doi: 10.18720/MCE.76.4

The simulation models of river flow management by a system of flood control facilities

Имитационные модели регулирования речного стока системой противопаводковых гидроузлов

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Key words: hydrotechnical construction; mathematical simulation; flood control; hydro facilities; river flow management; environmental protection

Ключевые слова: гидротехническое строительство; математическое моделирование; защита от наводнений; гидроузлы; управление речным стоком; охрана окружающей среды

Abstract. The authors propose the simulation models of river flow management during extreme river discharges for a hydro complex with a hydroelectric power plant (HPP) on a main river and a flood control facility on its side tributary, including the case of their joint operation as part of a system of flood control facilities distributed on a drainage basin. The possibility of applying the previous years' water-management plans for the choice of flood control facilities locations is considered. In the mathematical model of a hydro complex, operating modes of a HPP are assigned using its reservoir operating rule curves, considering the requirements of safety and environmental protection. In the mathematical model of a flood control facility, the scheme for flood discharge through uncontrolled bottom spillways and an uncontrolled surface spillway is considered. The use of the models makes possible to determine the operating modes of hydro facilities, considering the modern economic and environmental requirements, revision of their parameters, estimation of the energy-economic and environmental effects from the creation of systems of flood control facilities distributed on drainage basins.

Аннотация. В статье предложены имитационные модели регулирования речного стока при экстремальных расходах для комплексного гидроузла с ГЭС на основной реке, противопаводкового гидроузла на боковом притоке, в том числе для случая их совместного функционирования в составе системы распределенных на водосборе гидроузлов. Рассмотрена возможность применения проектных проработок прошлых лет по схемам использования гидроэнергетических ресурсов речного бассейна при выборе створов гидроузлов. В математической модели комплексного гидроузла режимы работы ГЭС назначаются с использованием диспетчерских графиков с учетом требований безопасности и охраны окружающей среды. В математической модели противопаводкового гидроузела рассмотрена схема пропуска паводка через нерегулируемые донные водопропускные сооружения и нерегулируемый поверхностный водослив. Использование моделей позволяет определить режимы работы гидроузлов, с учетом современных хозяйственных и природоохранных требований провести корректировку их параметров, оценить энерго-экономический и экологический эффект от создания распределенной на водосборе системы гидроузлов.

Федоров М.П., Чусов А.Н., Масликов В.И., Молодцов Д.В., Того И. Имитационные модели регулирования речного стока системой распределенных на водосборе противопаводковых гидроузлов // Инженерно-строительный журнал. 2017. № 8(76). С. 40–49.

Introduction

Flood control has always been an important task for many countries, including Russia, as floods cover large areas, lead to many casualties and huge economic losses. It can be noted that, due to the global climate change, the intensity of floods in many regions of the world is increasing. This requires improving the methods of flood control [1–6].

The task of flood management through the creation of a system of flood control facilities distributed on a drainage basin was considered in the article [7]. First, it is advisable to analyze the previous years' water-management plans for the choice of flood control facilities locations and the assessment of their parameters, since they include all the necessary justifications for hydro complexes with hydroelectric power plants. Usually these materials have not lost their significance even now. But it is necessary to revise the flood storage of the main river's reservoir, since considering the current and prospective economic and environmental conditions in its upper pool during flood flows accumulation, it may be necessary to reduce the designed maximum water level. If, at the same time, the reduction of extreme water flow to the maximum allowable value is not ensured in the lower pool, then to reduce the flow entering to the main river's reservoir, the missing volume is redistributed into flood control facilities on side tributaries. Thus, the system of hydro facilities of different functional purposes distributed on the drainage basin will be created (Figure 1).

The goals of this work are revision of flood storage of a reservoir of a hydro complex with a HPP on a main river, determination of flood control effects from flood control facilities on its side tributaries, justification of the structure of the system of flood control facilities distributed on the drainage basin.

The following simulation models were developed for these goals:

- the mathematical model of operating modes of a hydro complex with a HPP;
- the mathematical model of extreme flows management by flood control facilities;
- the integrated model of river flow management by a system of flood control facilities distributed on a drainage basin.

