ECOLOGY

UDC 626/627
DOI: 10.15587/1729-4061.2022.252710

DEVISING A PROCEDURE TO CALCULATE AND ANALYZE PARAMETERS FOR PASSING THE FLOOD AND BREAKTHROUGH WAVE TAKING INTO CONSIDERATION THE TOPOGRAPHICAL AND HYDRAULIC RIVERBED IRREGULARITIES

Artur Onyshchenko
Doctor of Technical Sciences, Associate Professor
Department of Bridges and Tunnels*
Borys Ostroverkh
PhD, Senior Researcher, Leading Researcher**
Liudmyla Potapenko
Engineer-Mathematician**
Vitalii Kovalchuk
Corresponding author
Doctor of Technical Sciences, Associate Professor
Department of Bridges and Tunnels*
E-mail: kovalchuk.diit@gmail.com
Oleksndr Tokin
PhD, Associate Professor
Department of Manufacturing, Repair and Materials Engineering*
Mykola Harkusha
PhD, Deputy Director for General Affairs***
Iryna Bashkevych
PhD, Associate Professor
Department of Bridges and Tunnels*
Andrii Koretskyi
PhD, Chief Engineer
Limited Liability Company «Institute Ukrdorproekt»
Chervonotkatska str., 29 a, Kyiv, Ukraine, 02094
Nadiia Khvoshchynska
PhD, Associate Professor, Researcher***
Iryna Rolinska
Engineer***
*National Transport University
Mykhaila Omelianovycha-Pavlenka str., 1, Kyiv, Ukraine, 01010
**Department of Hydrodynamics of Wave and Channel Processes
Institute of Hydromechanics of the National Academy of Sciences of Ukraine
Mariyi Kapnist str., 8/4, Kyiv, Ukraine, 03680
***Limited Liability Company MTZK (LLC MTZK)
Sichovykh Striltsiv str., 77, Kyiv, Ukraine, 04050

It has been established that the most likely period of breakthrough wave occurrence is the time of spring flooding or heavy rain when water-head facilities are subjected to significant loads that lead to the collapse of their individual elements or the entire structure. In addition, the possibility of man-made accidents that can occur at any time cannot be ruled out.

It has been proven that breakthrough wave formation depends on the nature of the destruction or the overflow through a water-head facility. For the study reported in this paper, a model of the kinematics of riverbed and breakthrough flows was used, which is based on the equations of flow, washout, and transport of sediments that are averaged for the depths of the stream. The differential equations describing the nonstationary flow averaged for depth are solved using the numerical grid system FST2DH (2D Depth-averaged Flow and Sediment Transport Model), which implements a finite-element method on the plan of a riverbed's topographic region. These tools are publicly available, which allows their wide application to specific loads and boundary conditions of mathematical models.

The construction of an estimation grid involving the setting of boundary conditions and the use of geoinformation system tools makes it possible to simulate the destruction of a culvert of the pressure circuit and obtain results for a specific case of an actual riverbed and a water-head facility.

It has been established that there is a decrease in the speed of wave propagation along the profile, from 3 m/s to 1 m/s.

The impact of bottom irregularities, the effect of floodplains, and the variety of bottom roughness have also been assessed, compared to the results of their calculation based on one-dimensional models given in the regulatory documents.

Hydraulic calculations were carried out taking into consideration the related properties of the main layer of the floodplain, which consists of peat accumulations, and the heterogeneity of the depths and roughness of floodplain surfaces of soils. It has been established that there is almost no erosion of supports in the floodplain zone in this case.

It was found that as the distance between the flow and breakthrough intersection increases, there is a decrease in the height of the head from 2.1 m to 1.25 m

Keywords: breakthrough wave, topographic and hydraulic heterogeneities, model of riverbed flow kinematics

Received date 05.01.2022
Accepted date 01.02.2022
Published date 25.02.2022

How to Cite: *Onyshchenko, A., Ostroverkh, B., Potapenko, L., Kovalchuk, V., Tokin, O., Harkusha, M., Bashkevych, I., Koretskyi, A., Khvoshchynska, N., Rolinska, I. (2022). Devising a procedure to calculate and analyze parameters for passing the flood and breakthrough wave taking into consideration the topographical and hydraulic riverbed irregularities. Eastern-European Journal of Enterprise Technologies, 1 (10 (115)), 6–16. doi: https://doi.org/10.15587/1729-4061.2022.252710*

1. Introduction

The presence of industrial and transport structures downstream of a reservoir poses a danger in the case of destruction of the pressure circuit of hydraulic engineering structures. The most likely time of breakthrough wave occurrence is the period of spring flooding or heavy rain when water-head facilities are subjected to significant beyond-the-boundary loads that lead to their destruction in whole or individual elements. In addition, the possibility of man-made accidents is not excluded.

Of particular danger is the arrival of catastrophic flooding to the reservoir with a non-capital (earthen) head structure. It is relevant to take into consideration the listed factors during calculations, as well as the effects of the interaction of flows with hydraulic engineering structures.

Factors of the hydrodynamic danger of violation of the state of hydraulic structures for a section of the road and bridge crossing in the lower downstream of a dam include both natural and man-made ones (for example, the destruction of the dam due to a decrease in its strength), as well as other factors.

Destruction (breakthrough) of a hydraulic structure is a multifactorial process that occurs as a result of the action of various forces of nature (earthquakes, storms, floods, heavy rains, and other hydrometeorological factors, even erosion due to concentrated filtration through animal burrows, etc.). It is impossible to exclude the effect of human activities (transport load, large-scale bombing, sabotage), as well as structural defects (poor-quality materials, cracks) and design errors.

Our consideration of the layout of a hydraulic unit in the process of surveys has made it possible to determine that the formation of a breakthrough wave is possible in two variants of the destruction of water-head facilities, namely as a result of overflow and erosion of a water-head ground dam and the destruction of water discharge structures.

The greatest danger, in this case, is the destruction of a culvert in the dam (for example, a water discharge facility).

The study of the state of existing dams in different countries shows that they are not absolutely reliable. Thus, in paper [1], it is indicated that 30 thousand artificial reservoirs in the world, with a total volume of 1 million m³, caused flooding of 0.25 % of the land area.

According to statistics, on average, 1.5 accidents happen per 15 thousand large hydraulic structures annually [1].

In this case, the formation of a breakthrough wave and its propagation along the riverbed under the real conditions of flow and return of flows can have a significant impact.

