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TWO-DIMENSIONAL MODELLING OF ACCIDENTAL FLOOD WAVES PROPAGATION

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Rezumat. *Studiul prezentat în acest articol descrie o metodologie modernă de modelare a propagării inundaţiei accidentale în cazul cedării unui baraj; această metodologie se aplică în România pentru prima oară într-un proiect pilot „Scenarii de cedare a barajului Poiana Uzului". Programele de calcul folosite realizează o modelare bidimensională (2D) a propagării undelor de viitură, luând în considerare şi atenuarea inundaţiei pe o direcţie normală la direcţia principală de curgere; atenuarea inundaţiei este foarte importantă în cazul cursurilor sinusoidale sau cu aşezări urbane foarte apropiate de albia râului. În cazul barajului de la Poiana Uzului, au fost simulate 2 scenarii cu ajutorul Prof. dr. ing. Dan Stematiu, dar cu şanse foarte mici de producere. Rezultatele au fost prezentate animat cu suprafeţe inundate în paşi succesivi.*

Abstract. *The study presented in this article describes a modern modeling methodology of the propagation of accidental flood waves in case a dam break; this methodology is applied in Romania for the first time for the pilot project „Breaking scenarios of Poiana Uzului dam". The calculation programs used help us obtain a bidimensional calculation (2D) of the propagation of flood waves, taking into consideration the diminishing of the flood wave on a normal direction to the main direction; this diminishing of the flood wave is important in the case of sinuous courses of water or with urban settlements very close to the minor river bed. In the case of Poiana Uzului dam, 2 scenarios were simulated with the help of Ph.D. Eng. Dan Stematiu, plausible scenarios but with very little chances of actually producing. The results were presented as animations with flooded surfaces at certain time steps successively.*

Keywords: breaking scenario, accidental flood wave, digital terrain model, bidimensional hydrodynamic calculation, flooded area

1. Introduction

The theme of the current study is the achievement of several real scenarios of breaking of Poiana Uzului Dam (see fig. 1). A 2D model representing the major river bed downstream the dam was created, stretching on 26 Km. The problem is solved by making a bidimensional hydraulic calculation by using the HYDRO_AS-2D program. This program is capable of reproducing the conditions of the flow in a nonpermanent mode, flow mode that is registered after a dam brakes.

The results of such calculations can be used at the interpretation of the flood wave and at the realization of the risk maps in case that Poiana Uzului dam breaks.

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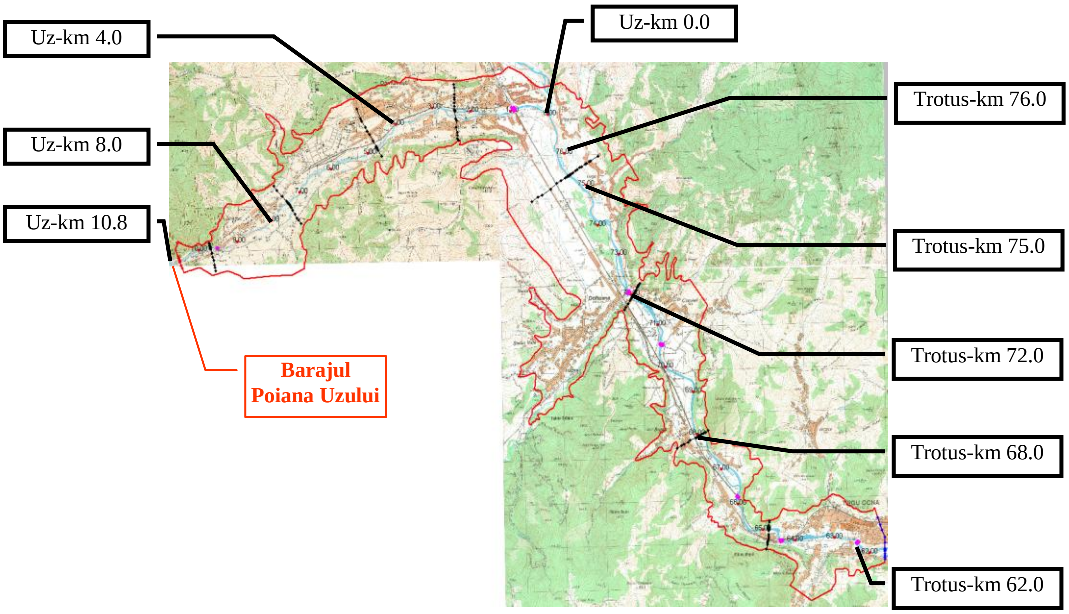


Fig. 1. Layout of investigated area.

2. Limits and characteristics of the investigated area

The investigated area is represented by the upstream part of Uz River, in between Poiana Uzului dam and the junction with Trotus River, and of the upstream part of Trotus River, from the junction with Uz River up to the outside border of Targu Ocna town.

Poiana Uzului dam is situated at km 10.8 of Uz river and Targu Ocna town is located at km 62.0 on Trotus river (15 km upstream of the junction with Uz river), meaning that a 26 km sector was modeled.

The slope of Uz River between the dam and the junction is of about 1.1%, while Trotus slope on the analyzed sector is of about 0.5%.