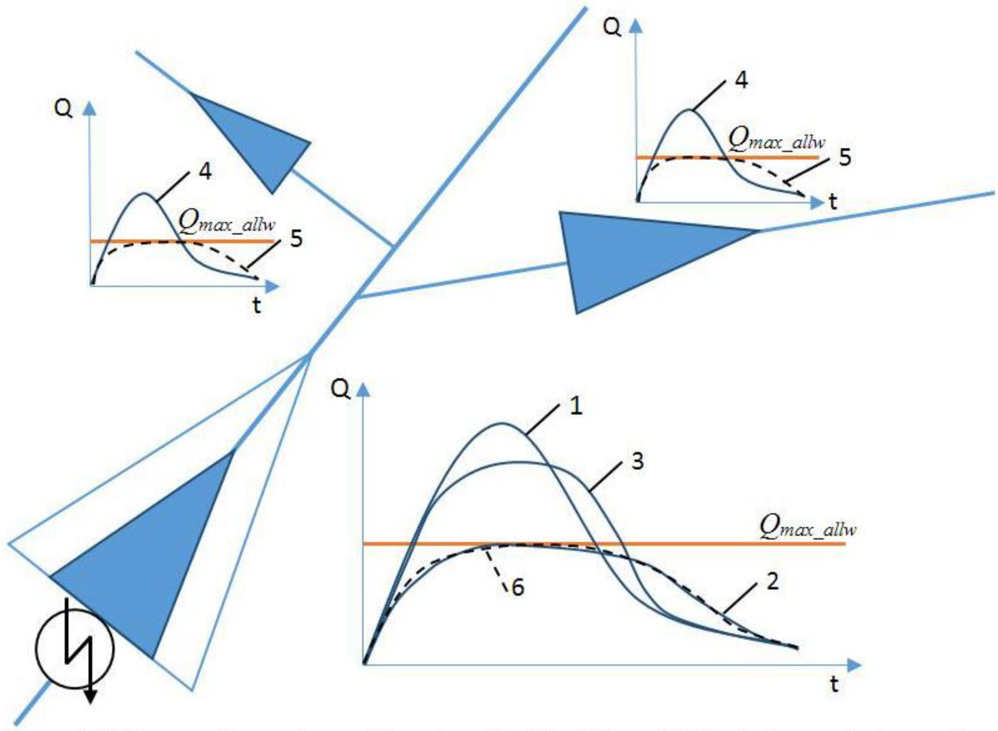

Figure 1. Scheme of a system of flood control facilities distributed on a drainage basin

where:

1 – conditional hydrograph of extreme natural water flow with 1% probability of the main river,

2 – conditional hydrograph of managed water flow in the lower pool of the hydro complex with the designed maximum water level,

3 – conditional hydrograph of managed water flow in the lower pool of the hydro complex with the reduced maximum water level considering the current and prospective environmental and economic conditions,

4 – conditional hydrograph of extreme natural water flow with 1% probability of the *i*-th side tributary,

5 – conditional hydrograph of managed water flow in the lower pool of the flood control facility of the *i*-th side tributary which accepts the missing volume for flood control from the main reservoir,

6 – conditional hydrograph of managed water flow in the lower pool of the hydro complex under the reducing extreme water flow on the side tributaries,

Q_{max_allw} – the maximum allowable river flow considering the current and prospective environmental and economic conditions.

Methods

The mathematical model of operating modes of a hydro complex with HPP

The mathematical model of long-term operation modes of a hydro complex with a HPP is created in accordance with the current recommendations [8] regulating the operation modes of water reservoirs and the use of their water resources, and which are based on the methodology of the integrated approach to river flow management [9]. These recommendations are based on Russian and foreign experience of water management.

