Dam break study on potential destruction of a dam in urban area of the south west part of Bulgaria

MARIANA MARADJIEVA

University of Architecture, Civil Engineering and Geodesy, Hydraulics and Hydrology, Christo Smirnenski blvd. 1, Sofia, Bulgaria,

m_marajieva@abv.bg

Abstract The main objective of the paper is to relate a mathematical model for the formation, the moving and the further stratification of a catastrophic wave. Particularly the study evaluated the full or the partial potential destruction of the dam situated in an urban area. The considered dam is constructed by a rock fill type with clay core. The numerical algorithm describes two stages of the dam destruction: a full and a partial one. The first stage is applied in the occasion of a sudden and full destruction. For that purpose a non linear system of three equations is adjusted in a parabolic form of the stream cross sections. The main parameters of that system are the depth in the front, the velocity in the front and the spreading velocity. Finite difference method describes the further moving of the catastrophic wave. The second stage considers a partial destruction with an appropriate boundary condition. This condition depicts the change of the water discharge when the flow runs over the breach of the dam. Generally engineering methods are used to describe this mathematical model; probabilistic approaches are possible as well; here an engineering analysis is applied.

Keywords break; full destruction; partial destruction; transformation

INTRODUCTION

Such an extreme flood events as the dam break can pose a threat to human life and huge economic and environmental damage. To manage and minimize risk factors effectively it is necessary to identify hazard and vulnerability of hydro technical structures and especially dam engineering constructions. The main object of the study is the formation and the moving of a positive wave caused by a potential destruction of a dam situated in the Eastern part of Rila Mountain at about 25 km from the town of Belovo. The dam is located at a distance of 7 km from a constructed pumped storage power plant presented in the first figure.

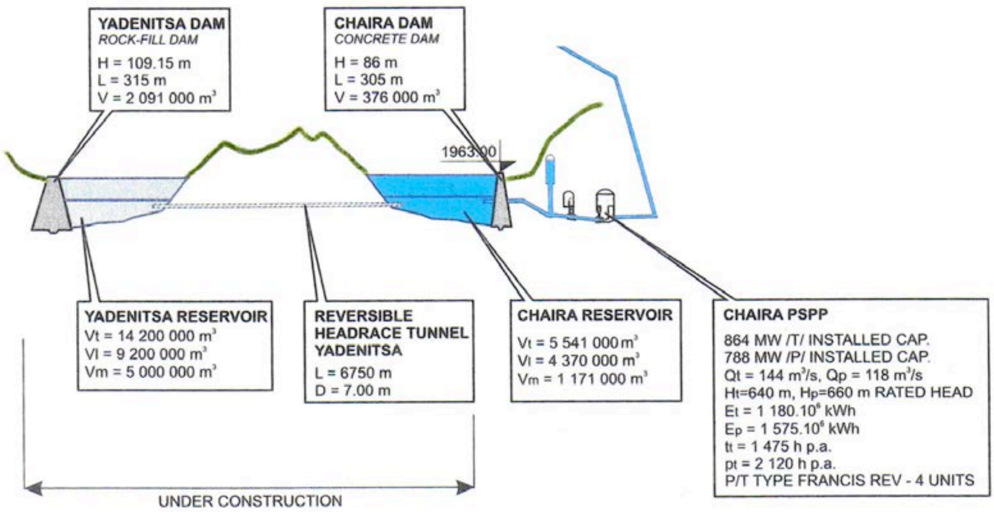

Fig. 1 Disposition of the dam with $Q_{0.01\%} = 190\, m^3 s^{-1}$

Numerical modeling tools consist of three key regions namely:

(1) Dependencies of the river Yadenitsa according to the characteristics features of the river cross sections. Topographic data are needed especially concerning urban areas. The use of GIS techniques assists to the rapid assessment of hydraulic data and calculations.

(2) The dam reservoir and the connected structures are very important in the next stage of the numerical computation; they are shown in Figure 2.

