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SUDDEN BLOCKING OF A SUPERCRITICAL OPEN-CHANNEL FLOW

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This paper reports results of experiments in which a steady-state nonuniform supercritical open-channel flow was suddenly blocked by a rapidly falling gate at a downstream distance of about one hundred critical depths. This results in a hydraulic jump propagating upstream. Experimental data on the shape, height, and propagation speed of its leading front are given. It is shown that the parameters of the jump differ significantly from the values found using a quasi-stationary approach.

Key words: *experiment, nonuniform supercritical flow, unsteady hydraulic jump, height and propagation speed of the leading front.*

Gravity waves of the bore type (a moving hydraulic jump) result from the propagation of a high tidal wave or a tsunami over a river or fjord, the fall of rock or meteorite fragments into a pool, dam or lock-gate breaking, reservoir's bank landslides, sudden stop of a tank partly filled with a liquid, etc. Mathematical models of various degrees of complexity have been developed to calculate such catastrophic waves. Computational methods based on the Saint Venant equations have the widest application [1–4]. Like the first shallow-water approximation [5], they use the assumption of a hydrostatic pressure distribution over the depth. The same assumption is also used in the control volume method [6].

In the first shallow-water approximation, cnoidal and solitary waves do not exist and all five types of hydraulic jump (see [7]) are simulated by a free-surface discontinuity. Cnoidal and solitary waves are described in the second shallow-water approximation [5], which takes into account deviations from the hydrostatic law. The most perfect mathematical models take into account not only deviations from the hydrostatic law but also flow vorticity and the turbulent mixing due to waves. Two such models are given in [8].

The results of an experimental verification of the first shallow-water approximation using as an example the problem of decay of an initial free-surface discontinuity above a bottom step in a rectangular channel are given in [9]. A mathematical model taking into account turbulent mixing was verified experimentally in [10]. The objective of the present study was to obtain experimental information to test different computational methods using as an example the problem of sudden blocking of a supercritical flow. Sudden blocking of a subcritical flow was studied experimentally in [11].

In practice, the hydrodynamic processes in question occur, for example, in reflection of nonlinear waves from vertical walls in shipping locks, ship elevators, tankers, ballast tanks, and submerged decks. The results of the studies performed are also useful for a number of gas-dynamics applications provided that the gas-hydraulic analogy is applicable.

A diagram of the experiment and the main notation are given in Fig. 1. In a rectangular channel of width $B = 6$ cm with an even horizontal bottom, a steady-state supercritical (rapid, according to the hydraulic nomenclature of [12]) flow was produced by flowage from an orifice under a sharp-edged shield. At a distance x_g downstream from the shield, a rapidly falling gate was placed, which could block the flow completely or partially in a time of about 0.01 sec. The moment of blocking (with the indicated uncertainty) was taken to be the reference time t. Before blocking, friction on the Plexiglas wall and bottom of the channel led to the formation of a so-called raised water curve [12], i.e., the free-surface level gradually increased downstream. In hydraulics, the pressure loss

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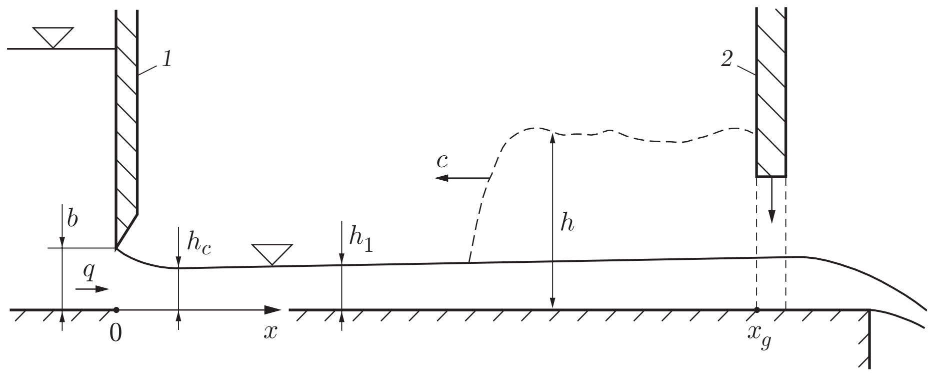

Fig. 1. Diagram of the experiment: 1) shield; 2) gate.

due to friction is taken into account by the Chesy coefficient C [12], which has the same dimension as $\sqrt{g}$, i.e., $\mathrm{m}^{0.5}$/sec (g is the acceleration of gravity).