As a research object, a hydro complex with a reservoir of annual flow regulation as the most common was chosen [10]. In earlier researches [11], the task of estimating the influence of the creation on a side tributary the flood control facility on the operating regimes and parameters of a hydro complex with a HPP on the main river was considered. In this paper another task of revision of flood storages of HPPs reservoirs (as a rule, in the direction of reducing) is solving, considering the current and prospective economic and environmental conditions and redistributing the missing regulating volume to flood control facilities on side tributaries.

When modeling HPP operating modes, the designed reservoir operating rule curves are applied, that should be periodically revised [12, 13] considering the changing economic and hydrological conditions over time, but the previous researches [14, 15] have shown that in modern conditions, as a rule, existing designed reservoir operating rule curves allow operational services to operate hydropower plants relatively safely.

The river flow management includes the following stages:

- the period of reservoir filling;
- the period of accumulation of flood flows;
- the period of reservoir draw-off.

Stage 1. The period of reservoir filling

Calculations should be made for a high-water year of estimated probability.

Case 1. The water level mark in the upper pool $Z_{UP}^{HPP}(t)$ is above the upper limit of water surface level on the reservoir operating rule curves in the range:

$$DSL \le Z_{UP}^{HPP}(t) \le FRL,$$

where: *DSL* – Dead Storage Level; *FRL* – Full Reservoir Level.

Depending on the water head *H(t)* at the calculated time (*t*), the HPP operates with the power of $N^{HPP}(t)$:

$$N^{HPP}(t) = \begin{cases} N_A^{HPP}(t), & H(t) \le H_R \\ N_R^{HPP}, & H(t) > H_R \end{cases}$$

$$H(t) = Z_{UP}^{HPP}(t) - Z_{LP}^{HPP}(t) - \Delta H,$$

where:

$N_A^{HPP}(t)$ – available power. It is determined using the operating curve $N_A^{HPP}(t) = f_1\big(H(t)\big)$.

N_R^{HPP} – rated power.

$Z_{LP}^{HPP}(t)$ – water level mark in the lower pool.

ΔH – head loss.

H_R – rated head.

According to the conditions of reliability and safety of hydro facilities operation, it is provided the restriction on the rise rate of the water level in the reservoir:

$$\frac{dZ_{UP}^{HPP}(t)}{dt} \leq h,$$

where h – the maximum safe value of the rise of the water level in the reservoir per day.

The water discharge in the lower pool at the time (t) is:

$$Q_{LP}^{HPP}(t) = Q_T^{HPP}(t) + Q_S(t),$$

where:

$Q_T^{HPP}(t)$ – water discharge through the hydro turbines.

$Q_S(t)$ – seepage discharge.

Accordingly, the power of the hydropower plant is determined by:

$$N^{HPP}(t) = k_N * H(t) * Q_T^{HPP}(t),$$
$$k_N = 9.81 \cdot \eta_T \cdot \eta_G,$$

where k_N – correction factor, considering the turbine efficiency (η_T) and the generator efficiency (η_G).

Electricity production is:

$$E(t) = \int_0^{T_1} N^{HPP}(t)dt,$$

where T_1 – estimated time interval.

The volume of water in the reservoir at time (t):

$$V(t) = V(t-1) + \left[Q_{ENT}(t) - \left(Q_T^{HPP}(t) + Q_S(t) + Q_{ID}(t) + Q_{EV}(t) + Q_{EN}(t)\right)\right] \cdot T_1$$

where:

$Q_{ENT}(t)$ – natural water flow, entering the reservoir.

$Q_{ID}(t)$ – idle discharge through the spillways.

$Q_{EV}(t)$ – loss of water due to evaporation from the surface of the reservoir.

$Q_{EN}(t)$ – the water, taken from the upper pool of the hydropower plant for economic needs.

$V(t-1)$ – the volume of water in the reservoir at a previous point in time.

Case 2. The water level mark in the upper pool is below the upper limit of water surface level on the reservoir operating rule curves within:

$$Z_{UP}^{HPP}(t) \leq FRL \text{ and } N^{HPP}(t) \leq N_R^{HPP}.$$

The spillways are closed:

$$n_{SW}(t) = 0,$$

where $n_{SW}(t)$ – the number of open spillways.

