Journal of Applied Mechanics and Technical Physics, Vol. 49, No. 5, pp. 754–761, 2008

EXPERIMENTAL VERIFICATION OF METHODS FOR CALCULATING PARTIAL DAM-BREAK WAVES

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UDC 532.532+532.59

This paper presents results of an experimental study of waves generated by partial break of two model dams. The previously proposed calculation methods are extended and compared with the experimental data obtained. It is shown that the wave propagation speed in the tailwater is significantly influenced by the energy losses due to flow through the breach.

Key words: *partial dam break, dam-break wave, experiment, calculation methods.*

Introduction. The propagation of dam-break waves in a real river channel with lateral inflows, floodplains, and various hydraulic resistances was calculated numerically using Saint-Venant equations. Information on the corresponding computer programs can be found in the Internet (http://www.niies.ru/chisl_metod.htm; http://www.wmo.ch/web/homs/projects/Components/Russian/k15302.html). A review of the research preceding the development of these programs can be found in [1–3]. As initial information in the calculations, one needs to specify full-scale data on the geometry and hydraulic resistances of the river system and initial data at the dam site. It is reasonable to obtain initial data by solving the dam break problem based on a simpler mathematical model — the first shallow-water approximation. The corresponding theory for the case of instantaneous full break of a dam above an even horizontal bottom is presented in [4, 5], and results of its experimental verification are given in [6, 7].

In the present paper, we consider a more complex partial-break problem with three-dimensional vortex flow in the vicinity of the undamaged part of the dam. In this case, the system of equations of the first shallow-water approximation is not closed. Khristianovich was the first to propose a solution of the closing problem [4]. The novelty of his approach is that empirical information is invoked. In fact, this approach has subsequently been used to calculate waves in a real river system based on Saint-Venant equations. In [4], it is also assumed that, in the vicinity of a partially broken dam, the flow rapidly becomes stationary, which is confirmed in experiments [8, 9].

The next important step in the solution of the closing problem was made independently in three papers [10–12] for the particular case of an uneven bottom at the dam site — a sharp bottom lowering downstream over the entire width of the channel (drop). In comparison with classical studies, in which the mass and momentum conservation laws were used for theoretical analysis, the fundamental novelty of papers [10–12] is that they also use the energy conservation law. An experimental verification of solutions [11, 12] for the case of dam break above a drop is performed in [13, 14].

The present paper presents experimental data on downstream waves resulting from a partial break of two dam models, gives an extension of theory [11, 12], and compares the calculation methods with the experimental data obtained.

1. Formulation of the Problem. A diagram of the model problem of partial dam break and the notation used below are given in Fig. 1. A dam with a rectangular breach is located in an infinitely long rectangular channel with a horizontal bottom. In the initial state, the breach is covered with a shield, which produces a difference between the headwater and tailwater levels $h_- - h_+ > 0$. At the time $t = 0$, the shield is removed instantaneously (in the present experiments, within 0.04 sec). As a result, a level depression wave propagates upstream and a