It should be noted that in the event of a breakthrough wave, it is necessary to take into consideration, in addition to the main parameters of a breakthrough wave, the parameters of wave transformation as well. That is, its passage to a certain distance from the dam, depending on the topographical conditions of the area. To do this, one needs to take into consideration the location plan of the dam and road using high-altitude shooting data. Meeting such requirements would help devise a methodology for the reliable and up-to-date calculation of breakthrough wave parameters.

2. Literature review and problem statement

All hydraulic engineering structures built by people are of particular danger to people's lives and their health. Without proper care, a hydraulic facility wears out and deteriorates. This could lead to its destruction and to an emergency. Climatic changes, the frequency and intensity of maximum precipitation, which significantly affect the work of a hydraulic structure, are analyzed in [2].

Paper [3] analyzes the risk of accidents at hydraulic structures. Attention is focused on the fact that it is necessary to take into consideration the hydrological safety of the structure. Hydrological safety refers to the resistance of a hydraulic structure during its operation to extreme hydrological influences of a natural and man-made nature. Assessment of the state of hydraulic structures is carried out by comparing the actual values of diagnostic indicators with their criterion values, which makes it possible to determine on time the deterioration of the technical condition of the structure and take measures to prevent accidents. In most cases, the cause of an accident is an excess of the discharge flow rate of the structure with the overflow of water through the dam. An important factor is also the wrong choice of the estimation model, which determines the capacity of water discharge structures.

Paper [4] reports the calculation of the probable damage in an accident at hydraulic structures. The initial stage of a hydrodynamic accident is a breakthrough. The breakthrough wave is destructive and even with a small reservoir can cause serious social and economic consequences. The calculation is reduced to determining the parameters of the dynamic interaction of a breakthrough wave with the structure, as well as the parameters of its downstream propagation across the territory. The main parameters of the destructive effect of a breakthrough wave are the speed, height, and depth of the breakthrough, the water temperature, and the time of the breakthrough wave. However, in that calculation, the author's programs with limited access are used.

Study [5] reports a method for analyzing flood risk for large dams based on the concept of general risk factor (TRF), which was originally used in the analysis of the seismic risk of dams. However, that does not make it possible to fully analyze the pass of the flood and breakthrough wave, to assess the degree of impact exerted on this process by various factors.

The calculation of a breakthrough wave's movement and its impact on the formation of the erosion of under-the-bridge structures is carried out in accordance with the regulatory rules for the design of a bridge crossing [6] based on a one-dimensional model. Within the framework of scientific support, it is recommended to make special programs or computing systems according to two-dimensional (in plan) or three-dimensional (spatial) mathematical models.

It should be noted that the use of spatial physical models is illustrative and approximate.

The calculation of the total erosion under bridges is carried out for the projected cross-sections of a riverbed [7] in order to determine, under the estimated conditions, the average depth of a flow under the bridge. This is done by comparing the total erosion coefficient with the permissible one P_{dop} and the hydraulic characteristics of the flow (velocity V, depth H) on the verticals of the sub-bridge section.

Requirements to calculate the general and local erosion, based on the solution to the equation of the balance of sediments on the sections of riverbeds in bridge crossings, are most fully fulfilled with a numerical solution of two-dimensional (planar) hydraulic equations and the balance of sediments [8]. This is usually done on a computer using a set of programs. Paper [9] proposes a simplified methodology for calculating a change in the water level in two reservoirs when passing a flood. The procedure is based on the joint solution of differential equations of water balances in reservoirs without taking into consideration the movement along their length. In article [10], based on field data, calculations of the reservoir's dam burst are carried out. The change in water consumption lengthwise the gate of the dam during the dam break is shown.

However, such calculations, along with significant time spent on the preparation of initial data, can be carried out by highly qualified specialists (usually developers) [11]. Therefore, they are resorted to within the framework of scientific support to projects of the complex for hydraulic calculation of large bridges under complex situational morphological and other conditions.

The proposed approaches [8–10], which involve the numerical methods of modeling, do not meet the real conditions for the propagation of a breakthrough wave. This remark also applies to laboratory modeling of processes in hydraulic trays that use such simplifications.

All this gives grounds to assert that the procedure for calculating the parameters of a wave breaking through a dam has been insufficiently developed and supported only by engineering formulas.

It should be noted that the use of the analyzed methods of calculation only roughly takes into consideration the hydro-morpho-dynamic river conditions and characteristics of soils, the mode of sediments, the degree of compression of the flow in the sub-bridge section. And there is no methodology for calculating the parameters of passing the flood and breakthrough wave through the dam, taking into consideration the topographic and hydraulic irregularities of a riverbed.

3. The aim and objectives of the study

The aim of this work is to devise a methodology for calculating the movement of a breakthrough wave and determining its impact on the erosion of sub-bridge structures, which would make it possible to take into consideration the factors of topographic and hydraulic conditions on a riverbed, which significantly affect the breakthrough wave propagation.

To accomplish the aim, the following tasks have been set:

– to build a model of the kinematics of riverbed and flood flows;

– to investigate the kinematics of the flow during the passage of flood flow rates;

– to study the hydrodynamic parameters of the movement of a breakthrough wave, taking into consideration the topographic and hydraulic conditions on a riverbed.

4. The study materials and methods

We studied the movement of a breakthrough wave and determined its impact on the erosion of sub-bridge structures using an example of specific topographic and bathymetric measurements of a riverbed. The area of modeling (Fig. 1) has a length of 550 m and a width of 300 m.

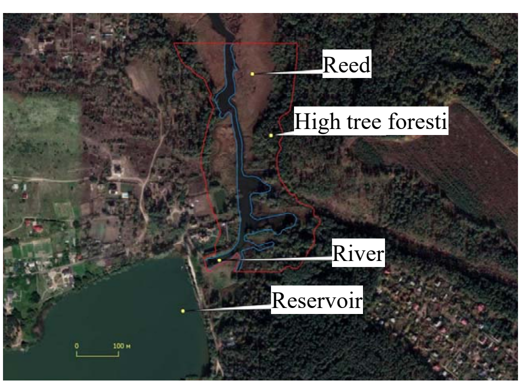

Fig. 1. Location of the estimated zone on the region of calculation of the kinematics of a flood current in the floodplain zone at the high-water level of 144.5 m

The marks of the bottom in the estimated area are obtained from natural measurements. The input data on depths were determined by the levels of the bottom and the level of high water of 144.5 m, corresponding to the flow rate of 117 m^3/s.