Consequently, it is only natural that the breaking of the dam produces a very high flood wave due to the steep slopes.

The river beds of the 2 sectors are formed mainly of grovel and River stones.

At the time of the visit, the water width was of about 15-20 m on Uz (fig. 2) and of 30-80 m on Trotus (fig. 3).

As one can observe in the photos, both rivers undergo a strong tendency to alter the course of the minor bed during the flooding. There are many areas with bank erosion, thus puts in danger the outskirts. At the site visit, nowhere on the investigated places exist protections or smaller dams.

Fig. 2. Uz river (km 10.0) on site visit day.

Fig. 3. Trotus river (km 72.0) on site visit day.

3. Basic data

For the creation of the model and for the calculation of the flood discharge speed, the following basic data was used:

- lake volume curve;

- situation plans and longitudinal profiles of the Poiana Uzului dam;

- information about the length of the two rivers;

- topographical data resulted from the field measurements (10000 points consisting of approx. 80 transverse sections and a weight plan on a 200-800 m along minor bed);

- 1:25000 maps;

- photos made during the site visit.

4. Breaking scenarios

Poiana Uzului dam is a buttress type dam, 300 m long at the base and 500 m at the cornice. The maximum height of the dam is of 82 m. The dam is made up of 33 plots, out of which 3 are down-comers and the others are normal plots. Bottom outlet is done through a metallic pipe with a 1.5 m in diameter at the base of each down-comer plot.

The maximum water volume of the lake is of approx. 90.000.000 m^3. The two breaking scenarios were materialized with the help of Ph.D. Eng. Dan Stematiu from the Hydrotechnic Structures Faculty inside UTCB and is based on his knowledge on the existing problems in 2 particular areas of the dam, knowledge acquired during the design period and while analyzing the behaviour of the construction. A potentially fragile area is around plot 9, and another one at the bottom outlet pipe on plot 19.

An important issue is that the scenarios described here-under, present a very low risk and that the analyzed areas do not endanger the safety of the dam.

Scenario I

The first scenario is graphically presented in fig. 4 and 5. It is structured in 4 stages:

<u>STAGE 1</u>: In a first stage there is a leak along the bindings of plot 9 and its 75 m element starts sliding; the more it slides the more the discharge flows at the contact with plots 8 and 10 (stages 2 and 3). In stage 4 (fig. 4) plot 9 has completely fallen. The opening created by the fall of the plot has a width of approx. 15 m, and the discharge reaches the value of 12000 m^3/s (fig. 6).

<u>STAGE 2</u>: Due to erosion on both sides, plot 8, with a height of 65 m becomes unstable and leans towards plot 10, sliding downstream in the same time; due to all this, water discharges also in between plots 8 and 7. At the end of stage 2, plot 8 has fallen and the opening is 30 m wide, the discharge reaching 18000 m^3/s.

<u>STAGE 3</u>: Due to side erosions, plot 7, with a height of 55 m becomes unstable and leans towards plot 10, sliding downstream in the same time (stage 6). At the end of this stage, plot 7 slides completely and collapses. At that point, the opening measures 45 m in width and the water discharge reaches the maximum value of 22500 m^3/s.

<u>STAGE 4</u>: In this last stage, the lake empties through the 45 m wide opening. Due to the decrease in the water volume, the discharge diminishes during a period of 110 minutes until it reaches the value of the normal discharge (see fig. 6). As you can observe in figure 5, the hydrograph of the breaking has a very fast growth and the emptying of the lake is completed in 148 minutes from the accident.

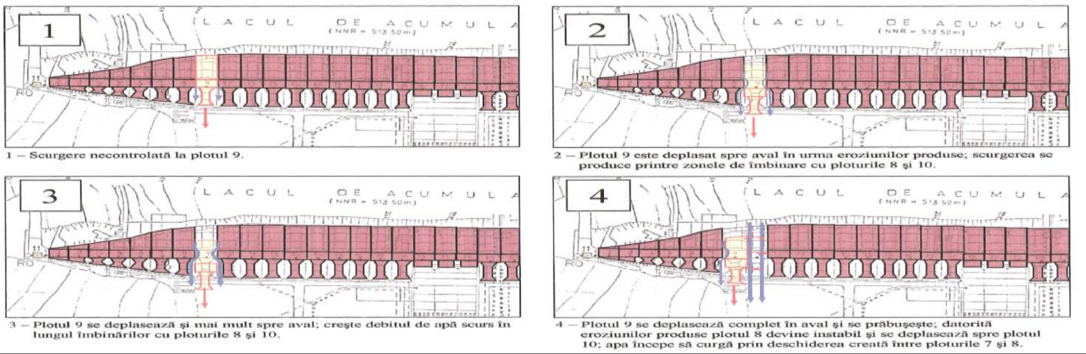


Fig. 4. Scenario 1 – Stage 1 and 2.