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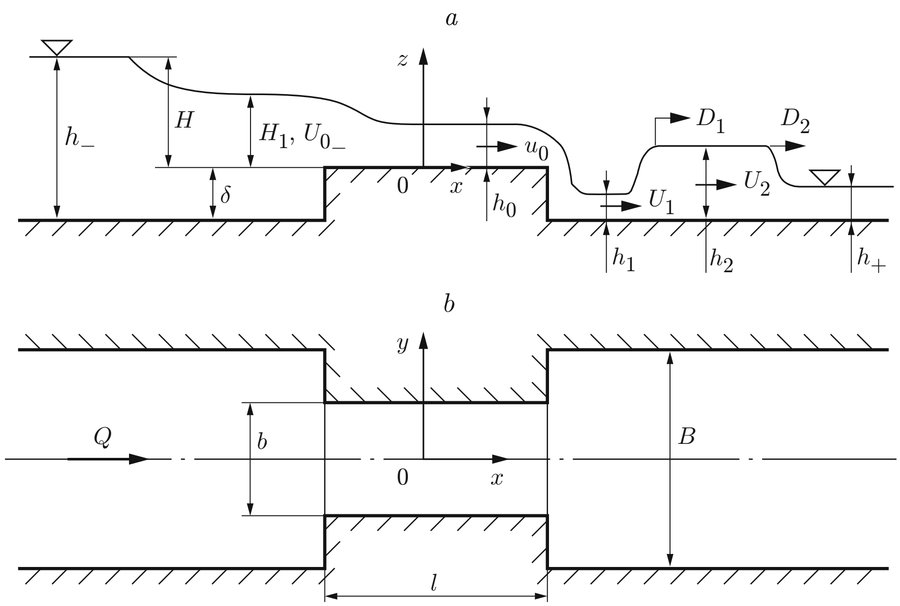

Fig. 1. Flow diagram: (a) side view; (b) top view.

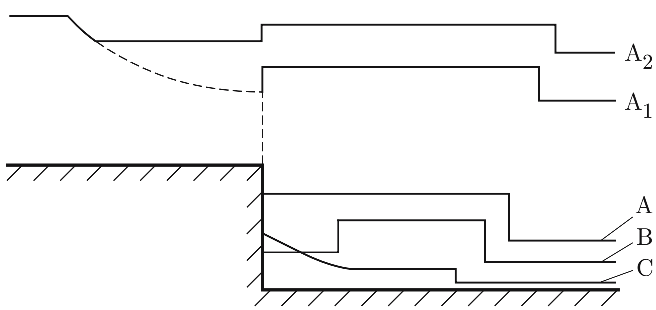

Fig. 2. Theoretically possible wave profiles in the tailwater in the case of a dam break above a drop [14].

dam-break wave propagates downstream. Below, the dam-break wave is primarily considered. The upstream wave needs to be studied in greater detail because, according to experimental data [8, 9], the shape of this wave is more complex than that in the first shallow-water approximation.

In theoretical analysis using the first shallow-water approximation, the liquid viscosity is ignored. In this case, the flow is determined only by six geometrical parameters: h_-, h_+, B, b, δ, and l (see Fig. 1) and the acceleration due to gravity g. After conversion to dimensionless quantities, the number of geometrical parameters decreases to five, and the parameter g in the construction models remains dimensional. In the problem of a full dam break, the number of dimensionless geometrical parameters decreases to one [7]. In experiments, the wave propagation is influenced by the liquid viscosity, the hydraulic unevenness of the solid boundaries, the shape of the edges at the entrance to the breach, and the characteristics of movement of the shield. This limits, in particular, the range of distances from the dam in which comparison with the theory will be correct. In experiments, the channel length is limited, and, therefore, a comparison with the theories in question becomes incorrect after the first reflected wave reaches the channel cross section with a given coordinate x or the upstream wave reaches the channel wall, so that the boundary conditions here become different from those in theory.

The experiments performed in [8, 9] and in the present study show that partial dam-break waves can be of the same types (A, B, and C) as those resulting from a dam break above a drop. Below, this is used in extension of theory [11, 12]. The wave shapes obtained theoretically in [12, 14] are given in Fig. 2. Estimates show that, for large values of the parameter δ/h_-, waves of type B (see Fig. 2) exist for small values of h_+ (about fractions of a millimeter under laboratory conditions) that cannot be obtained experimentally. The waves of type A_1 and A_2 shown in Fig. 2 exist for the submerged state of head and tail conjugation. (The state of conjugation is called free if the tailwater flow does not influence the headwater flow; otherwise, it is submerged.)