When modeling, the depth values were calculated according to the water surface level based on the simulation results.

When modeling, structural elements are taken into consideration – ten pillars of the bridge in the form of rectangular rounded solid columns.

The effect of compression of the flow by bridge supports is taken into consideration by the uniform distribution of the hydrodynamic resistance across the element of the grid containing the support.

In the upper current gate, the set flow rate is Q=117 m^3/s, in the lower gate – the water surface level is z_w=144.6 m.

When modeling, the types of underlying surface in the floodplain of the river were taken into consideration (Fig. 2) based on the geological, geodetic, and field studies of the plant layer. This is taken into consideration by the corresponding roughness coefficients: deep-water riverbed plants – 0.025; floodplain thin wood – 0.10.

The estimation grid of flow modeling is shown in Fig. 2.

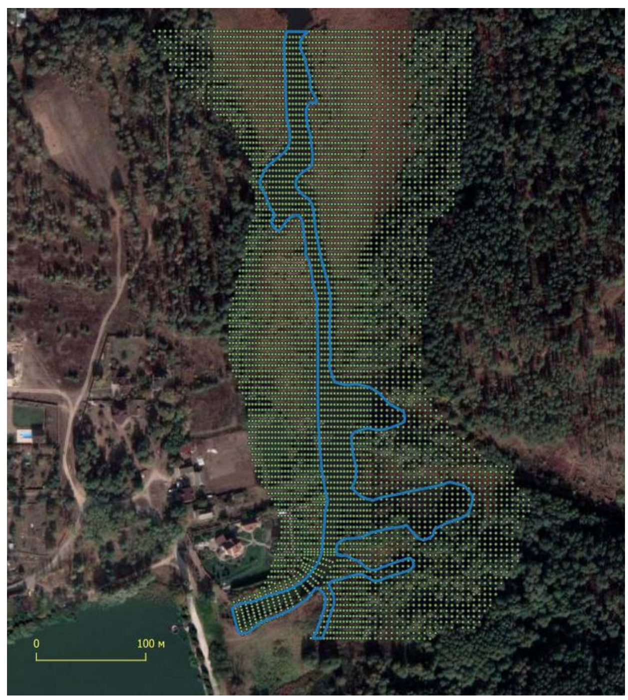

Fig. 2. Estimation flow modeling grid

The elements of the estimation grid are directed along the riverbed and have a size of 3–5 m. The total number of elements in the grid is 6,200.

5. Results of studying the pass parameters of the flood and breakthrough waves taking into consideration the topographic and hydraulic riverbed heterogeneities

5. 1. A model of the kinematics of riverbed and flood flows

The model of the kinematics of riverbed and flood flow is built on the basis of averaging the depth of the flow and the transport of sediments [11]. The differential equations describing the nonstationary flow averaged for depth are solved using the numerical model FST2DH (2D Depth-averaged Flow and Sediment Transport Model) [12], which implements a finite-element method. The steps that are usually taken with the use of FST2DH to study the flow of surface water and the transport of sediments require the general necessary tools to build a grid and assign boundary conditions. Here, it is performed by means of a geoinformation system (publicly available [13]).

The system of estimation equations includes the equations of mass transfer and momentum of flood flows, erosion, and transport of sediments. The calculation scheme is shown in Fig. 3.

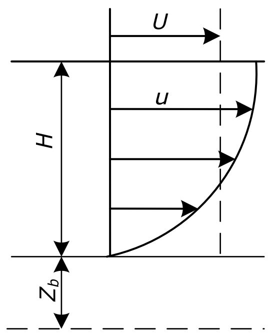

Fig. 3. Scheme for the estimated model of a riverbed flow

The equation of mass transfer, taking into consideration the deformation of the bottom, takes the form

$$\frac{\partial z_w}{\partial t}+\frac{\partial q_1}{\partial x}+\frac{\partial q_2}{\partial y}=q_m, \tag{1}$$

where $z_w=z_b+H$ is the water surface level, z_b is the depth of the erosion zone (determined from the equations of the balance of sediments); q_1, q_2 are the volumetric flow rates of x, y per unit of flow width at velocity $\mathbf{u}$, averaged for depth to U; q_m is the discharge (runoff) per unit area.

The equations of pulse transfer in specific flow rates, bottom shift stresses and stresses caused by turbulence are as follows

$$\frac{\partial q_1}{\partial t}+\frac{\partial}{\partial x}\left(\beta\frac{q_1^{\;2}}{H}+\frac{1}{2}gH^2\right)+\frac{\partial}{\partial y}\left(\beta\frac{q_1q_2}{H}\right)+gH\frac{\partial z_b}{\partial x}+$$
$$+\frac{1}{\rho}\left[\tau_{bx}-\frac{\partial\left(H\tau_{xx}\right)}{\partial x}-\frac{\partial\left(H\tau_{xy}\right)}{\partial y}\right]=0, \tag{2}$$

$$\frac{\partial q_2}{\partial t}+\frac{\partial}{\partial y}\left(\beta\frac{q_2^{\;2}}{H}+\frac{1}{2}gH^2\right)+\frac{\partial}{\partial x}\left(\beta\frac{q_1q_2}{H}\right)+gH\frac{\partial z_b}{\partial y}+$$
$$+\frac{1}{\rho}\left[\tau_{by}-\frac{\partial\left(H\tau_{yx}\right)}{\partial x}-\frac{\partial\left(H\tau_{yy}\right)}{\partial y}\right]=0, \tag{3}$$

where τ_{bx}, τ_{by} are bottom shear stresses, τ_{xx}, τ_{yy}, $\tau_{xy}=\tau_{yx}$ are the stresses caused by flow turbulence; β is the coefficient of correction of the flow pulse, which takes into consideration the change in speed in the vertical direction.

The components of bottom shear stresses are defined as follows

$$\tau_{bx}=\rho c_f m_b\frac{q_1\sqrt{q_1^{\;2}+q_2^{\;2}}}{H^2},$$

$$\tau_{by}=\rho c_f m_b\frac{q_2\sqrt{q_1^{\;2}+q_2^{\;2}}}{H^2}, \tag{4}$$

$$m_b=\sqrt{1+\left(\frac{\partial z_b}{\partial x}\right)^2+\left(\frac{\partial z_b}{\partial y}\right)^2},$$

where $c_f=gn^2/H^{1/3}$ is the bottom friction coefficient, n is the Manning roughness coefficient.