The volume discharge Q was measured by a standard Venturi flowmeter, the raised water curve $h_1(x)$ before blocking by a measuring needle, and the fluid depth change after blocking $h(t)$ at a number of fixed points on the longitudinal x coordinate by wavemeters. The time Δt of longitudinal displacement of a chosen point on the wave profile by a distance Δx between two fixed wavemeters was used to calculate the quantity $c = \Delta x/\Delta t$, which depends on the choice of the point on the profile, on x and t, and on a set of parameters of the problem Π_i ($i = 1, \ldots, n$). The data given below pertain to the speed of motion of the point in the middle of the height of the leading wave front. It is called the speed of wave propagation. In comparing calculation and experimental data, one should bear in mind that in theory the speed of motion of a chosen point on the wave profile and the (phase) speed of propagation coincide only in the case of steady-stet solutions of the corresponding equations.

The obtained value of c referred to the coordinate of the middle of the interval Δx. The results of measurements with wavemeters were used to determine the height of the leading wave front $a(x, t, \Pi_i)$ and the time of propagation of the leading front from the rapidly falling gate to the wavemeter $t_p(x, \Pi_i)$. The total root-mean square error did not exceed 2% for c, 3% for a, and 1% for t_p.

The set of basic dimensional parameters of the problem Π_i includes the channel width B, the specific discharge $q = Q/B$, the Chesy coefficient C, the acceleration of gravity g, the opening of the shield b (a hydraulic term from [12]), and the distance from the shield to the rapidly falling gate x_g. For the time intervals studied, the parameters characterizing the real law of motion of the gate, the fluid viscosity, and the entrainment of air during wave breaking were of secondary importance. Next, the critical depth $h_* = (q^2/g)^{1/3}$ [12] is used as the characteristic linear scale and the quantities $V_* = (gh_*)^{1/2}$ and $T = (h_*/g)^{1/2}$ are the characteristic velocity and time scales. The corresponding dimensionless quantities are denoted by the superscript 0.

The effect of the channel width was manifested mainly in the fact that the depth in the compressed section h_c (a hydraulic term from [12]) and the Chesy coefficient differed from the corresponding values for an infinitely wide channel. If h_c is represented as $h_c = \varepsilon b$, then for an infinitely wide channel, $\varepsilon = 0.65 - 0.67$ [12]. In the experiments, $\varepsilon \approx 0.62$. Information on the Chesy coefficient is given below. In the experiments discussed, the parameters B, C, and x_g did not vary. Before flow blocking at the channel exit (at $x = x_e$), the water freely escaped into the atmosphere and the so-called second critical depth, $h_{**} \approx 0.77h_*$, was established [13]. The rapidly falling gate was placed at a value $x_g < x_e$ such that the effect of the conditions at the channel exit could be neglected.

Figure 2 shows examples of a raised water curve $h_1^0(x^0)$ and a conjugate depth curve $h_2^0(x^0)$. By the definition [12],

$$h_2^0 = h_1^0[\sqrt{1 + 8/(h_1^0)^3} - 1].$$

Figure 2 gives the values of the compressed depth h_c^0 and the coordinates of three characteristic cross sections of the channel: with the compressed depth x_c^0, at the location of the rapidly falling gate x_g^0, and at the channel exit x_e^0. On the boundary separating the subcritical and supercritical states of the flow, $h_1^0 = h_2^0 = 1$. The data of Fig. 2 demonstrate that before blocking, the flow was in the supercritical state ($h_1^0 < 1$) over the entire length of

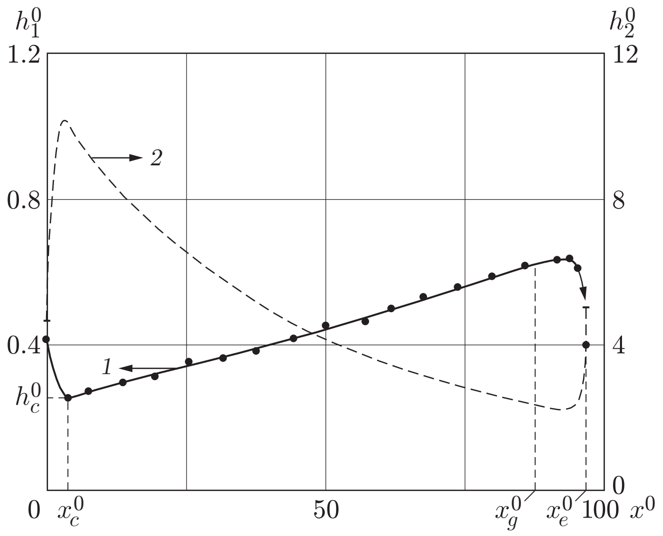

Fig. 2. Mutually conjugate depths at $h_* = 2.5$ cm and $b = 1.66$ cm (the values of C are given in Fig. 3).