In the case of a wave of type B with two hydraulic jumps moving one after the other, it is of interest to determine the depths h_1 (directly behind the dam) and h_2 (behind the leading jump), the velocities of motion of the liquid U_1 and U_2 in the corresponding cross-sections, and the characteristic wave propagation speeds D_1 and D_2 (see Fig. 1). According to theory, these six quantities do not vary in time. Four rigorous relations for them can be obtained from the mass and momentum conservation conditions in coordinate systems moving at constant speeds D_1 and D_2. Two more relations can be obtained in a motionless coordinate system, but only after making some assumptions on the relationship between the headwater and tailwater flows. This is due to the fact that the condition of momentum conservation between the cross sections directly ahead of and behind the dam should take into account the unknown pressure forces on the top and bottom of the unbroken part of the dam.

In a theoretical study [11], the headwater and tailwater flows were related using the Bernoulli equation along the streamline in a motionless coordinate system with the possibility of energy losses taken into account. In [11], only the free state of head and tail conjugation is considered and it is assumed that the critical depths and speed are established immediately at the drop and that the wave shape in the headwater is the same as in the case of a full dam break above an even bottom. This does not cause objections in the case of an uneven bottom in the form of a drop over the entire channel width ($b/B = 1$) and the free state of head and tail conjugation. In the case of a partial dam break, the assumption that the critical depth is established in the breach can be used only under some restrictions imposed on the breach shape and size. In particular, this assumption is unsuitable if $l/h_- < 2$ [15]. The submerged state of conjugation is considered in a theoretical study [12], where a motionless jump of the free-surface level directly above the drop is assumed. In experiments, this jump is absent. For the submerged states of conjugation, theories [11, 12] yield identical results, except for some details insignificant for the further investigation [12, 14].

In the case of waves of type A, the relations $h_1 = h_2$ and $D_1 = 0$ hold. The quantities h_2, U_2, and D_2 remain unknown. From the conditions of conservation of the discharge and momentum in a coordinate system moving at the speed D_2, one can obtain two rigorous relations for these quantities. The third relation in [11] is obtained in the same way as was done for waves of type B, but the energy conservation law is not used. In this case, energy losses are ignored.

In the problem of a partial dam break, energy losses can be so significant that the tailwater flow is subcritical and hydraulic jumps are therefore not formed. This flow pattern is observed, for example, in the case of a narrow breach in a washed-out earth dam or in the case of water falling from a great height. Shallow-water theory includes the assumption of potential (irrotational) nature of motion. At the same time, before the breach, in the breach, behind the breach, and in the tailwater flow, powerful vortices are formed which absorb a significant portion of the translation energy. In [4], the discharge is additionally specified by empirical formulas and it is assumed that in the headwater free surface is the same as in the case of a full dam break above an even bottom in the submerged state of head and tail conjugation. Energy losses are ignored in this theory.

2. Calculation Methods Verified. The calculation method called below method 1 uses the following system of equations from [4]:

$$(U_2 - D_2)h_2 = -D_2 h_+; \tag{1}$$

$$gh_2^2/2 + (U_2 - D_2)^2 h_2 = gh_+^2/2 + D_2^2 h_+; \tag{2}$$

$$q = m(b/B)H_1\sqrt{2gH_1}\,; \tag{3}$$

$$q = 2(\sqrt{gh_-} - \sqrt{g(H_1 + \delta)}\,)(H_1 + \delta) = U_2 h_2 \tag{4}$$

(q is the specific discharge per unit of the channel width and m is an empirical discharge coefficient). This closed system does not contain the parameters h_1, U_1, and D_1; therefore, strictly speaking, it is suitable only for calculating the parameters of waves of type A.