The Manning coefficients [14] and Shazi in the FST2DH model [12] are determined by linear functions of the depth of water. Variations in flow resistance with water depth can occur when short vegetation is submerged and possibly bent by a stream, or when tree branches collide with the flow at high water levels. The corresponding resistance coefficients for natural and constructed channels, as well as for floodplains, can be estimated using studies [6, 15, 16].

However, the coefficients in those references were determined on the basis of one-dimensional approximates of the flow and implicitly take into consideration the effects of turbulence and deviations from the uniform velocity in the cross-section. Little information is available to select coefficients for two-dimensional average depths of flow calculations. The flow resistance factor can be estimated based on existing references and experience. In the upper gate of the current, the flow rates Q are set, in the lower gate – the water surface level z_w.

The mathematical apparatus of research into river processes adopted here makes it possible to calculate in the planar approximation the erosion of the bottom, taking into consideration the interaction with coastal and riverbed hydraulic structures, including local erosion, according to equation (2). However, many problems of river morpho formation should be considered in a spatial three-dimensional approach. As a test calculation, we considered the problem of interaction of the flow with the cylindrical support on the erosion bottom with the help of a suite of solving programs of three-dimensional currents REEF3D [16]. Solving the problem in spatial format (short-term calculation was carried out) showed the possibility of studying turbulent effects in the interaction of flow with structural elements.

5. 2. Calculating the kinematics of the current during the passage of flood flows

Fig. 4–6 show the results of the calculation of the depth, level of the surface of the water, and the flow velocity field after leaving the near-dam water discharge facility during the passage of flood flows.

Fig. 4 demonstrates that the depth of the flood flow across a riverbed varies from 5.5 m to 0.2 m. This is explained by the rapid flow spread along the riverbed.

After leaving the near-dam water discharge facility, the flow velocity changes from 1.35 m/s to 0.33 m/s.

The results of our calculations showed that the kinematics of the current (modules and directions of velocity vectors) significantly depend on the surface plant species and the depths of floodplains (Fig. 2, 3). However, the floodplain and riverbed are wide enough for estimated flow rates and do not create conditions for the significant erosion of the supported platform. Minor erosion depths during the estimated flood period (less than 4 days) are forecast due to the slowness of the erosion.

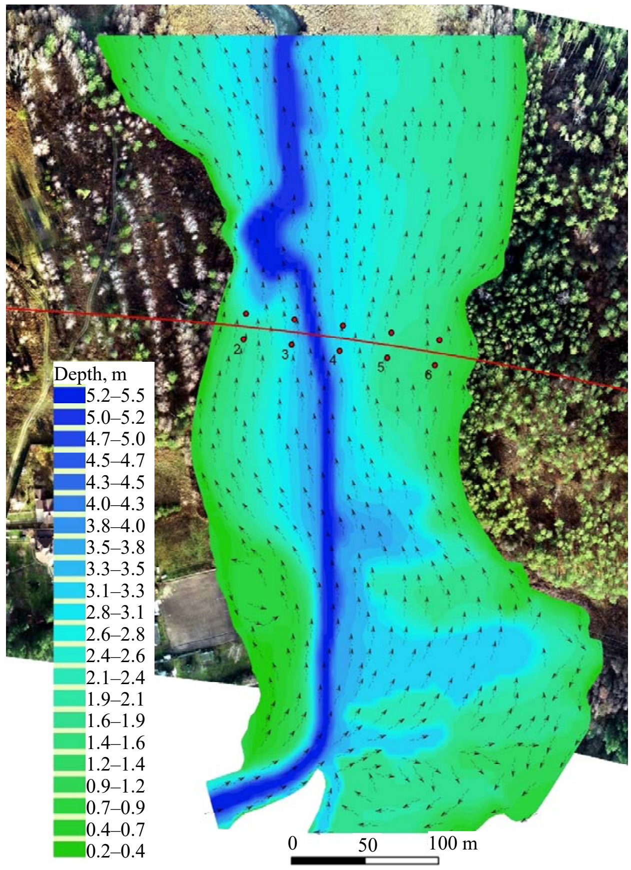

Fig. 4. Distribution of depths and velocities of flood flow within the estimated area

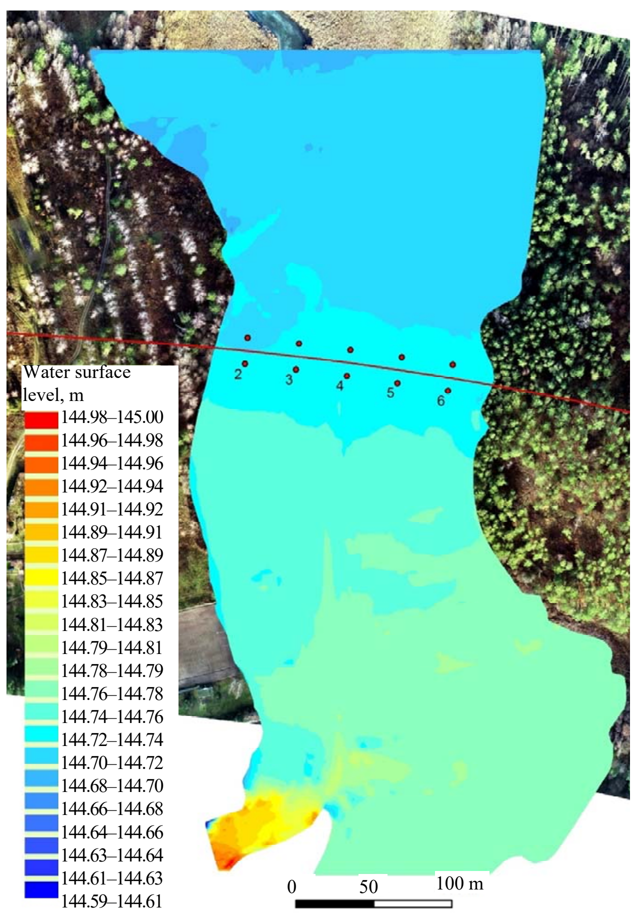

Fig. 5. Water surface level according to the results of modeling a breakthrough wave