The water flow in the lower pool is determined by the requirements of water consumers:

$$Q_{LP}^{HPP}(t) = Q_T^{HPP}(t) + Q_S(t) \cong Q_{WC}(t),$$

where $Q_{WC}(t)$ – water flow according to the requirements of water consumers.

Stage 2. The period of accumulation of flood flows

The water level mark in the upper pool is in the range:

$$FRL \leq Z_{UP}^{HPP}(t) \leq MWL,$$

where *MWL* – Maximum Water Level.

The main requirements for managing flood flows during accumulating extreme flows with low probability (1%):

- The water discharge in the lower pool of the HPP should not exceed the maximum allowable Q_{max_allw} (it is assumed to be equal to the peak natural flow with probability of 10%), providing safety requirements for economic activities. In this case, the flood-alluvial regime of the river is also preserved:

$$Q_{LP}^{HPP}(t) \leq Q_{max\_allw}$$

- The water level mark in the upper pool must not exceed the *MWL*:

$$Z_{UP}^{HPP}(t) \leq MWL.$$

The additional requirements are minimization of socio-economic damage and conservation of biodiversity of ecosystems in the area upstream of the HPP. For this purpose, areas, types of flooded lands and duration of standing water etc. should be analyzed. Using the criteria of not decreasing the diversity and the ratio between anthropogenic and natural ecosystems [16,17], the MWL is corrected. When it decreases, the flood storage ΔV is redistributed into flood control facilities on side tributaries:

$$\Delta V = V_{MWL} - V_{MWL*},$$

where:

V_{MWL} – designed maximum volume of the reservoir.

V_{MWL*} – revised maximum volume of the reservoir in accordance with environmental requirements.

The number of open spillways is determined in accordance with the spillways operating rule curve, depending on the actual water level (*z*). In this case, each spillway is operating in the "full opening" mode.

$0 \leq n_{SW}(t) \leq n_1$ spillways are opened when $FRL = z_1 \leq Z_{UP}^{HPP}(t) \leq z_2$.

$n_2 \leq n_{SW}(t) \leq n_3$ spillways are opened when $z_2 \leq Z_{UP}^{HPP}(t) \leq z_3$.

…

$n_{x-1} \leq n_{SW}(t) \leq n_x$ spillways are opened when $z_{m-1} \leq Z_{UP}^{HPP}(t) \leq z_m = MWL$.

The additional requirement is the restriction on the frequency of opening/closing operations of the spillways – the minimum time between opening and closing according to ensuring reliability of a hydro complex under operating conditions.

Stage 3. The period of reservoir draw-off

$$DSL \leq Z_{UP}^{HPP}(t) \leq FRL$$

Case 1. The water level mark in the upper pool is above the upper limit of water surface level.

The HPP is operating with the rated power.

$$N^{HPP}(t) = N_R^{HPP}$$

Case 2. The water level mark in the upper pool is below the upper limit of water surface level.

The HPP is operating with the guaranteed power.

$$N^{HPP}(t) = N_G^{HPP},$$

where N_G^{HPP} – guaranteed power of the HPP with the rated probability.

The water discharge through the turbines is:

$$Q_T^{HPP}(t) = Q_G^{HPP},$$

where Q_G^{HPP} – guaranteed water discharge with rated probability.

The reservoir is drawing-off to the dead storage level in winter.

Case 3. The water level mark in the upper pool is below the lower limit of water surface level.

The HPP is operating with the reduced power.

$$N^{HPP}(t) = p \cdot N_G^{HPP},$$

where $p < 1$ – reduction ratio of the guaranteed power.

The mathematical model of extreme flows management by flood control facilities

It is noted in the researches [18,19], that flood control facilities with uncontrolled bottom spillways and an uncontrolled surface spillway, characterized by relative reliability and safety in operation, are often used for the flood flow management on side tributaries. The volume of flood water in a reservoir on a side tributary $V^{st}(t)$ at time (t) is determined using the following equation:

$$V^{st}(t) = V^{st}(t-1) + (Q_{ent}^{st}(t) - Q_{reg}^{st}(t) - Q_{ev}^{st}(t) - Q_s^{st}(t)) \cdot T_2,$$

where:

$Q_{ent}^{st}(t)$ – natural water flow with rated probability, entering the reservoir.