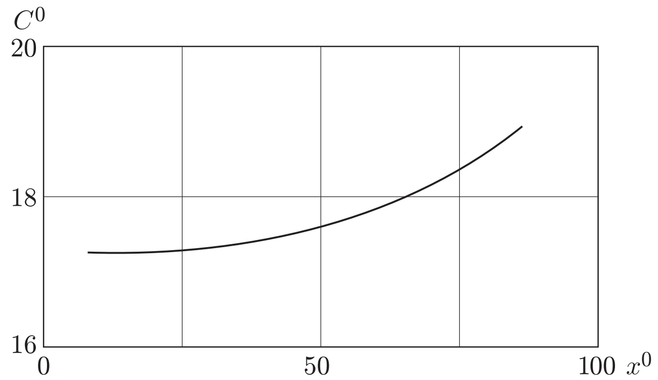

Fig. 3. Chesy coefficient $C^0 = C/\sqrt{g}$ for $h_* = 2.5$ cm and $b = 1.66$ cm.

the channel downstream of the shield. After blocking, transition to the subcritical state from the depth h_1^0 to the depth $h_{2e}^0 > 1$ occurred by a hydraulic jump moving upstream.

The Chesy coefficient was determined by solving the inverse problem based on the differential equation for the raised water curve, which is given, for example, in [12]. In the case of a channel with a zero bottom slope, this equation leads to the formula

$$C(x) = \sqrt{\frac{q^2 g(B + 2h_1)}{-(dh_1/dx)B(gh_1^3 - \alpha q^2)}},$$

where the coefficient α takes into account the nonuniformity of the velocity distribution over the depth. If, in this formula, one uses measured values of $h_1(x)$ and the derivative calculated from them dh_1/dx, the obtained values of C have a wider spread, which is typical of the solution of inverse problems. Therefore, the experimental data for $h_1(x)$ were fitted by the least-squares method. In this method, the choice of the smoothing function is of significance. For the experimental data in Fig. 2, the optimal approximation (at least in the interval $5 < x^0 < 85$) is given by

$$h_1^0(x^0) = 0.247 + 0.0037x^0 + 0.000007(x^0)^2.$$

The results of calculation of the dimensionless coefficient $C^0 = C/\sqrt{g}$ using this approximation function are given in Fig. 3. Since the channel walls and bottom were smooth, the values of C^0 were smaller than those for

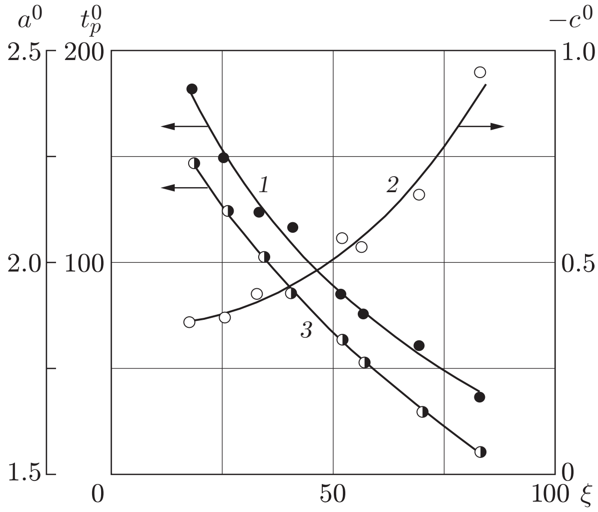

Fig. 4

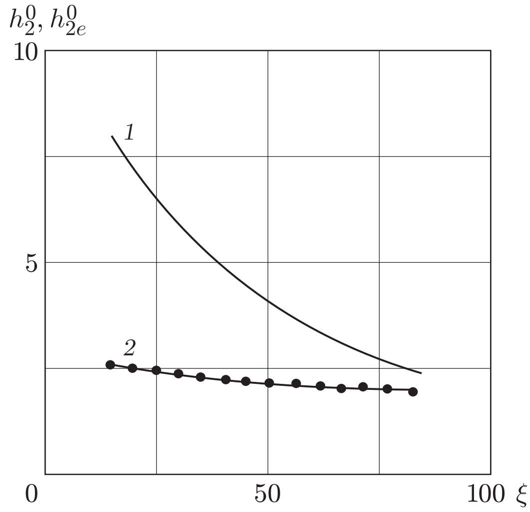

Fig. 5

Fig. 4. Height (1), travel time (2), and speed of propagation of the leading wave front (3) after flow blocking for $h_* = 2.5$ cm and $b = 1.66$ cm.