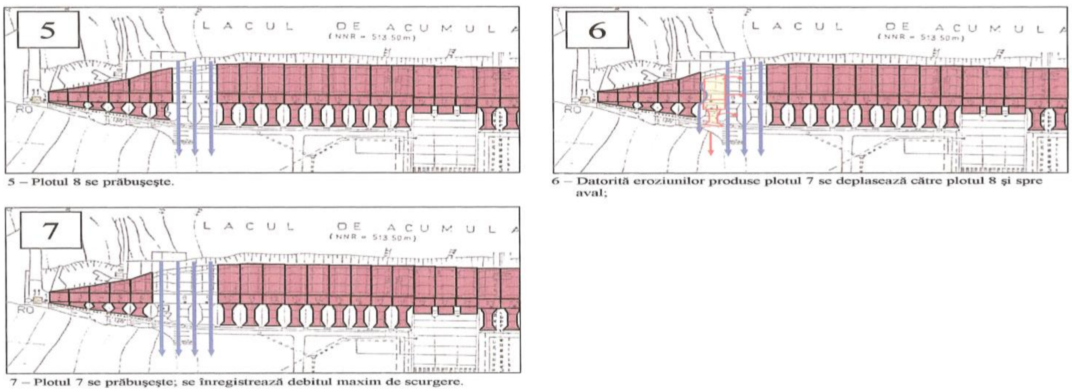


Fig. 5. Scenario 1 – Stage 3 and 4.

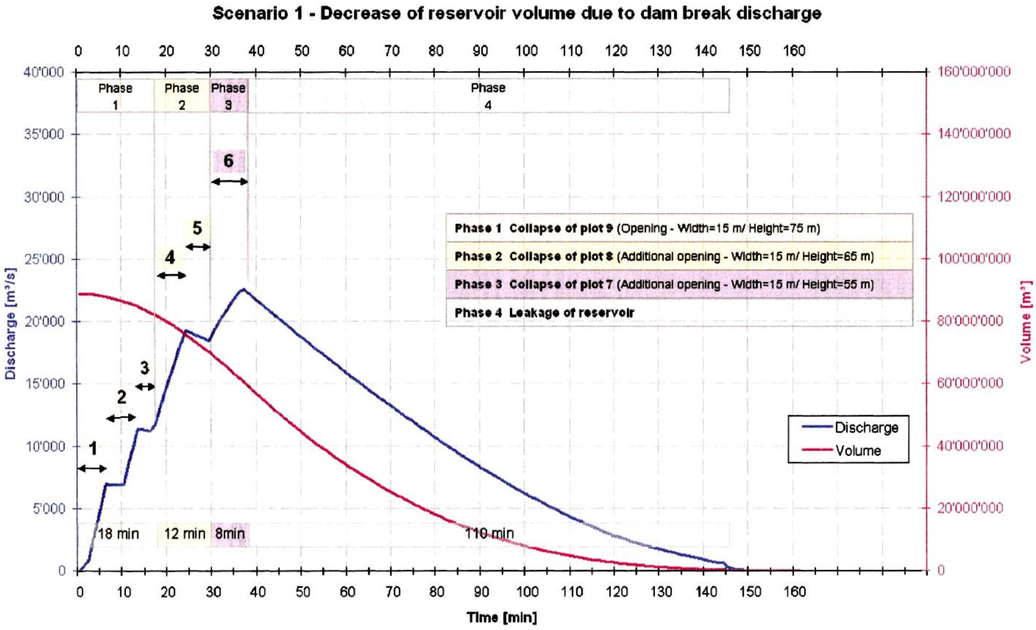


Fig. 6. Scenario 1 – Discharge hydrograph.

Scenario II

The second scenario is graphically presented in fig. 7 and 8. The resulted hydrographs are presented in fig. 9. This scenario is also structured in 4 stages:

<u>STAGE 1</u>: In a first stage there is a breaking in the bottom outflow pipe inside plot 19 and the filling material in between the plots is washed away. The maximum discharge has a small value still, and is estimated at around 100 m^3/s.

<u>STAGE 2</u>: In the second stage, plot 19, with a height of 80 m starts sliding and the water begins to leak through the bindings with plots 18 and 20. At the end of this stage, plot 19 slides completely and collapses. The opening measures approx. 15 m in width, and the water discharge reaches a value of 12000 m^3/s (fig. 9).

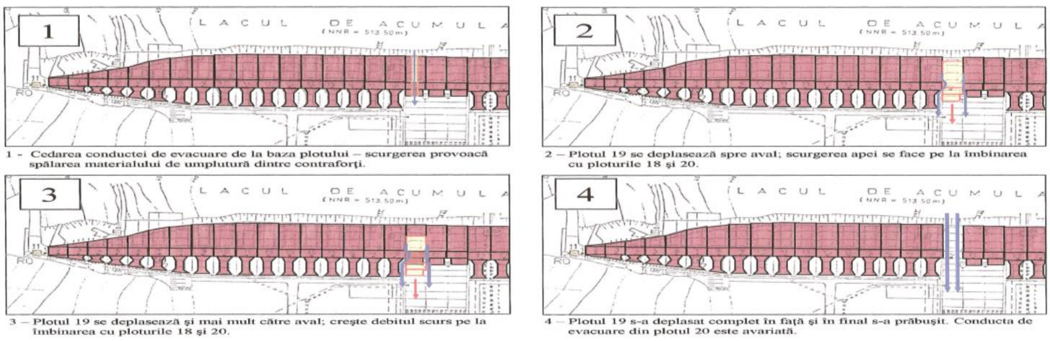


Fig. 7. Scenario 2 – Stage 1 and 2.