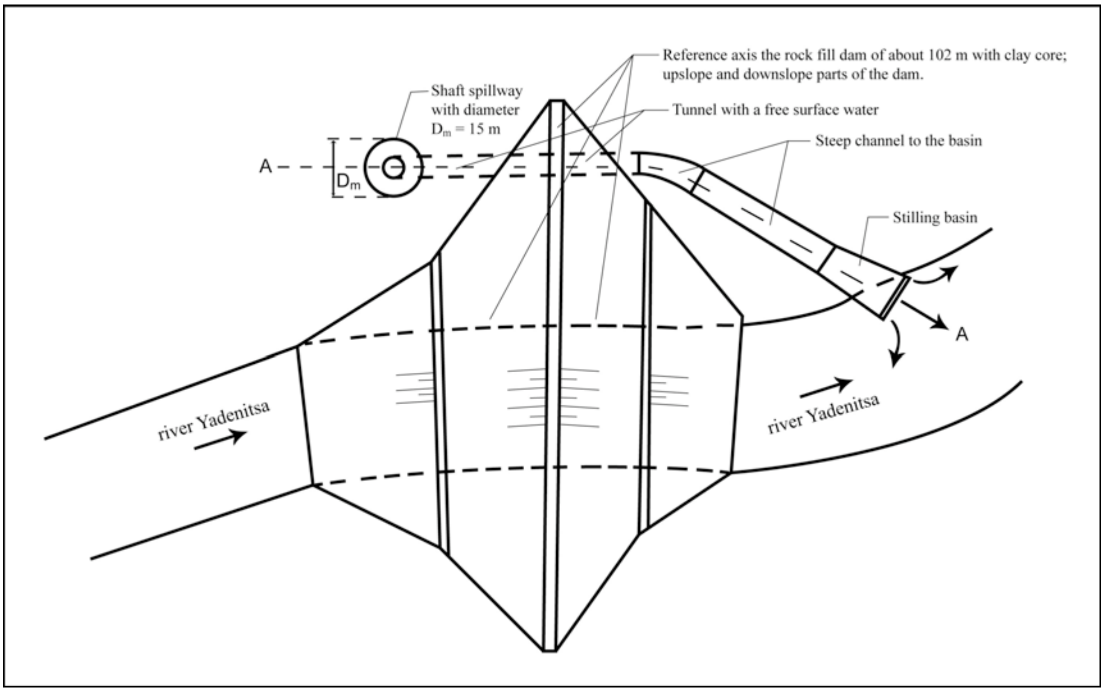

Fig. 2 Dam reservoir and the connected structures

The water discharge with two specific probabilities are $Q_{0.01\%} = 190\,m^3 s^{-1}$ and $Q_{0.1\%} = 130\,m^3 s^{-1}$.

(3) The formation of a positive wave from downstream is generated close to the mouth of Yadenitsa' river - Figure 3.

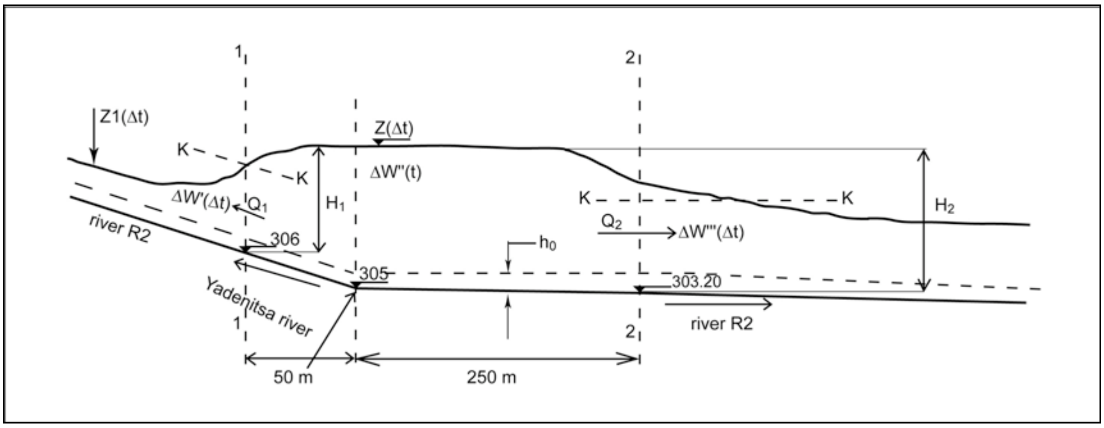

Fig. 3 Layout of the mouth of Yadenitsa' river at its influx with river R_2

Two balance equations manage the water volume when the catastrophic wave enters in the river $R2$.

$$
\begin{aligned}
&W' + W'' + W''' = W \\
&W + W' + W'' = W'''
\end{aligned}
\qquad (1)
$$

The first equation is valued for an accumulation of the volume $W(t)$ with the elevation $z(t)$ and the second one describes the perturbation of the initial water level and the positive wave coming from downstream of $R2$ river with a water discharge Q_1.