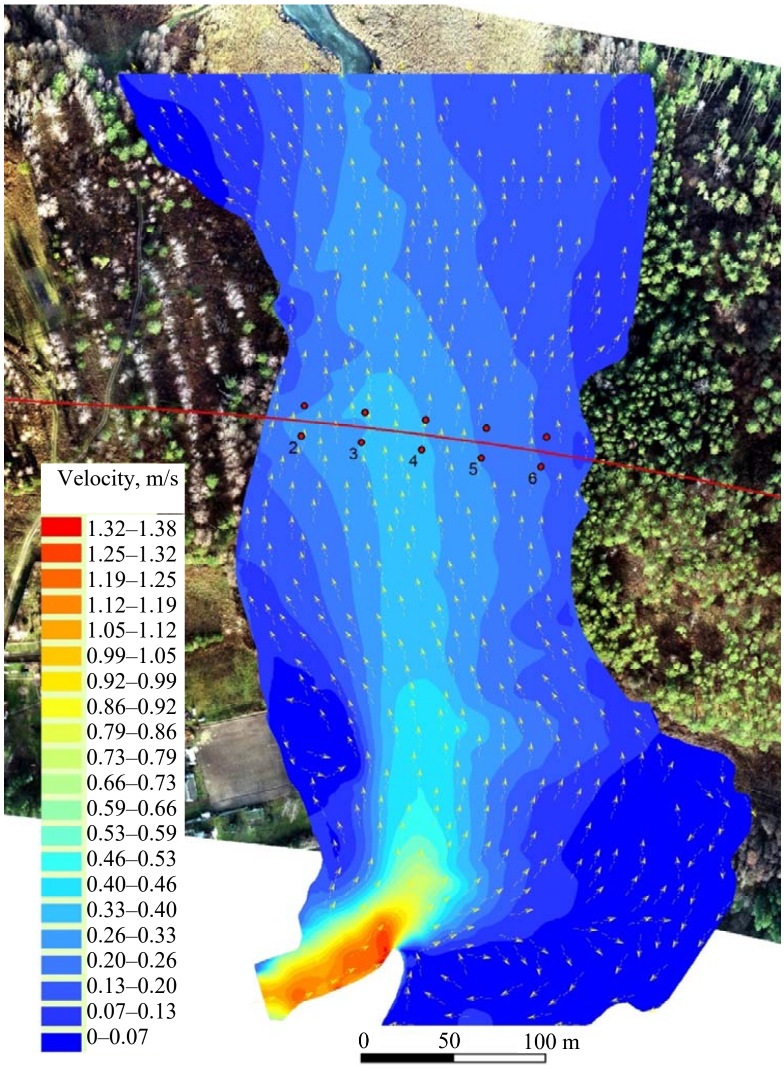

Fig. 6. Flow velocity field after exiting the near-dam water discharge facility

5. 3. Results of modeling the hydrodynamics of a breakthrough wave movement, taking into consideration the topographic and hydraulic riverbed conditions

Hydraulic calculations were carried out taking into consideration the interrelated properties of the main layer of the floodplain, which consists of peat accumulations, and the heterogeneity of the depths and roughness of floodplain surfaces of soils. There is almost no erosion of supports in the floodplain zone.

In the lower reservoir gate, there is a bridge crossing the river (Fig. 4–6). The bridge crossing crosses the section of the previous cascade reservoir at a distance of L=350 m from the dam's water discharge facilities.

As a result of monitoring, it turned out that a mine-type water discharge facility was designed for flow rates that are more than half the estimated flow rate ($Q_{p1\%}$=117 m^3/s). Because of this, questions arose about the need to assess negative events in which there may be threats associated with overflow effects and the formation of breakthrough waves.

Because of the lack of reliable initial data and small volumes of accumulation and flow of floodwaters, we shall perform calculations by approximate methods using empirical formulas recommended in such cases by regulatory and reference manuals [5]. The design distance between the road track and the dam is set at 350 m, which should be justified in accordance with the current design standards.

In its physical essence, a breakthrough wave is the uncontrolled movement of the flow of a mixture of water and sludge, in which the depth, width, inclination of the surface, and the speed of flow change over time [3, 17], (Fig. 7).

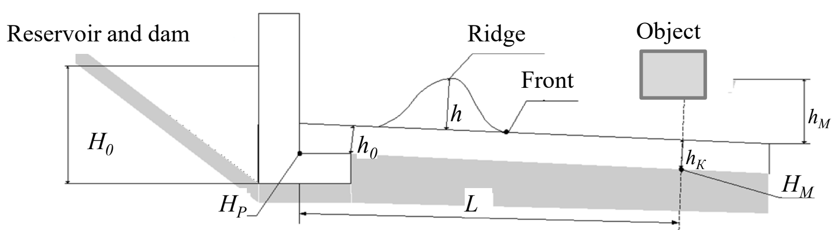

Fig. 7. Schematic showing the longitudinal section of a hydraulic unit with a section of the river, the shape of a breakthrough wave, and the object of hydrodynamic danger [12]

In this case, we determine the parameters of the transformation of a breakthrough wave (pass) to the specified distance L from the dam, depending on the topography of the terrain and other obstacles.

To calculate the transformation of a flood flow, we define the maximum discharge of a breakthrough wave using the following formula:

$$Q_{SB} = Q_P\left(1-\frac{W_{0\text{P}}}{W_\text{P}}\right)k_h, \tag{5}$$

where k_h=0.85 is the coefficient of smoothing the fractures of the triangular hydrograph, the duration of which, according to hydrological data, is T_{days}=4 days;

$W_{\text{P}1\%}$=86400$Q_p T_{\text{days}}$=20.2·10^6 m^3 is the volume of the estimated flood over the duration of the hydrograph;

$W_{0\text{P}}$=$S_0 H_0$ is the regulatory volume of the reservoir, which is equal to the product of the surface area of the pond S_0=0.35*10^6 m^2 (calculated on the map of the estimated area) and the average depth of operation H_0=2 m. According to these data, Q_{SB}=95 m^3/s, which is less than the estimated flood flow rate, $Q_{\text{p}1\%}$=117 m^3/s. This is evidenced by the presence of transit properties of the section of the breakthrough wave to the route of the bridge crossing without additional flow – a significant increase in the catchment according to the classification of breakthrough modes.

To build a spatial model of the breakthrough wave propagation, one should use the available drawings of the planned location of the dam and motor road with the involvement of high-altitude shooting, which is indicated by marks and isolines in Fig. 2, 3.