$Q_{reg}^{st}(t)$ – regulated water flow in the downstream of the flood control facility.

$Q_{ev}^{st}(t)$ – loss of water due to evaporation from the surface of the reservoir.

$Q_s^{st}(t)$ – seepage discharge.

T_2 – estimated time interval.

$V^{st}(t-1)$ – the volume of flood water in a reservoir on a side tributary at a previous point in time.

The main requirements for flood flows management on side tributaries are:

- The reduction of extreme water discharges from values with probability of 1 % to values with probability of 10 % in downstream of a flood control facility. This protects the flood-alluvial regime of the river.
- Not exceed the maximum allowable level of the upper pool (Z_{max_allw}) to minimize the area of land flooding and preserve biodiversity of ecosystems [16].

$$Z_{up}^{st}(t) \leq Z_{max\_allw},$$

where $Z_{up}^{st}(t)$ – The water level mark in the upper pool of the flood control facility on the side tributary.

At the initial stage of hydraulic calculations, the design parameters of the flood control facility are taken as the main ones: the accumulating volume, the water level marks, the number of bottom and surface spillways, their sizes, etc.

The operating mode of the flood control facility is determining for the following cases:

1. The low-flow period.

The water level mark in the upper pool is in the range:

$$Z_1 < Z_{up}^{st}(t) \leq Z_1 + a,$$

where:

Z_1 – bottom level mark of the uncontrolled bottom spillways.

a – height of the uncontrolled bottom spillways.

The bottom spillways operate in not drowned mode. The equation for water flow through a spillway with a wide crest is used to calculate the water-carrying capacity of the flood control facility [20] (the water-level in the lower pool is not considered).

2. The high-water period.

2.1. The water level mark in the upper pool is in the range:

$$Z_1 + a < Z_{up}^{st}(t) \leq Z_2,$$

where Z_2 – bottom level mark of the uncontrolled surface spillway.

The bottom spillways are operated in the drowned mode with variable pressure [20].

2.2. The water level mark in the upper pool is in the range:

$$Z_2 < Z_{up}^{st}(t) < Z_{max},$$

where Z_{max} – the maximum allowable water level in the upper pool of the flood control facility, considering the safety requirements, minimizing socio-economic damage and preserving the biodiversity of ecosystems.

The bottom spillways are operated in the drowned mode with variable pressure, and the surface spillway is in operated mode. In this case, the water discharge is determined by the water flow through the bottom and surface spillways [20]. The model allows to specify different versions and numbers of spillways (type, size, layout, etc.).

The maximum allowable value of accumulating capacity, the number and sizes of bottom and surface spillways are specified based on the results of calculations.

The integrated model of river flow management by a system of flood control facilities distributed on a drainage basin

The designed flood storage, MWL and the maximum water flow in the lower pool of the HPP should be revised to ensure its projected electricity generation and minimization of economic and environmental damage in the modern and prospective conditions. If they are exceeded, the missing volume (ΔV) is determined for redistribution into flood control facilities on side tributaries. The criterion of availability of the required total flood storage of the hydro complex with the HPP on the main river and flood control facilities on side tributaries: the flow in the lower pool of the HPP must not exceed the maximum allowable discharge (Q_{max_allw}).

$$Q_{LP}^{HPP}(t) \leq Q_{max\_allw}$$

The water volume in the reservoir at time (*t*):

$$V(t) = V(t-1) + \left[Q_{ENT}^{*}(t) - \left(Q_{T}^{HPP}(t) + Q_S(t) + Q_{ID}(t) + Q_{EV}(t) + Q_{EN}(t)\right)\right] \cdot T_1,$$

where:

$Q_{ENT}^{*}(t)$ – natural water flow, entering the reservoir of the HPP, when flood control facilities on side tributaries are operating.

$$Q_{ENT}^{*}(t) = Q_{ENT}(t) \pm \sum_{1}^{k} \Delta Q_i^{st}(t),$$

where:

k – number of flood control facilities on side tributaries on the drainage basin.