SIMULATION OF DAM BREAK FLOW IN THE RIVER

The consolidated close-to-the dam sections of the river are approximated by parabolic formula because this dependence describes well the river sections with their parameters

$$
\left|
\begin{aligned}
&B = aH^{1/k} \\
&\omega = \frac{ak}{k+1} H^{\frac{k+1}{k}}
\end{aligned}
\right.
\qquad (2)
$$

where B is the top width, ω area of the flow section, H is the flow depth, k, a are shapes parameters and the parameters are equal to - $k = 1.458$, $a = 11.62$.

The following three non linear equations are adjusted for a parabolic form of the stream cross section with prismatic or non prismatic beds

$$
\left|
\begin{aligned}
&(\xi - v_0) h_0^{\frac{k+1}{k}} = (\xi - v_f) h_f^{\frac{k+1}{k}} \\
&\frac{k}{2k+1} h_0 + \frac{(v_0 - \xi)^2}{g} = \left(\frac{h_f}{h_0}\right)^{\frac{k+1}{k}} \left[\frac{k}{2k+1} h_f + \frac{(\xi - v_f)^2}{g} \right] \\
&\frac{2(k+1)}{k} \sqrt{c_f} + v_f = \frac{2(k+1)}{k} \sqrt{\frac{gk}{k+1} h_1}
\end{aligned}
\right.
\qquad (3)
$$

where h_0 is initial depth after the dam, v_0 is initial velocity after the dam, h_f is the depth at the front, v_f is the velocity at the front, ξ is the velocity of a propagation of the front, called the celerity of a translator wave; h_1 is the height of the dam before the destruction (about 90 m).

The unknown values are h_f, v_f, ξ and a velocity c_f depends on the depth h_f - $c_f = \sqrt{\frac{gkh_f}{k+1}}$. The velocity c_f is known as the celerity of the small amplitude or a relative celerity $c_f = \xi - v_0$. It is clear that the velocity c_1 is equal to $c_1 = \sqrt{gh_1 \frac{k}{k+1}}$ and it is a constant. According to Stoker (Stoker, 1975) the depth h_f is a constant in the limits

$$
(v_f - c_f)T \le s \le \xi T \qquad (4)
$$

The form of the created wave could be extended in the negative region

$$-c_1 T \le s \le (v_f - c_f)T \tag{5}$$

where length s is the section, T is the time.

By the means of the system (3), inequalities (4), (5) and using the length $s = 0$ the main equations are calculated at the dam, in front of the dam and behind it (Figure 4).

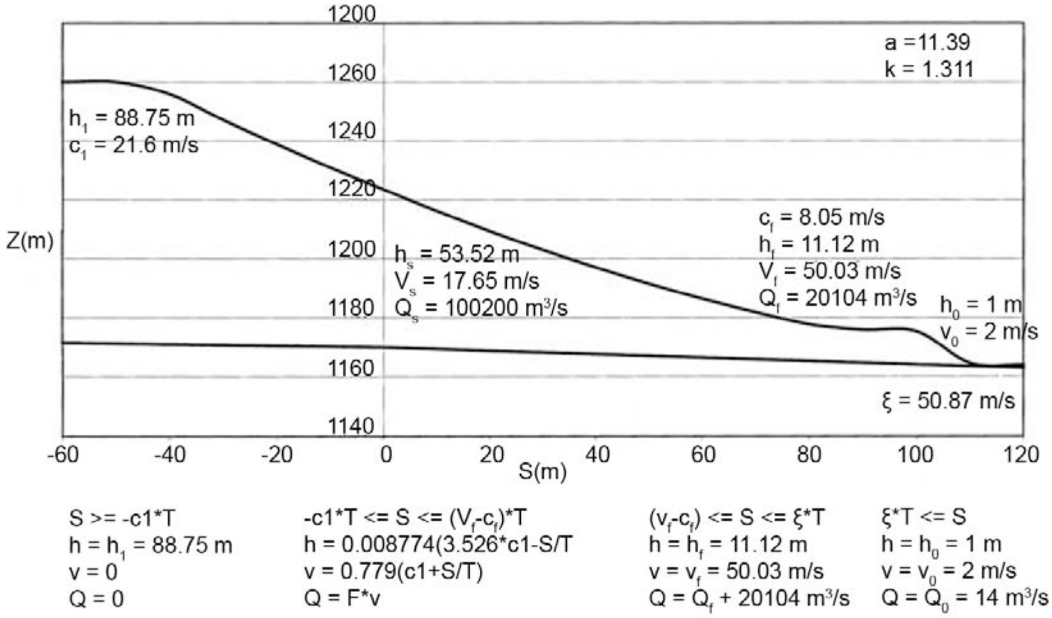

Fig. 4 Parameters of a catastrophic wave for a full destruction at $T = 2s$.