The method called below method 2 is based on a combination of the basic ideas from [4, 11, 12]. This method uses relations (1)–(4) and the relations

$$q_0 = qB/b, \qquad h_0 = kH_1, \qquad u_0 = q_0/h_0; \tag{5}$$

$$e_0 = \delta + h_0 + u_0^2/(2g); \tag{6}$$

$$U_1 = \sqrt{2g(e_0/(1+\zeta) - h_1)}\,; \tag{7}$$

$$(U_1 - D_1)h_1 = (U_2 - D_1)h_2; \tag{8}$$

$$gh_1^2/2 + (U_1 - D_1)^2 h_1 = gh_2^2/2 + (U_2 - D_1)^2 h_2; \tag{9}$$

$$\zeta = 0, \tag{10}$$

where q_0, h_0, u_0, and e_0 are the specific discharge, depth, and velocity of the liquid and the specific energy (above the channel bottom) at the exit from the breach, respectively; k and ζ are empirical coefficients. The coefficient k takes into account that, generally, the depth $h_0 \neq H_1$ (see Fig. 1). The presence of the free parameter k in the model allows method 2 to be used in the case of the submerged state of conjugation and a breach of an arbitrary shape. The coefficient ζ is introduced to take into account energy losses. In this method, it is set equal to zero [Eq. (10)].

In the case of an uneven bottom having the shape of a drop and the free state of conjugation, the headwater flow is such that there is no need to introduce the empirical coefficients m and k [11]. This is due to the establishment of the critical depth upstream of the drop [13]. The postulate that the critical depth is established in the breach is also true for the particular case of a breach in the form of a broad-crested weir and only for the free state of conjugation [9, 15]. In this case, taking into account the standard definition of the critical depth h_* [15], the relation

$$h_0 = h_* = \sqrt[3]{q^2/g}$$

holds and one does not need to introduce the empirical coefficient k.

For waves of type A, the following relations hold:

$$h_1 = h_2, \qquad U_1 = U_2, \qquad D_1 = 0; \tag{11}$$

they are used in method 2 to eliminate relations (5)–(10) from the equations of the system. In the analysis of waves of type A, methods 1 and 2 coincide; therefore, in this case, both method 2 and method 1 does not allow one to take into account energy losses.

The method called further method 3 differs from method 2 in that condition (10) is replaced by the condition $\zeta > 0$ only in the analysis of waves of type B and C and in the determination of the boundary of the region of their existence in the space of the specified parameters. We note that this region is narrowed with increasing ζ, other conditions being equal. Waves of type A are also analyzed using only relations (1)–(4) and ignoring energy losses.

In the case of waves of both type B and type A, energy losses are taken into account in method 4. In this case, waves of type B and C and the regions of their existence are analyzed using method 3. However, in the analysis of waves of type A, equalities (11) are used differently. Equations (2), (8), and (9) are eliminated, the condition $\zeta > 0$ is retained, and Eqs. (6) and (7) are replaced by the following relations:

$$e_0 = \delta + H_1 + q^2/(2g(\delta + H_1)^2), \qquad U_2 = \sqrt{2g(e_0/(1+\zeta) - h_2)}\,.$$

3. Experimental Technique. The experiments were performed in a rectangular channel of length of 8.3 m, width $B = 0.2$ m, and height 0.25 cm with an even horizontal bottom. Two series of experiments were performed. In the first series, a dam with a breach was modeled by a thin-walled rectangular weir with lateral compression, and in the second series, it was modeled by a broad-crested weir. The parameters $b = 0.06$ m and $\delta = 0.072$ m in the two series were identical. The length of the broad crest was $l = 0.38$ m. The initial head above the weir crest H and the initial tailwater depth h_+ were varied.

