+2b}$ is the hydraulic radius; V is the flow velocity at the outlet of the water intake opening.

For the variant with the stilling basin l = 38 - 15 = 23 m, and the variant with a jet discharge by a toe-springboard l = 41 - 15 = 26 m.

The definition of the losses of friction along the length is made in Table 1.

As seen from Table No 1, the energy loss along the length of the dam threshold behind the outlet pressure opening is negligible compared to the flow energy, so the flow depth in the outlet section can be considered equal to the height of the water intake opening.

For the variant with a pressurized water outlet of the water discharge opening, the jet's parameters formed in the flight behind the outlet section after the free flow section were calculated using the same method as for the variant with a free overflow.

Table 1. Determination of losses along the length of the threshold

Hole height a (m)	Speed V (m/s)	Hydraulic radius R (m)	λ	Length loss h_l	
				l = 23 m	l = 26 m
7	15.71	2.92	0.0111	0.275	0.311
8.05	13.66	3.08	0.0109	0.194	0.219
9	12.22	3.21	0.0108	0.147	0.166
10	11.00	3.33	0.0107	0.114	0.129

Figure 8 shows the jet for the option with stilling basin, while Figure 9 shows the jet for the variant spillway with a jet discharge by a toe-springboard.

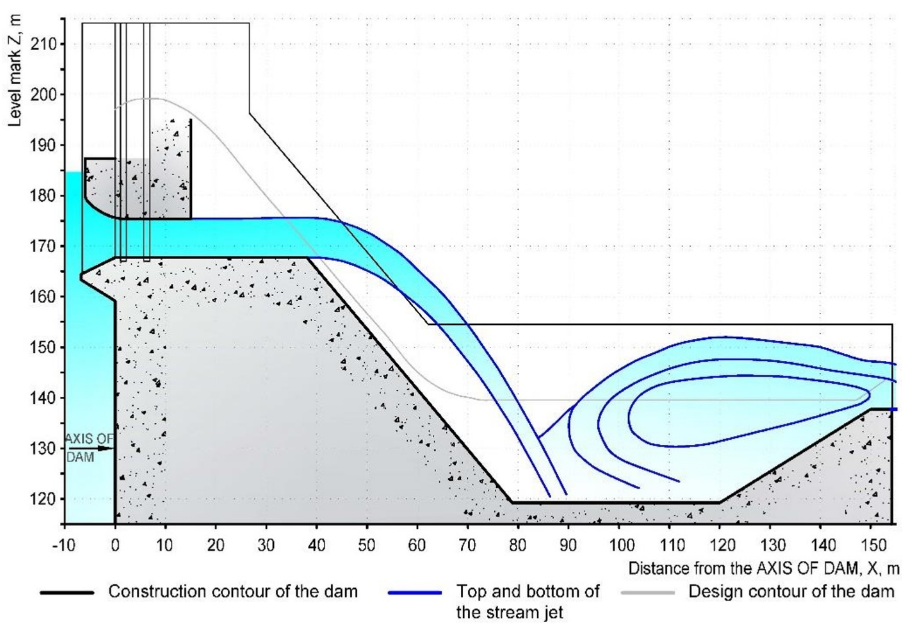


Fig. 8. Longitudinal profile of spillway No. 2 for construction period with pressure passage of water out from opening of spillway and energy dissipation in stilling basin.

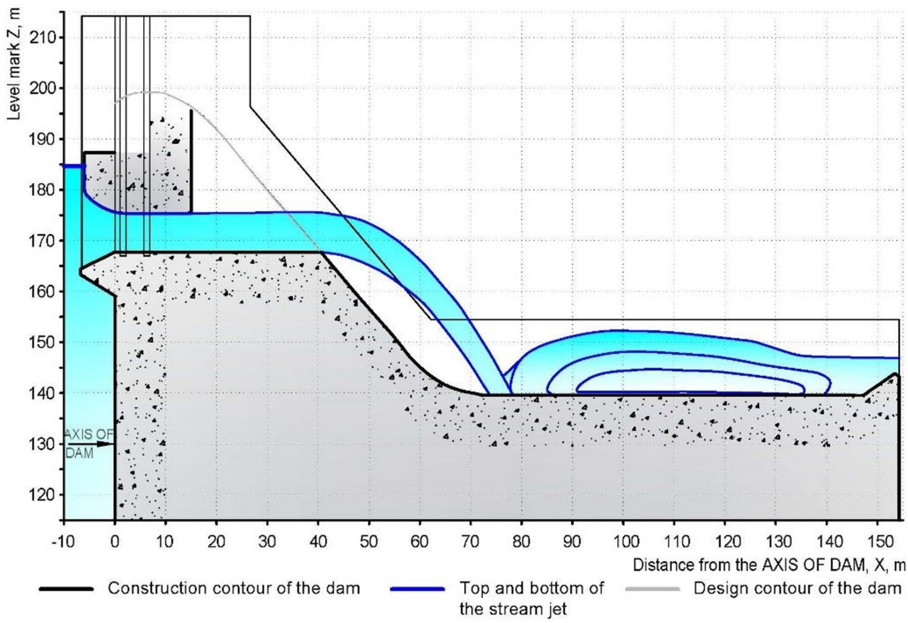


Fig. 9. Longitudinal profile of spillway No. 2 for construction period with flow discarded and with pressure passage of water out from opening of spillway.

In the variant with the stilling basin, the water jet falls on the discharge surface at a distance of 87.5 m.

In the variant with the flow discarded, the water jet falls on the discharge surface at a distance of 75 m.

As these figures show, in the variant of spillway No 2 with a pressure outflow, there is a need to increase the height of the sidewalls of the spillway toward the downstream to 9...10 m in the range of the marks range from 176.0 to 154.5 m, i.e., 3 meters more than for the option with free water overflow.

4 Conclusions and Recommendations

1. The capacity of spillway No. 2, operating in pressure flow mode according to a short nozzle scheme, has been determined. The throughput depending on the headwater can be determined using formula (14) and Figure 6.

2. The dependence of the water discharge opening height by the flow head for a variant of skipping of the flow rates for the construction period is obtained (see Figure 7).

3. The outlines of free-falling jets behind the water discharge openings, operating in free overflow mode and pressure flow mode, have been obtained that enables specification of the contours of the spillway side walls. The sidewalls of the spillway must be at least 15.0 meters high when compared to the spillway surface.

The article's support

The article was prepared with the support of the Russian Ministry of Education and Science under agreement No. 075-15-2021-032 dated March 23, 2021 on the provision of a grant in the form of a subsidy for the creation and development of an engineering center based on an educational organization of higher education and/or a scientific organization within the framework of the federal project "Development of infrastructure for scientific research and training" of the national project "Science and Universities".

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