The further moving of the catastrophic wave should be reasonably good to use the classical Saint -Venant equations under the form

$$\left| \begin{aligned} &\frac{\partial v}{\partial t} + v\frac{\partial v}{\partial x} + g\frac{\partial h}{\partial x} + g(i_f - i_0) = 0 \\ &\frac{\partial h}{\partial t} + v\frac{\partial h}{\partial x} + \frac{k}{k+1}h\frac{\partial v}{\partial x} = 0 \end{aligned} \right. \tag{6}$$

where t is the time, x is the distance along the waterway, v is the velocity, h is the water depth, g is gravity acceleration, i_f is the friction slope, i_0 is the slope of the bottom, k is the shape parameter.

The finite difference method was introduced on the bases of explicit scheme proposed by Leleve (Richtmyer, 1960). This numerical method is preferred because its simplicity and because it represents the solution of the explicit scheme as a linear function of every cell. At the same time the condition for convergences and stability exist for every time step. This means that a maximum Currant number is

$$t \le \frac{x}{(|v| + c)_{\max}} \tag{7}$$

where velocity is given by $c = \sqrt{gh\frac{k}{k+1}}$.

The values of the limits of the cell are computed at the time $t_n + \ t$ and at space $x_i \pm 0.5\ \ x$ .The unknown values of the functions are

$$\begin{cases} v_i^{n+1} = v_i^n + \frac{tg}{x}\left[h_{i-1/2}^n - h_{i+1/2}^n + v_i^n \frac{v_{i-1}^n + v_i^n}{x} - (i_f - i_0)\ \ x\right] \\ h_{i+1/2}^{n+1} = h_{i+1/2}^n + \frac{t}{x}\left[v_i^{n+1}(h_{i-1/2}^n - h_{i+1/2}^n) + \frac{k}{k+1} h_{i+1/2}^n (v_i^{n+1} - v_{i+1}^{n+1})\right] \end{cases} \qquad (8)$$

The boundary condition at the place of the reflected negative wave has the form of

$$h_i^{n+1} = h_i^n + \frac{t}{x}\left[\left(v_i^n - c_i^n\right)\left(\frac{c_1}{g}\left(v_{i+1}^n - v_i^n\right) - h_{i+1}^n + h_i^n\right) - \frac{c_1}{g} v_i^n + \frac{c_1}{g}\ \ t(i_f - i_0)\right] \qquad (9)$$

FULL DESTRUCTION OF THE DAM

Graphical illustration of the hydrograph $Q(t)$ imposed at the upstream end of the river in Figure 5.