Changes in the free-surface level in time were measured at several points in the x direction using wavemeters. Before each series of experiments, the wavemeters were calibrated directly on the experimental setup by submerging in water to a specified depth. Comparison with calculations was made for the parameters h_2 and D_2. The value of D_2 was calculated from the time of propagation Δt of the height-averaged point of the leading front of the wave between two wavemeters separated by a specified distance Δx. The experimental data used below were obtained for values of the longitudinal coordinate x for which the wave-generated flow was one-dimensional and in such time

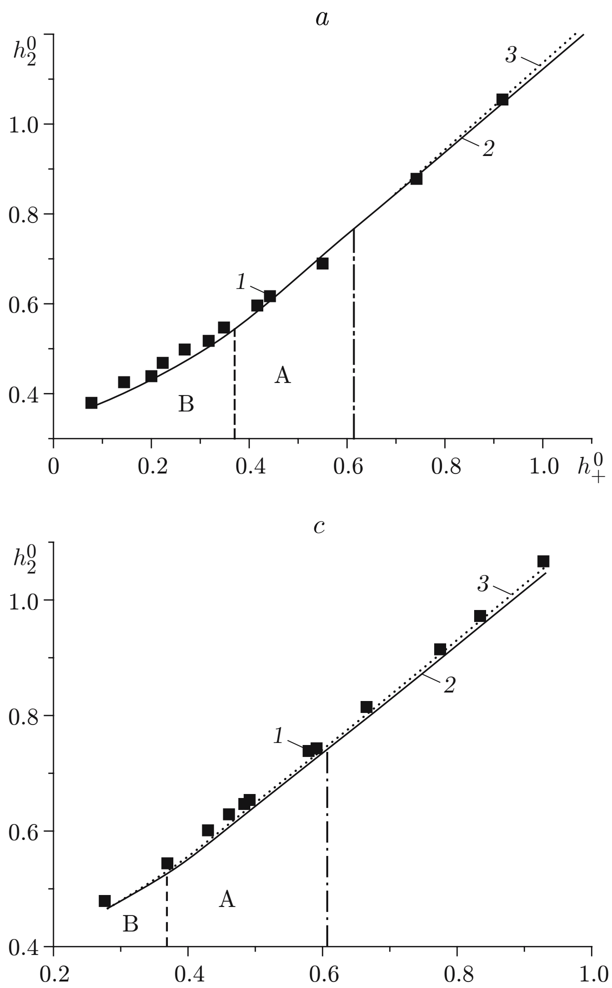

Fig. 3. Depth behind the leading front (a and c) and the propagation speed of the leading front (b and d) waves in the tailwater for a breach in the form of a thin-walled weir (a and b) and for a breach in the form of a broad-crested weir (c and d); points 1 refer to the experimental data and curves 2 and 3 refer to the data calculated using methods 3 and 4, respectively; the boundary of the regions of existence of waves of type B and A is shown by the dashed line and the experimental upper boundary of the free state of conjugation is shown by the dot-and-dashed line.

intervals in which the values of h_2 and D_2 did not change. This measurement technique has been used previously in studies of the waves resulting from a full dam break above an even bottom [7] and above a drop [13]. The reliability of the measurements is supported by the fact that, in these two particular cases, the experimental data are in good agreement with the corresponding data of theories [4] and [11, 12], which do not contain empirical coefficients.

The values of the coefficient m determined in preliminary experiments with specified stationary discharge are in good agreement with reference data. In particular, in the case of a dam in the form of a thin-walled weir, it is possible to use the Basen empirical formula [15], in which the initial head above the weir crest H must be replaced by H_1:

$$m = \Big[0.405 + \frac{0.0027}{H_1} - 0.03\Big(1 - \frac{b}{B}\Big)\Big]\Big[1 + 0.55\Big(\frac{b}{B}\Big)^2 \frac{H_1}{H_1 + \delta}\Big]\sigma. \qquad (12)$$

Here σ is the submergence coefficient, whose values are determined from table [15]. It should be noted that the author of formula (12) represented the expression $0.0027/H_1$ in dimensional form. This notation is kept in modern reference books on hydraulics [15], and formula (12) is therefore suitable only for water under normal conditions and the linear dimensions in it should be specified in meters.