STAGE 3: The bottom outflow pipe inside plot 20 is damaged and, in a similar way as that of stage 2, plot 20, with a height of 80 m, will be destabilized. Due to the loss of the stabilizing filling, plot 20 becomes unstable, leans towards plot 18 and starts sliding. The water also flows through the bindings of plot 20 and 21. At the end of stage 3, plot 20 hours collapsed and the opening measures now 30 m in width. The discharge now reached the maximum value of 21700 m^3/s.

STAGE 4: During the last stage, the lake empties through the 30 m wide and 80 m tall opening. Due to the decrease in the water volume, the discharge diminishes during an 80 minute period, until the value of the discharge reaches its normal value (see fig. 9). Similar to scenario 1, the hydrograph of the breaking in scenario 2 has a very fast growth time and the emptying of the lake ends 155 min after the accident (see fig. 9). The two hydrographs for scenario 1 and 2 will constitute the boundary conditions for the 2D calculation.

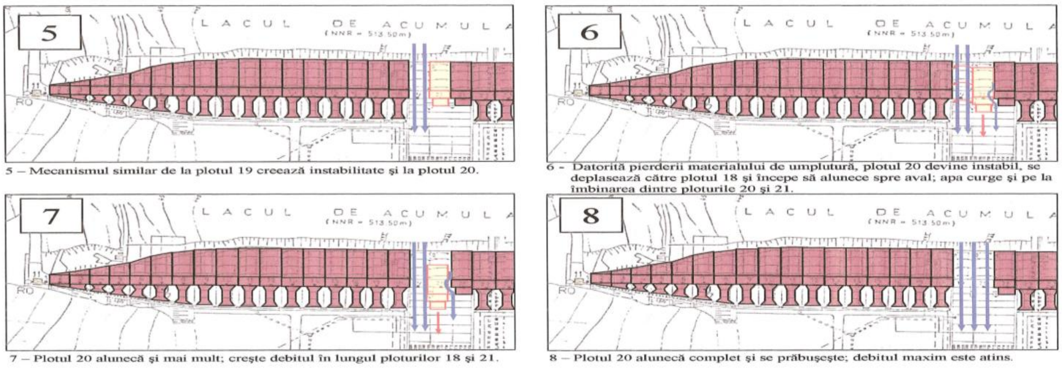


Fig. 8. Scenario 1 – Stage 1 and 2.

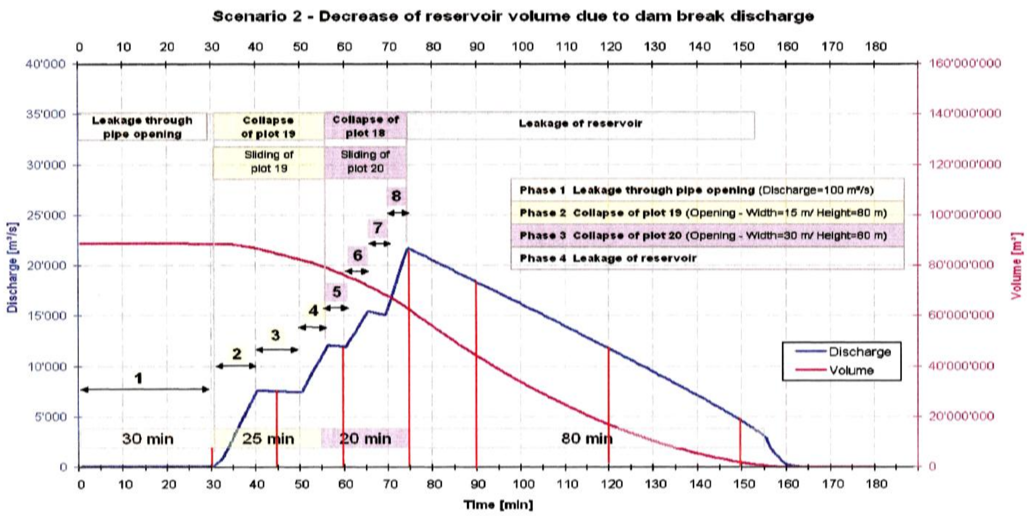


Fig. 9. Scenario 2 – Discharge hydrograph.

5. The calculation method for the propagation of the accidental flood wave

The propagation of the flood wave and the extension of the flooded areas were determined through a two-dimensional hydrodinamical calculation, with HYDRO_AS-2D computer program. The modeling of the discharge hydraulic conditions is based on the numeric solution of the bidimensional equation of medium water depth with Galerkin type volume elements. In this case, the discharge speed is approximated with respect to the water depth. In order to simulate such flow, a rectangular and triangular elements model was created, having a series of pre-established initial and boundary conditions. The calculation is controlled by the pre-established time step and by the total calculation duration.

The general modeling and calculation procedure is presented here-under.

5.1. Programs used for modeling and calculations

Model creation and flow calculation were realized with the help of two computer programs:

- Surface Water Modelling System (SMS) from BOSS International INC, USA and

- HYDRO_AS-2D created by Dr. Nujic, Rosenheim, Germany.