Fig. 5. Second conjugate depth under steady-state (h_2^0) and unsteady (h_{2e}^0) conditions: curves 1 and 2 refer to h_2^0 and h_{2e}^0, respectively; $h_* = 2.5$ cm and $b = 1.66$ cm.

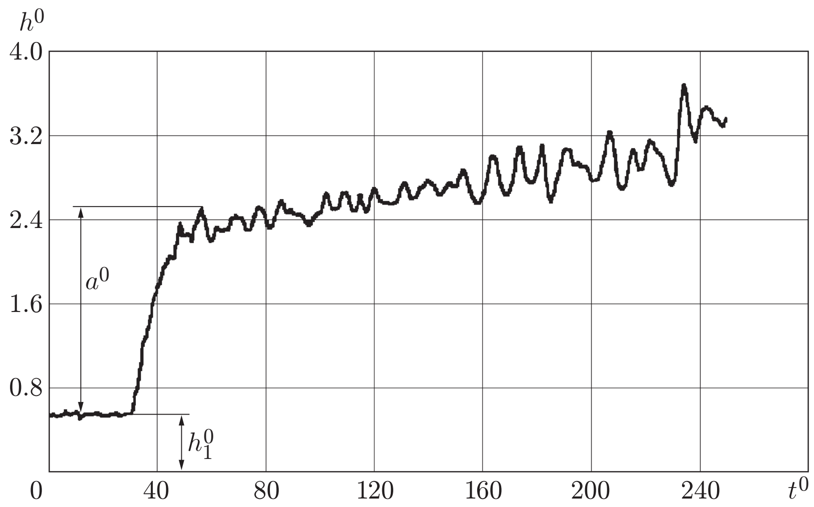

Fig. 6. Variation in the free-surface level $h^0(t^0)$ in the channel cross section $x^0 = 65.4$ for $h_* = 2.5$ cm and $b = 1.66$ cm (the time is reckoned from the moment of flow blocking).

natural channels [12]. In addition, in these experiments, the coefficient C^0 increased with increase in x^0. This was due to rather small values of x^0 and B.

Figure 4 gives plots of the functions $a^0(\xi)$, $t_p^0(\xi)$, and $c^0(\xi)$, where $\xi = x_g^0 - x^0$. The height of the jump and the travel time decrease continuously, and the propagation speed of the leading front increases continuously (in absolute value) with increase in ξ. This is due to the fact that after flow blocking the hydrodynamic processes are substantially unsteady.

The quasi-stationary approach is inappropriate for the analysis of the processes occurring after blocking of supercritical flows. Figure 5 shows the variation in flow depth behind the jump $h_2^0(\xi)$ for the quasi-stationary approach (see also curve 2 in Fig. 2) and the variation in the real depth behind the jump $h_{2e}^0(\xi) = h_1^0 + a^0$ after flow blocking. It is evident that the values of h_2^0 and h_{2e}^0 differ considerably, especially for small ξ.

To test the computational methods used, Fig. 6 shows as an example the depth variation with time at a fixed cross section of the channel. In this example, the wave head has the shape of a classical hydraulic jump. Unlike in the case of reflection of a dam-break wave from a vertical wall [1, 2], the depth behind this jump increases continuously due to supply of discharge water from the inward flow.

Experiments for various combinations of parameters showed that for the blocking of supercritical flows, the region of existence of undular hydraulic jumps in the space of problem's parameters is considerably narrower than that in the case of sudden blocking of a subcritical flow [11]. Therefore, one should expect that the computational method based on the Saint Venant equations, whose main drawback is an inadequate description of undular jumps, has a wider range of applicability in the problem in question than in problems of dam breaking or subcritical-flow blocking. An advantage of this method over the other methods is that it provides better accounting for the energy losses due to friction, which are of greater significance in supercritical flow calculations than in calculations of wave propagation over a quiescent fluid or a subcritical flow.

Recently, in tests of computational methods for nonlinear waves in finite-depth water, wide use has been made of dam-breaking processes in the case of an initially quiescent fluid [14]. The features of these processes for nonuniform flows are of interest for testing computational methods in which discharge appears in the boundary conditions. In applications, such boundary conditions must be specified, for example, in problems of a tsunami entering a river or the propagation of release waves from reservoirs.

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