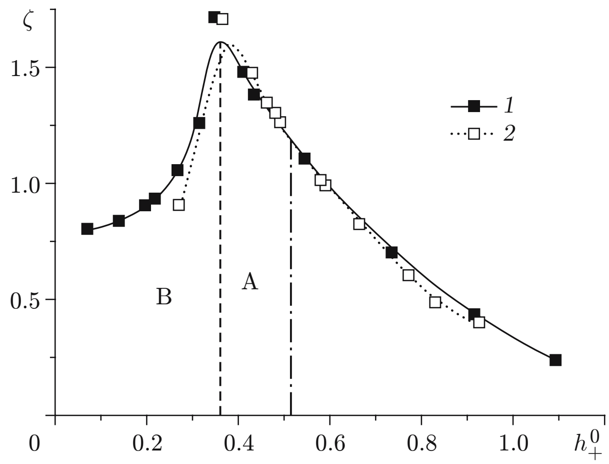

Fig. 4. Energy-loss coefficient ζ versus the parameter h_+^0 for thin-walled weir (1) and broad-crested weir (2); points correspond to experimental data; and curves to the results of approximation using B-splines; the boundary of the regions of existence of waves of type B and type A is shown by the dashed line; the theoretical upper boundary of the free state of conjugation is shown by the dot-and-dashed line.

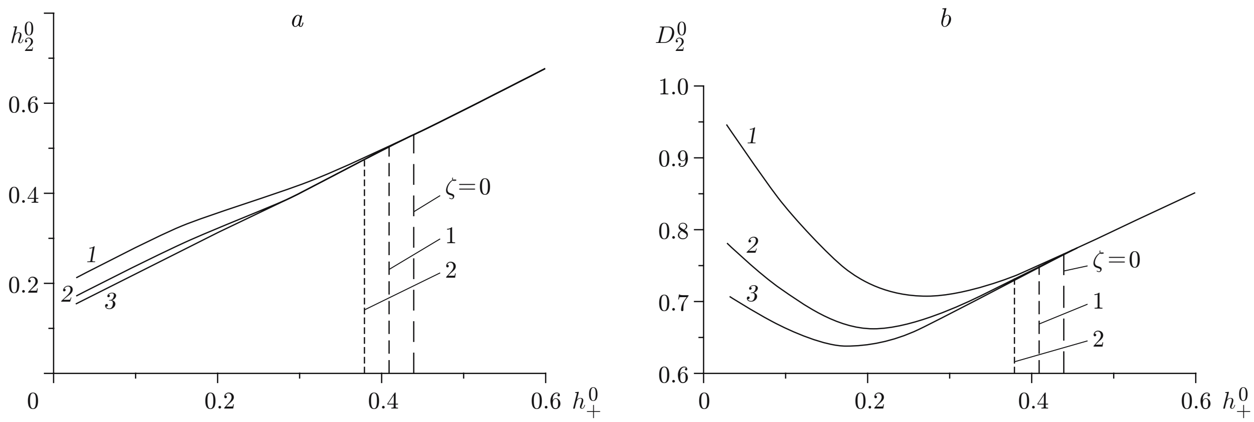

Fig. 5. Calculation of the parameters h_2^0, and D_2^0 using method 3 for various values of ζ: 1) $\zeta = 0$, 2) $\zeta = 1$, 3) $\zeta = 2$; dashed curves show the upper boundaries of the regions of existence of waves of type B.

In the case of a broad-crested weir, the values of m are also contained in reference books [15]. Under the conditions of the experiments performed and for the free state of conjugation, the coefficient m was constant: in the case of a thin-walled weir, $m = 0.423$, and in the case of a broad-crested weir, $m = 0.314$.

The measured values of the coefficient k for the free state of conjugation were also constant: for a thin-walled weir, $k = 0.85$, and for a broad-crested weir, $k = 0.67$. As the degree of submergence increases, $k \to 1$.