So, SMS is a pre- and post-processing program with which one can create a network of elements constitutes the digital terrain model and there are defined the boundary conditions, the control structures of the discharge (such as weirs and bridges) and control points of the discharge. With the help of SMS computer program, a linear unstructured network was modeled by using triangular and rectangular elements. This means that the model can be perfectly adjusted after the contour surface. The data regarding the relief, the inner part of the blowers of the bridges and the control points of the discharge are connected to the elements' knots and the roughness is attached to each element.

5.2. Pre-processing (creation of the model)

For the creation of the digital model of the land, topographical data were used to sum up approx. 10000 marking points, representing approx. 80 transverse sections and multiple points on a weight plan with a width of 200-800 m along the minor river bed of the analyzed sector. The model was then created on the basis of these measurements and on the basis of the photographs and information gathered on the site visit. (fig. 10, 11 and 12).

The standard values of the roughness's from the minor and major river bed were established on the basis of the information gathered during the site visit and of the existing maps. The roughness from the major river bed were estimated on the basis of the experience in the realization of more than 50 such projects in 2D modeling. A Strickler coefficient (opposite of Manning coefficient) equal to 5 (n=0.20) was established for the inhabited areas. The vegetation, such as bushes and the wooded areas have a value of Kst = 5...15 according to the density of the vegetation. The roughness of the minor river bed were settled at Kst = 25 (fig. 13). The bridges were introduced by defining the inferior level and by the disconnection of piers and abutments from the model (fig. 14). Considering that the level of water will be higher than that of the bridges, the discharge flowing over the bridge is calculated by applying the Poleni equation for flow over the weir.

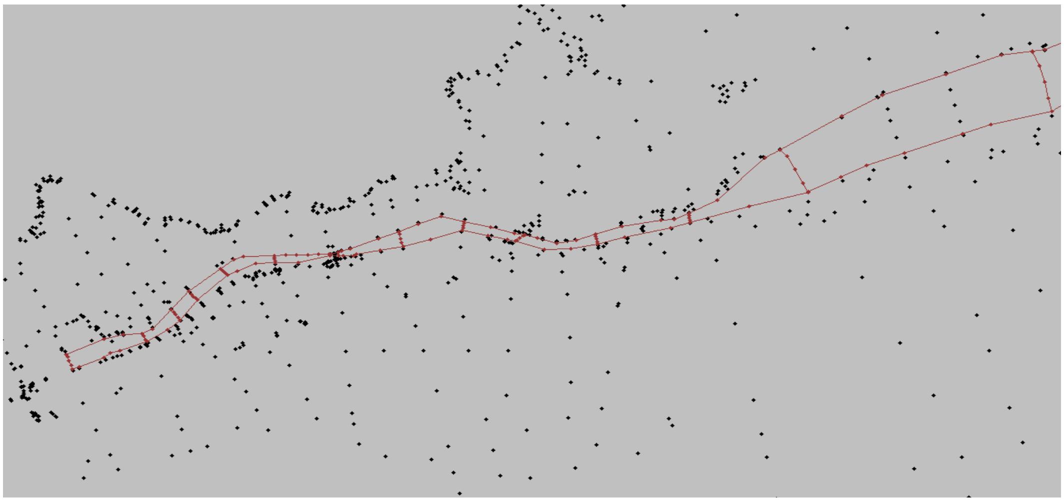


Fig. 10. Creating the discretisation network of the minor river bed by multiplying the cross sections.

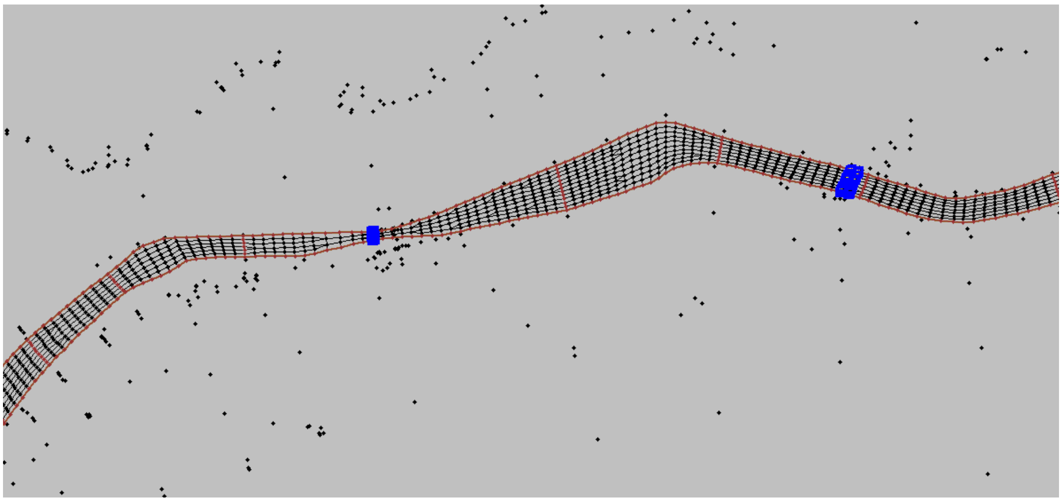


Fig. 11. Creating the rectangular elements of the minor river bed.