The method of step-by-step approximation was used to calculate the passage of flow rates by the mine-type water discharge facility and their redistribution with an overflow through the ridge with a length of B=220 m, which made it possible to determine the overflow head in the amount of H_P=0.72 m. This is much more than the normatively acceptable size of H_P=0.1 m in order to neglect the possibility of dam erosion and the formation of a breakthrough wave, which must be taken into consideration when calculating the structures of a bridge crossing. Our calculations based on these parameters make it possible to conclude that the distance of the route from the dam meets the following regulatory requirements: L=350 m>2Bk_h=2·220·0.34=149.6 m.

For comparison, we determine the height of a breakthrough wave near the dam:

$$H_{BW}=0.6h-h_0,\ \text{m}, \tag{6}$$

where h is the depth of the reservoir near the dam, m; h_0 – the depth of the river below the dam, m.

The time of passage of the breakthrough wave through the hole of the destroyed dam determines the time of emptying the reservoir in hours [18]:

$$T=\frac{W_0 A}{3{,}600\mu B_i h\sqrt{h}},\ \text{h}, \tag{7}$$

where W_0 is the volume of the reservoir; A is the coefficient of curvature of the reservoir, approximately, for the calculation, is taken equal to 2; μ – a parameter that characterizes the shape of a riverbed (μ=0.6 with a riverbed shape close to the parabolic form); B_i – the width of the breakthrough, m.

According to calculations, T=2.8 min., which should correspond to the maximum values of the height and speed of propagation of the breakthrough wave, which are determined from the formulas without taking into consideration the transformation in the plan

$$h_{\max} = \frac{2h^2}{Li+3.3h}, \tag{8}$$

$$v_{\max} = \frac{\beta_{\max}\sqrt{h^{1.33}i}}{n\left(\frac{Li}{h}\right)^{0.37}}, \tag{9}$$

where L is the distance from the dam, n and β_{max} are the coefficients of roughness and curvature of the riverbed (dimensional), usually, β_{max} is 0.6, the value n varies from 0.02 to 0.5; i=0.0015 – the average slope of the riverbed.

These parameters change very slowly as they move away from the dam (Fig. 8) but significantly depend on determining the parameters of the riverbed (roughness, curvature, etc.) and the viscosity of the water-soil mixture formed during the destruction of the dam.

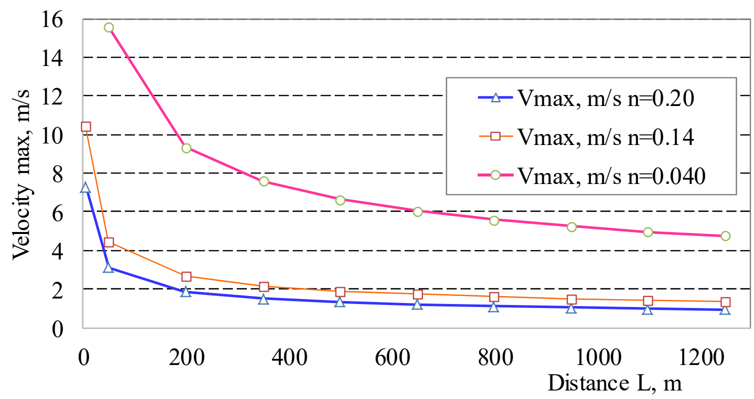

Fig. 8. Change in the maximum velocity values of wave propagation from the dam

We determined the time over which a breakthrough wave reaches the specified gate as follows:

$$t=L_i/V,\ \text{h}, \tag{10}$$

where L_i is the length of the river section with the velocity of breakthrough wave movement V. For example, for a river with a well-formed riverbed, with narrow floodplains without large heads, at a slope of the bottom i=0.0012, the average velocity of movement along the riverbed and floodplain sections, according to [7], is V_1=10 km/h. In this case, according to Table 1, this is a riverbed with a characteristic of 2 at a slope i=0.0015 and an average roughness n=0.140.

In another way, the time of emptying the reservoir in seconds (with the instantaneous destruction of the dam to the limit level) [6, 19] is determined:

$$T_0 = \frac{4.5\Omega}{B_i\sqrt{2gh}}, \tag{11}$$

where Ω is equal to the area of the reservoir, taking into consideration the proportional reduction of the mirror (for a particular reservoir in question, the area is S_R=3.5·10^5 m^2 (Fig. 9) and the emptying time of the pond will be T_0=8.7 min).

Table 1

The movement velocity of a breakthrough wave along the riverbeds of natural watercourses

Riverbed and floodplains characteristics	i=0.01	i=0.001	i=0.0001
1. On rivers with wide floodplains	4–8	1–3	0.5–1
2. On winding rivers with thickets or uneven stony floodplains, with widening and narrowing of the floodplain	8–14	3–8	1–2
3. On rivers with a well-formed channel, with narrow floodplains without large heads	14–20	8–12	2–5
4. On gently winding rivers with steep banks and narrow floodplains	24–18	12–16	5–10

Fig. 10 shows the results of the formation of a breakthrough wave along the floodplain. In Fig. 4, 10, it is noticeable that when the breakout flow spreads, it returns in accordance with the reversal of the river, a stagnant zone is formed on part of the riverbed.

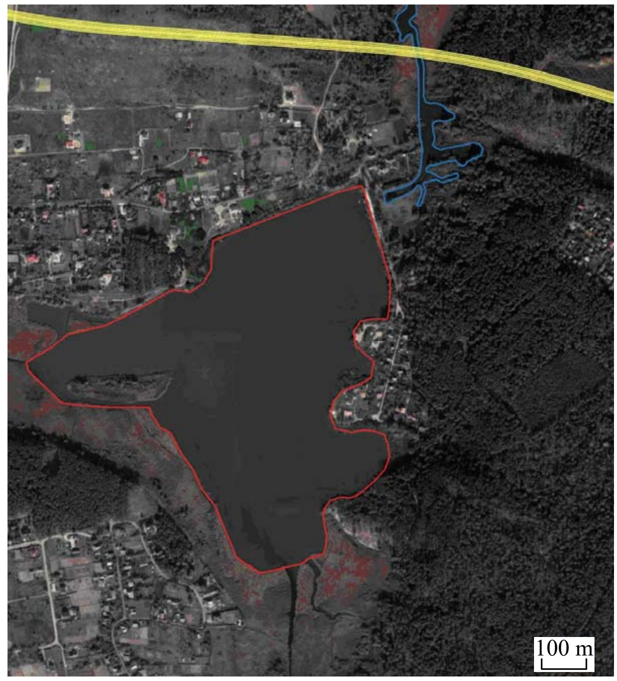

Fig. 9. Location of the reservoir near a bridge crossing on the highway, with an area of S_R=350,000 m^2