The coefficient ζ was chosen from experimental data on the wave parameters h_2^0 and U_2^0 so as to provide the best fit to the theory considered. This widely used method is called the identification of model parameters or model calibration. In particular, this method is employed to chose the empirical Chezy coefficient in the Saint-Venant equations from full-scale data for a particular channel.

4. Comparison of Calculated and Experimental Results. Comparison is made for calculated and experimental dependences of the depth $h_2^0 = h_2/H$ and propagation speed $D_2^0 = D_2/(gH)^{1/2}$ on the initial tailwater depth $h_+^0 = h_+/H$. As noted above, in the case of waves of type A, the results calculated using method 1 coincide with the results obtained by method 2, and for waves of type B and C, method 1, strictly speaking, is unsuitable. In turn, if in method 3, we set $\zeta = 0$, the results obtained by method 2 will coincide with the results obtained by method 3. Therefore, it is sufficient to compare experimental data only with the results obtained by methods 3 and 4. The results of the comparison are given in Fig. 3 for the cases of a breach in the form of a thin-walled weir and a breach in the form of a broad-crested weir. It was noted above that both method 3 and method 4 are suitable for the submerged state of conjugation, hence, the comparison for the submerged state is correct.

The results obtained by methods 3 and 4 can differ only in the region of existence of waves of type of A. However, from Fig. 3a, it is evident that in this region, too methods 3 and 4 give almost identical values for the parameter h_2^0 for both the free and submerged states of conjugation. The difference in the value of D_2^0 is more considerable (see Fig. 3b). In the calculations using method 3, which ignores energy losses, the depth behind the leading front is slightly underestimated and the propagation speed for waves of type A is overestimated. Similar results are obtained for a breach in the form of a broad-crested weir (see Fig. 3c and d).

Figure 4 gives curves of energy loss coefficient ζ versus the parameter h_+^0. The effect of the weir shape on this curve is insignificant. The maximum value of ζ is reached in the vicinity of the boundary between the regions of existence of waves of type A and B. The data in Figs. 3 and 4 were obtained for $\delta/h_- = 0.395$, but even for this relatively small value of this parameter, the energy losses are significant. For $h_+^0 = 0.37$, the losses were 63% of the energy at the entrance to the breach.

The data presented in Fig. 4 show that, in experiments with a breach of different shapes, the values of ζ differ only slightly. This is also true for the parameters h_2^0 and D_2^0. In particular, in two series of experiments for the same initial depths h_+^0, the difference between the values of ζ does not exceed 10%.

In the calculations using methods 3 and 4, the coefficient ζ was determined separately for each value of h_+^0. Figure 5 gives the results of calculations using method 3 for various values of ζ in the case of a breach in the form of a thin-walled weir. The case $\zeta = 0$ corresponds to method 2. Dashed curves show the boundaries of the regions of existence of waves of type A and B. The position of the boundary depends on the energy-loss coefficient ζ. As ζ increases, the boundary is shifted to the coordinate origin.

Conclusions. The results of the experimental verification of four calculation methods show that method 4, which takes into account energy losses the most fully, gives the most accurate values of the wave parameters in the tailwater. In method 3, energy losses are taken into account only in the region of existence of waves of type B, and in methods 1 and 2, they are not taken into account at all. Methods 3 and 4 yield identical results in the region of existence of waves of type B. In the region of existence of waves of type A, the results obtained by methods 1–3 coincide and differ from experimental data by not more than 5%. The results of the experiments and calculations using methods 3 and 4 show that the greater the energy losses, the wider the region of existence of waves of type A. Accordingly, the region of practical applicability of the simplest method 1 also broadens. Methods 1–3 give an overestimated wave propagation speed. For the conditions of the experiments performed, the propagation speed of the leading front of the dam-break wave calculated using the simplest method 1 exceeds its experimental value by not more than 10%.

This work was supported by the Russian Foundation for Basic Research (Grant No. 07-01-00015) and an Integration Project of the Division of the Russian Academy of Sciences No. 4.14.1.

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