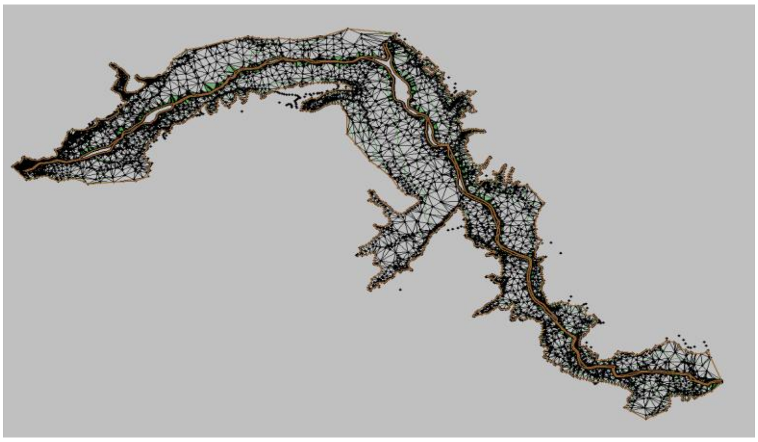


Fig. 12. Creating the discretisation network of the major river bed by multiplying the points on the areas with fragmented relief.

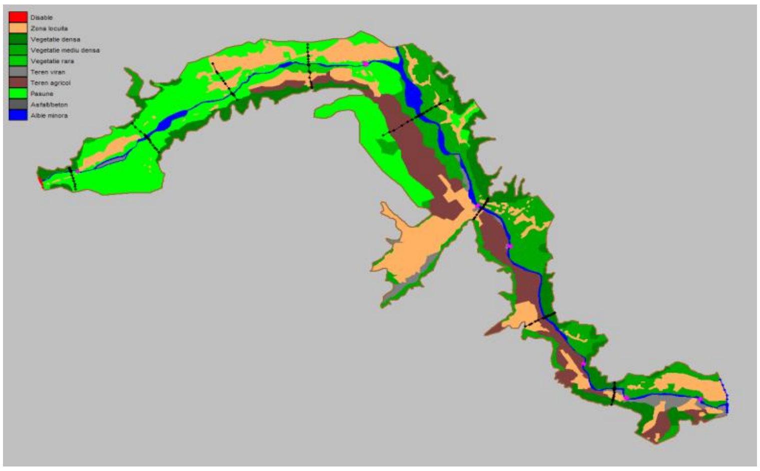


Fig. 13. Land uses for the analyzed sector.

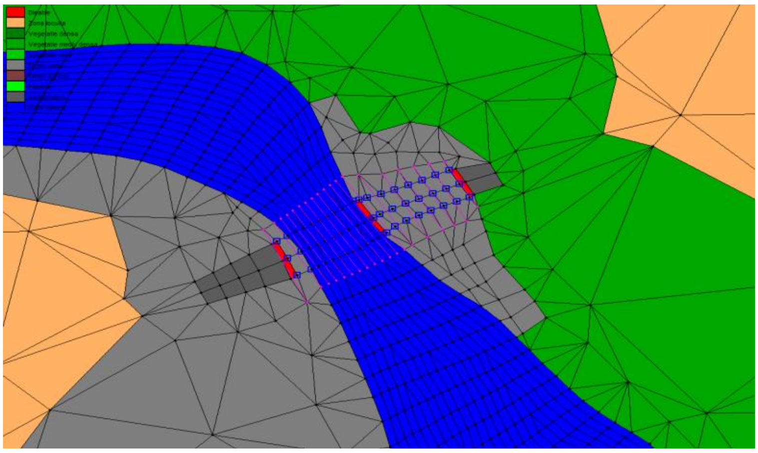


Fig. 14. Boundary conditions for defining a bridge.

Implementation of the initial and boundary conditions. Model calibration.

The initial conditions refer to the overall calculation time and to the time step, parameters considered to be the global parameters of the model. In the case of Poiana Uzului pilot project, the overall calculation time was Tt = 40000 s, and the time step t = 300 s. The boundary conditions are implemented at the inlet and outlet of the model and they are necessary for solving the equations system during the calculation.

For the two scenarios an „inflow hydrograph" was introduced at the inlet of the model (fig. 15), and downstream, at the outlet of the model, an „energetic slope" was defined (fig. 16). Due to the non-existence of a similar incident in the history of the area, the calibration of the model could not follow a real event. The calibration might have helped to obtain an accurate adjustment of the roughness coefficients. Due to the lack of such data, the calibration of the model was not possible. As mentioned before, the value of the roughness in the minor and major river bed was approximated on the basis of informations gathered during site visits and existing maps.

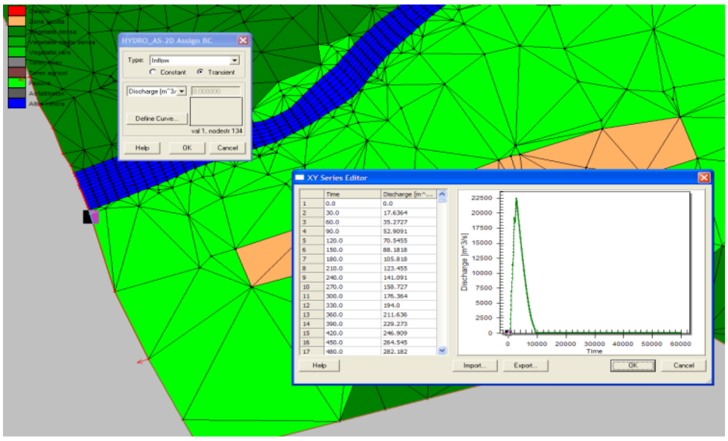


Fig. 15. The inflow hydrograph at the inlet of the model.