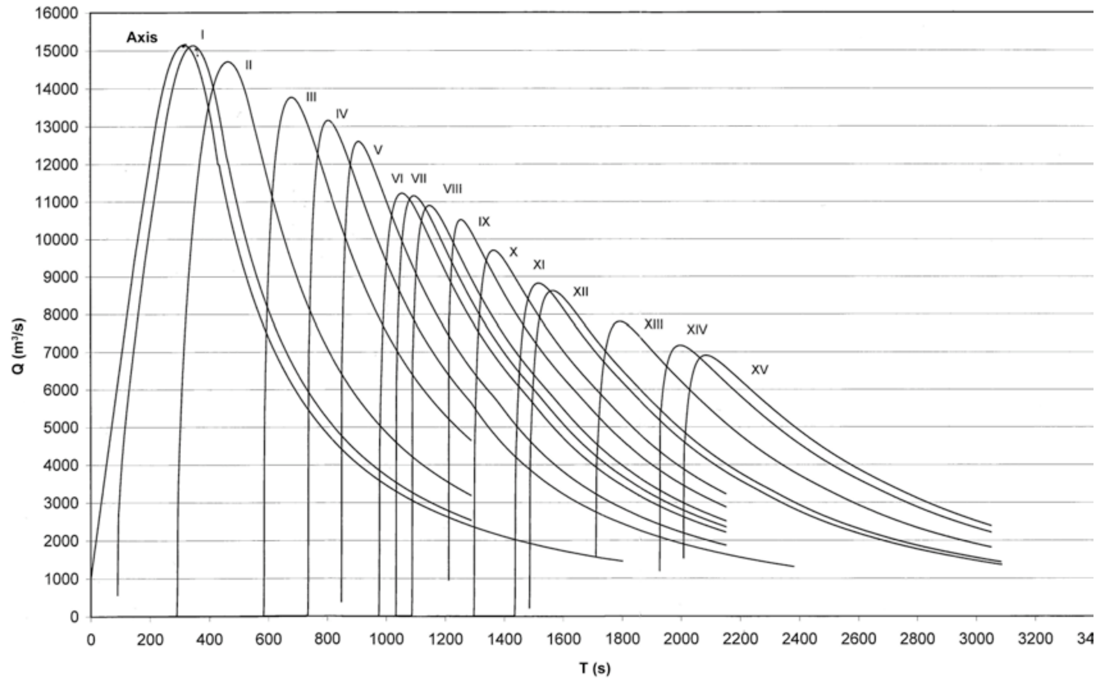

Fig. 5 Hydrograph $Q(t)$ of the consolidated sections

The axis of the dam are represented in the beginning of the hydrograph that coincide with the maximum water discharge $Q_{max}\ \ 100.10^3\ m^3 s^{-1}$. The tendency of decreasing of the water discharge is observed near the mouth of the river – section XV with a water discharge of $26\,900\ m^3 s^{-1}$. At the upstream end of the river the flood hydrograph is a function of the time and it is used as a boundary condition. On the table below are presented some numerical results from the full destruction of the dam.

Table 1 Full destruction of the dam

Sections	Length (km)	Elevations (m) Max water level	River bottom	Max depth (m)	Real time for Max level (s)	Maximum water discharge (m^3s^{-1})	Velocity of the stream (m s^{-1})
1	2	3	4	5	6	7	8
I	0,020	1216,5	1168,8	47,7	5	97700	20,9
	0,200	1200,6	1158,3	42,3	20	90700	23,8
	0,400	1185,6	1144,7	40,9	40	87300	24,2
II	0,620	1173,9	1133,9	40,0	45	85300	24,5
	0,700	1174,1	1130,9	43,2	50	93800	20,6
	1,200	1156,4	1116,3	40,1	75	87400	22,0
	1,800	1137,4	1098,8	38,6	105	85500	21,6
III	2,380	1121,1	1081,8	39,3	135	83500	19,3
	2,500	1118,2	1077,5	40,7	140	82000	21,8
	3,500	1079,6	1041,1	38,5	195	75000	21,5
	4,500	1041,8	1004,7	37,1	250	66800	21,2
	5,400	1007,5	972,0	35,5	300	60800	21,0
………							
XIV	18,300	387,6	367,6	20,0	1150	34600	10,7
	18,900	372,9	353,4	19,5	1200	32600	10,5
	19,500	358,3	339,2	19,1	1260	30700	10,3
	20,200	341,3	322,6	18,7	1330	28900	10,1
XV	20,300	338,0	319,4	18,6	1335	28500	10,1
	20,500	333,9	315,4	18,5	1350	27900	10,0
	20,700	329,1	310,7	18,4	1370	27400	9,9
	20,860	325,2	306,9	18,3	1395	26900	9,8

PARTIAL DESTRUCTION OF THE DAM

The formation, moving and further stratification of the catastrophic wave are considered in the case of the partial dam destruction. Assuming that the schematic breach of the dam is formed at the top of the dam and under the overflow level. Let as presume that the breach has appeared for a very short time and the size of a rectangular breach is 30/100 m i.e. 30 meters depth and 100 meters width. This breach operates as a spillway from the hydraulic standpoint at the beginning. The volume is 9.5 million cubic meters having in mind the curve of the reservoir volume. The following table gives an idea for the different time periods, weir head, and water discharge and water volume.

Table 2 Time of overflow periods.

Period	Time step t (s)	Head of spillway (m)	Water discharge Q (m^3s^{-1})	Volume W $(m^3\,10^6)$	Sum $\sum W$ $(m^3\,10^6)$	Time T (s)
1	300	23.8	15 100	4.53	4.53	300
2	320	14.3	7 000	2.24	6.77	620
3	330	9.5	3 800	1.25	8.02	950
4	350	6.5	2 150	0.75	8.77	1300
5	487	5.1	1 500	0.73	9.50	1787

A hydrograph $Q(t)$ is shown in the next figure for all sections along the river Yadenitsa.