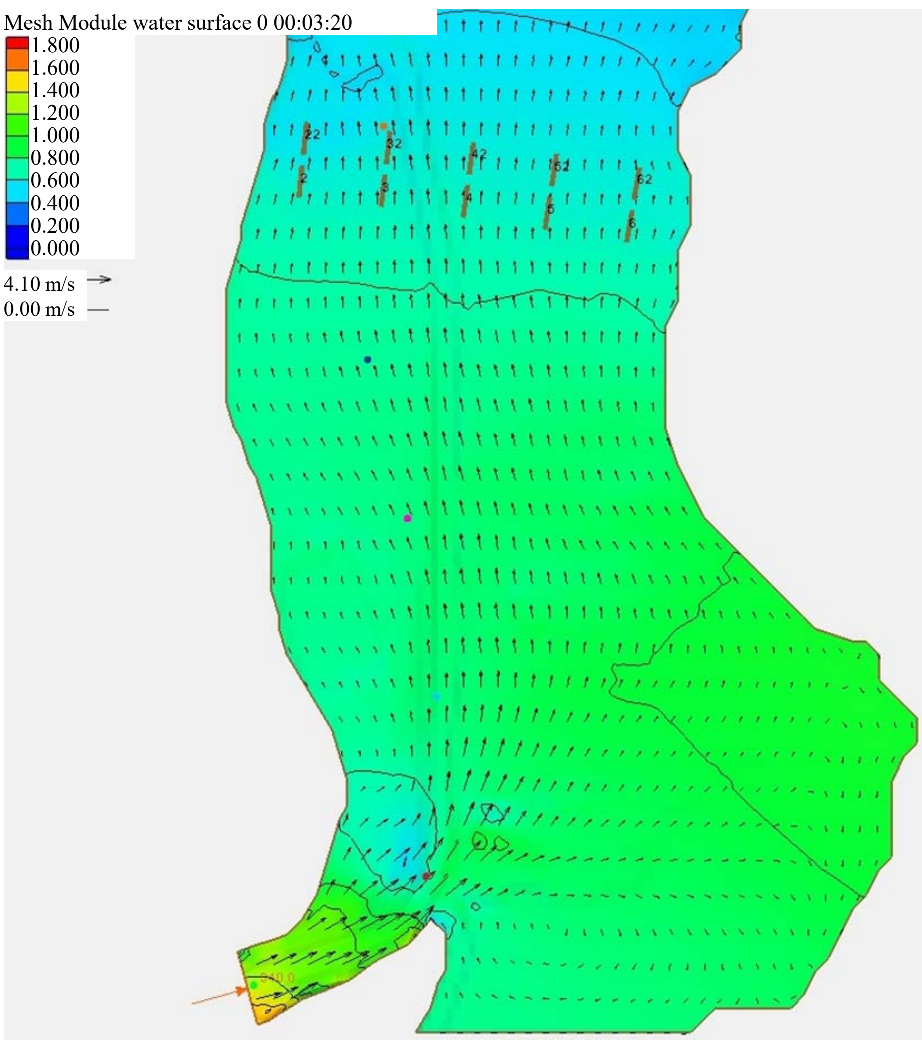

Fig. 10. Formation of a breakthrough wave across the floodplain

According to the calculations based on a one-dimensional model of the speed of propagation of the breakthrough wave, the results of the planned model are much higher (Fig. 10).

Our calculations according to the plan model (6) to (9) (preliminary calculations were performed on the basis of a one-dimensional model) showed that during the propagation of a breakthrough wave from the dam, the front of the wave spreads along the riverbed. This leads to a much larger, compared to one-dimensional calculations, slowing down its movement, reducing the height (Fig. 11), reducing the concentration of the water-soil mixture. Thus, the use of the proposed technique makes it possible to more accurately determine the load from the breakthrough wave when calculating the stability of bridge structures.

Fig. 11 shows the dimensionless values on the Arc 1 profile (along the riverbed depicted in Fig. 4) of the estimated heights of the breakthrough wave h_w (relative to the initial head h_0=5.8 m) and velocities (relative to the free-fall rate from the height of the initial head).

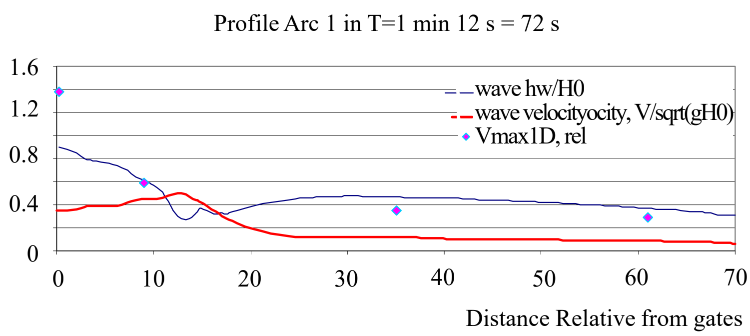

Fig. 11. Relative parameters of the propagation of a breakthrough wave along the profile in the direction of the riverbed

The height of a breakthrough wave and the change in the propagation velocity of the breakthrough wave in the mathematical model is calculated in all nodes of the estimation grid. In the process of post-analysis, the results are represented in the identified seven Pt1–Pt7 sensors evenly located along the Arc 1 profile (Fig. 12).

We take into consideration not only the reduction of flow rate when emptying the reservoir by 95 m^3/s but also the decrease in the head height from 2.1 m to 1.25 m, which is why there is a decrease in velocity, which is seen when comparing the values of the velocity of wave propagation along the profile (Fig. 13).