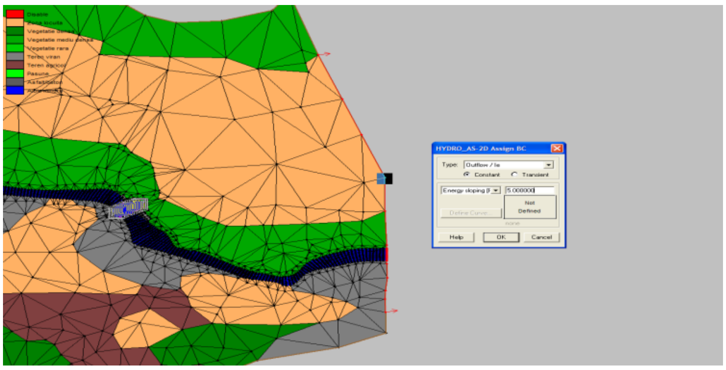


Fig. 16. The outflow condition at model exit.

5.3. Numerical calculation

The numerical calculations were done with HYDRO_AS-2D computer program (Prof.PhD. Marinko Nujic, Germany).

The program uses a equations system derived from the basis Barre de Saint Venant equations for non-permanent water flow in open channels. In 1999 Prof. Ph.D. Marinko Nujic write a compact form of those equations that can easily be utilized by the program:

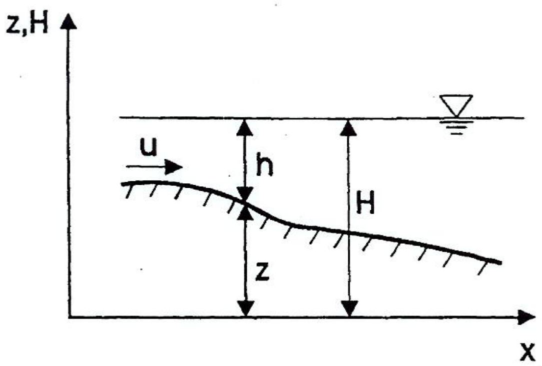
$$\frac{\partial \mathbf{w}}{\partial t} + \frac{\partial \mathbf{f}}{\partial x} + \frac{\partial \mathbf{g}}{\partial y} + \mathbf{s} = 0$$

where

$$\mathbf{w} = \begin{bmatrix} H \\ uh \\ vh \end{bmatrix} \qquad \mathbf{f} = \begin{bmatrix} uh \\ u^2 h + 0.5gh^2 - \nu h \dfrac{\partial u}{\partial x} \\ uvh - \nu h \dfrac{\partial v}{\partial x} \end{bmatrix}$$

$$\mathbf{s} = \begin{bmatrix} 0 \\ gh(S_{fx} - S_{bx}) \\ gh(S_{fy} - S_{by}) \end{bmatrix} \qquad \mathbf{g} = \begin{bmatrix} vh \\ uvh - vh \dfrac{\partial u}{\partial y} \\ v^2 h + 0.5gh^2 - vh \dfrac{\partial v}{\partial y} \end{bmatrix}$$

with $H = h + z$ – water level above the reference level;
u, v – velocity in x and y directions;
S_f – contains the terms for roughness;



$$S_{bx} = -\frac{\partial z}{\partial x} \qquad S_{by} = -\frac{\partial z}{\partial y}$$

$$S_f = \frac{\lambda v|v|}{2gD} \qquad \lambda = 6.34\frac{2gn^2}{D^{1/3}}$$

S_{bx}, S_{by} – bed slope on x and y directions, n – Manning coefficient, g – gravity and D – hydraulic diameter = $4r$.

The results are written in a number of folders and presents: water level, water velocity and discharge for each point of the mesh. With these values, other parameters can be calculated: Froude number, shear stress etc.

5.4. Post-processing

After numerical calculation, the results can be imported in SMS. Thus, the flooded surface can be presented and the water depth can be determined. Layers with these parameters at each time step can be exported in GIS programs, this way the structural or non-structural decisions can be easily made.

For Poiana Uzului dam brake scenarios, flood wave propagation times were determined. Tabels 1 and 2 contains this parameters for four localities downstream the dam. Figures 17, 18, 19 and 20 shows the propagation wave at certain time steps for scenario 1.

Table 1. Propagation times if scenario 1 takes place.

Localitatea	*Distanţa faţă de baraj*	*Timpul de propagare*
Sălătruc	2 km	10 min
Dărmăneşti	6 km	30 min
Dofteana	11 km	70 min
Târgu Ocna	25 km	105 min

Table 2. Propagation times if scenario 2 takes place.