$\Delta Q_i^{st}(t)$ – the difference between the natural $Q_{nat\_i}^{st}(t)$ and regulated $Q_{reg\_i}^{st}(t)$ discharges in the downstream of the *i*-th flood control facility on the side tributary:

$$\Delta Q_i^{st}(t) = Q_{nat\_i}^{st}(t) - Q_{reg\_i}^{st}(t).$$

The calculations should be carried out considering the delay for travel water flows from a flood control facility to the HPP reservoir.

In river flow management, it is necessary to consider possible asynchrony and locality of rain precipitation in a catchment area [21], that lead to a mismatch in time of maximum water flow on a main river and its side tributaries. It is necessary to perform an analysis of possible combinations of rain-water discharges with different probability in the side tributaries, the effect of their accumulation in flood control facilities in the form of reducing extreme water discharges in downstream side and the corresponding decrease in the flow entering to the HPP reservoir on the main river.

Results and Discussion

At the first stage, using the simulation models, the regulating effect of joint operation of the hydro complex with each flood control facility on side tributaries is estimated. The estimated frequency of the natural flood flow is accepted for hydro complex on the main river and in sequence for each of flood control facilities on side tributaries. The assessment of the operating modes of each flood control facility and the revision of the water flow value entering the reservoir on the main river are carried out. Then the indicators of electricity generation of the HPP, the area of land flooding and the environmental and economic effect should be determined. This makes it possible to determine the main flood control facility (with the maximum regulating effect) from the many variants possible for construction, as well as to make the ranking of them on this basis. Only flood control facilities with positive ecological and economic effect should be selected for the further consideration.

The possibility of distribution of the flood storage to a chain of flood control facilities on each of the side tributaries should be considered.

At the next stage, the regulating effect of the joint operation of the hydro complex on the main river with the selected ones is estimated, for which hydrographs of the flood discharges with different probabilities are used, reflecting asynchrony and locality of rain precipitation on the catchment area.

Test calculations of parameters for one of the representative flood control facilities were performed and showed that its regulated water discharge in the lower pool can be reduced by 30 % compared to the natural with probability of 1% (corresponds to the natural water discharge with probability of 1 %). At the same time, the extreme water discharge in the main river entering the reservoir of the HPP is reduced by approximately 10%. Accordingly, the volume of accumulation of extreme water flow is reduced by ~11%, and the flooding area in the upper pool is reduced by ~11.5 %.

A number of works, for example [22, 23], is devoted to the substantiation of parameters and operating modes of self-regulating flood control facilities on side tributaries, successful experience of their usage for protecting lands from floods by reducing extreme water discharges. The models considered in this article show a similar effect from the use of self-regulating flood control facilities on side tributaries, but they are also aimed at solving a new problem of use of hydro potential of a main river in conditions of necessary revision of a HPP's designed MWL and, accordingly, the flood storage due to changed economic and environmental conditions in the lower and upper pools using flood control facilities on side tributaries.

Conclusions

In this paper, the developed simulation mathematical models of the operation modes and parameters of a system of flood control facilities distributed on a drainage basin are presented. Based on them the algorithm is proposed for solving the task of revision of flood storages of hydro complex reservoirs on main rivers (as a rule, in the direction of reducing), considering the current and prospective economic and environmental conditions. This task achieved by redistributing a missing regulating volume into the self-regulating flood control facilities on side tributaries.

Acknowledgements

The research was supported by Russian Science Foundation (grant №16-17-00050).

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Fedorov M.P., Chusov A.N., Maslikov V.I., Molodtsov D.V., Togo I. The simulation models of river flow management by a system of flood control facilities distributed on a drainage basin. *Magazine of Civil Engineering*. 2017. No. 8. Pp. 40–49. doi: 10.18720/MCE.76.4.