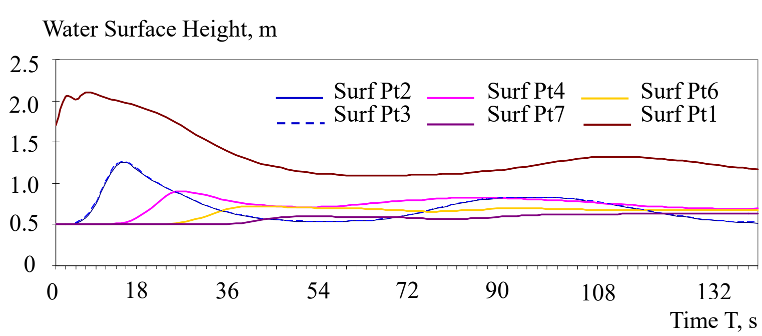

Fig. 12. Breakthrough wave velocities at different distances from a head front

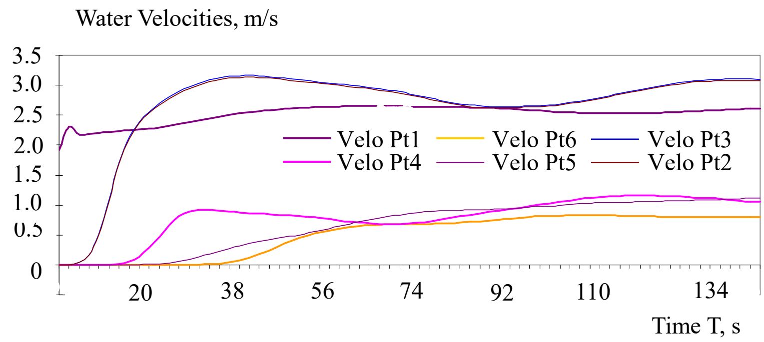

Fig. 13. Change in the height of a breakthrough wave during its propagation along the riverbed

The value of a breakthrough wave height h_w is lowered by 0.85 m as it propagates along the riverbed and the front expands along the floodplain. The comparison with the calculation results based on a one-dimensional model shows that the velocity of a breakthrough wave movement decreases much faster as a result of accounting the spread across the floodplain.

Changes in the velocity and height of a breakthrough wave over time at points at different distances from the head front on the estimation plots (Fig. 12, 13) indicate a significant decrease in these parameters when the front of the breakthrough wave moves away. This may help determine the danger to objects at the lower gate.

6. Discussing the construction of a numerical model and the results of studying the parameters of a breakthrough wave

This paper reports the devised methodology for calculating the parameters of the flood pass and the movement of a breakthrough wave. The main advantage of our procedure is the consideration of factors of the topographic and hydraulic conditions in the riverbed, which significantly affect the breakthrough wave propagation. To this end, we use a model of the kinematics of riverbed and breakout flows, which is built on the basis of equations of flow velocity, the erosion, and transport of sediments that are averaged for the depths of the flow.

The differential equations describing the nonstationary current averaged for depth are solved using the numerical system FST2DH (2D Depth-averaged Flow and Sediment Transport Model) [12], which implements a finite-element method in the plan of the topographic region of the riverbed.

The model built has made it possible to obtain results for a specific case of an actual riverbed and a water-head facility, to assess the impact of irregularities and roughness of the bottom. The impact of the flow on the floodplain is also taken into consideration.

The results of our studies of the hydrodynamic parameters of a breakthrough wave movement showed that there is a decrease in the velocity along the profile, from 3 m/s to 1 m/s. At the same time, when the flow moves away from the intersection of the breakthrough, there is a significant decrease in head height, from 2.1 m to 1.25 m.

Fig. 4, 9 demonstrate that when the breakout flow spreads, it returns in accordance with the reversal of the river, a stagnant zone is formed on part of the riverbed.

Calculations according to the plan model (6) to (9) showed that during the propagation of a breakthrough wave from the dam, the front of the wave spreads across the riverbed. This leads to a much greater slowdown in its movement compared to the one-dimensional calculations proposed by regulatory documents, and a decrease in height (Fig. 11). There is also a decrease of 0.65 m in the height of the concentration of the water-soil mixture, which allows for more accurate calculations of the load from the effect of the breakthrough wave in the calculations of the stability of transport and other structures.

The results of our calculation showed that Q_{SB}=95 m^3/s, which is less than the estimated flood flow rate of $Q_{p1\%}$=117 m^3/s. This indicates in the presence of transit properties of the section of the coming breakthrough wave to the bridge crossing route without additional flows – a significant increase in the catchment according to the classification of breakout modes.

The method of step-by-step approximation was used to calculate the passage of flows through a mine-type water discharge facility and their redistribution with an overflow through a ridge with a length of B=220 m. That has made it possible to determine the overflow head, H_P=0.72 m, which is much larger than the normative value, H_P=0.1 m, in order to neglect the possibility of dam erosion and the formation of a breakthrough wave. This should be taken into consideration when calculating the structures for a bridge crossing.

One of the disadvantages of the current study is a two-dimensional method of calculating the pass parameters for the flood and a breakthrough wave. The development of methods for applying a spatial mathematical model is a promising area of further research.

7. Conclusions

1. We have built a model of the kinematics of riverbed and flood flows, which makes it possible to calculate the pass parameters for the flood and a breakthrough wave and determine their impact on the formation of the erosion of sub-bridge structures taking into consideration the factors of topographic and hydraulic conditions in the riverbed.

Unlike known models, the proposed model makes it possible to calculate in the plan approximation the erosion of the bottom, taking into consideration the interaction with coastal and riverbed hydraulic engineering structures, including local erosion.

2. A study into the kinematics of the current during the passage of flood flows was carried out. It has been established that the modules and directions of the vectors of flow velocities significantly depend on the surface plant species and depths of the floodplains. Minor erosion depths during the estimated flood period (less than 4 days) are forecast due to the slowness of the erosion. The calculation results showed the flow rate Q_{SB}=95 m^3/s, which is less than the estimated flood flow rate, $Q_{p1\%}$=117 m^3/s.

It is established that the depth of the flood flow across the riverbed varies from 5.5 m to 0.2 m.

3. The results of studying the hydrodynamic parameters of a breakthrough wave movement, taking into consideration the topographic and hydraulic conditions in the riverbed, showed that there is a decrease in velocity along the profile, from 3 m/s to 1 m/s. When the flow moves away from the intersection of the breakthrough, there is a significant decrease in the height of the head, from 2.1 m to 1.25 m.

Acknowledgments

We are sincerely grateful for data on the bathymetric and hydrological measurements, acquired during the field research by the expedition “Geodetic Agency” headed by O. Primak, organized by the “Megaproekt” consortia (Chief Engineer V. Paliy).

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