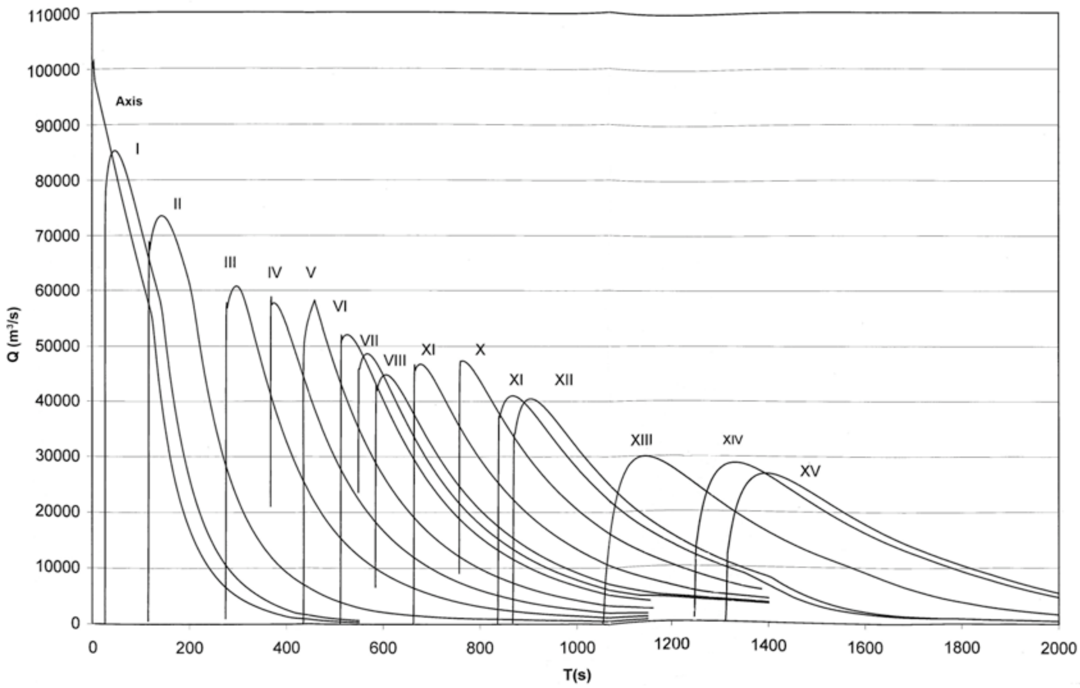

Fig. **6** Hydrograph of all section.

At the upstream end of the river the flow hydrograph, imposed by the breach, should be used as a boundary condition for the numerical modeling. The comparison between the full and the partial destructions shows evident differences between the maximum depth, water discharge and velocity. For example the maximum water discharge of the full destruction has values which are four to six time bigger. The maximum velocities reduce twice while the maximum depth decreases 45 %.

TRANSFORMATION OF THE CATASTROPHIC WAVE

The transformed wave is observed in two directions – upstream and downstream flow of the R2 river. Numerical modeling is performed by the means of system (6). The balance equations (1) are included in the above mentioned system. Hydrological data (GIS) are used as a boundary condition to solve exactly the system of four equations. The positive wave from downstream of R2 of river is coming from the perturbation of the catastrophic wave. After the running of the tributary (Yadenitsa river) the reduction of the water discharges and velocities decrease considerably because of the smaller bed slope. The positive wave from downstream has a maximum discharge of 21.8 10^3 m^3s^{-1} and the velocity is about 9 m s^{-1}. The maximum depth reaches 12.8 meters. These numerical results show a potential overflow through the constructed dikes and a risk for flooding. The negative wave from upstream of R2river indicates a further decrease of the hydraulic parameters of the R2 river.

CONCLUSION

Recently many dam break modeling techniques have considerably contributed to the overcoming of the destruction caused by the catastrophic wave. To validity of a 1-D model for simulating dam break wave is a numerical engineering problem (Paquier, 1996). This study a mathematical model for a full and partial destruction of a dam is presented. The main results summarized and compared the both cases and provide a possibility for an effective protection of urban areas and for providing of economic losses. Finally we could say it would be better to use some different technique for partial destruction that happens more often in practice.

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