Localitatea	*Distanţa faţă de baraj*	*Timpul de propagare*
Sălătruc	2 km	40 min
Dărmăneşti	6 km	50 min
Dofteana	11 km	100 min
Târgu Ocna	25 km	130 min

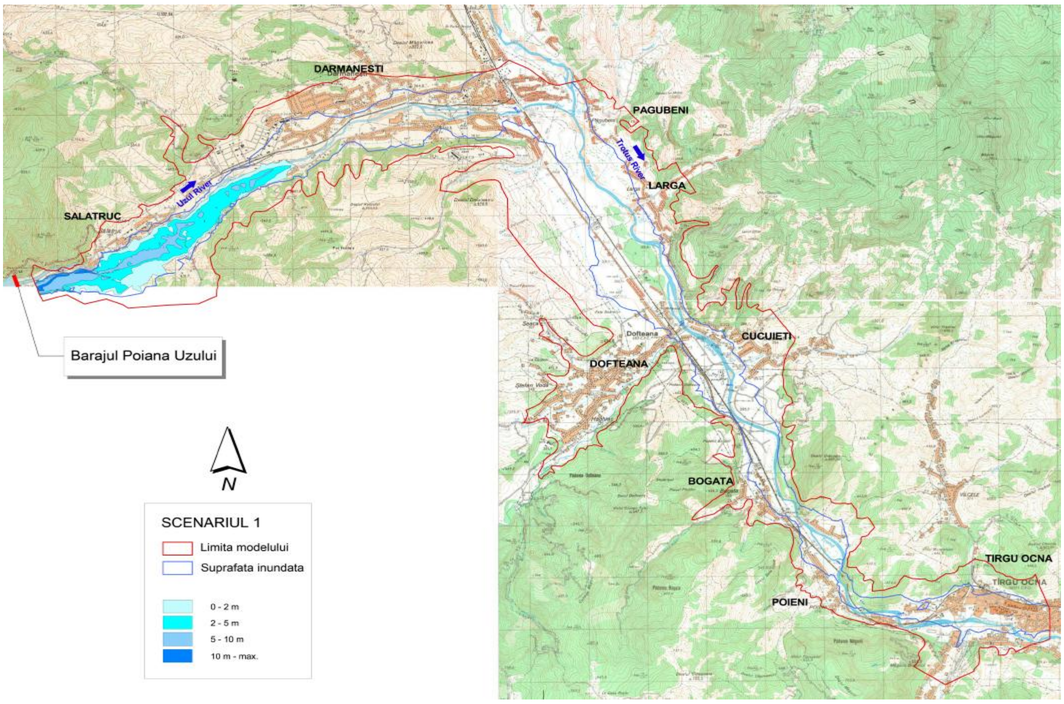


Fig. 17. Flood wave at 30 min after dam breaks.

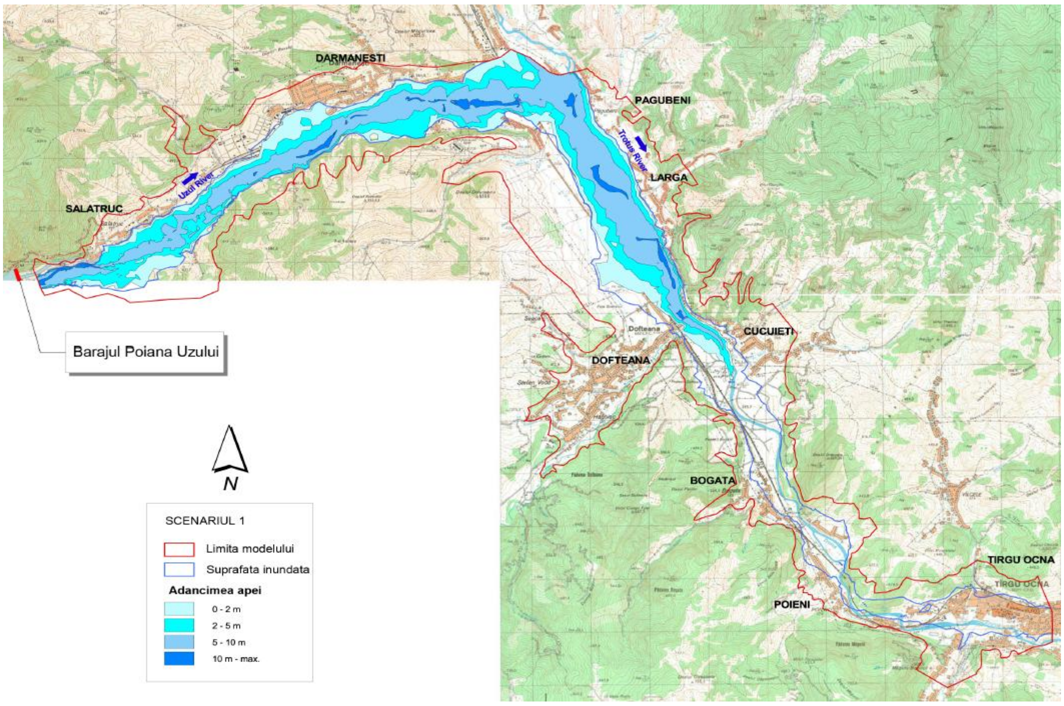


Fig. 18. Flood wave at 75 min after dam breaks.

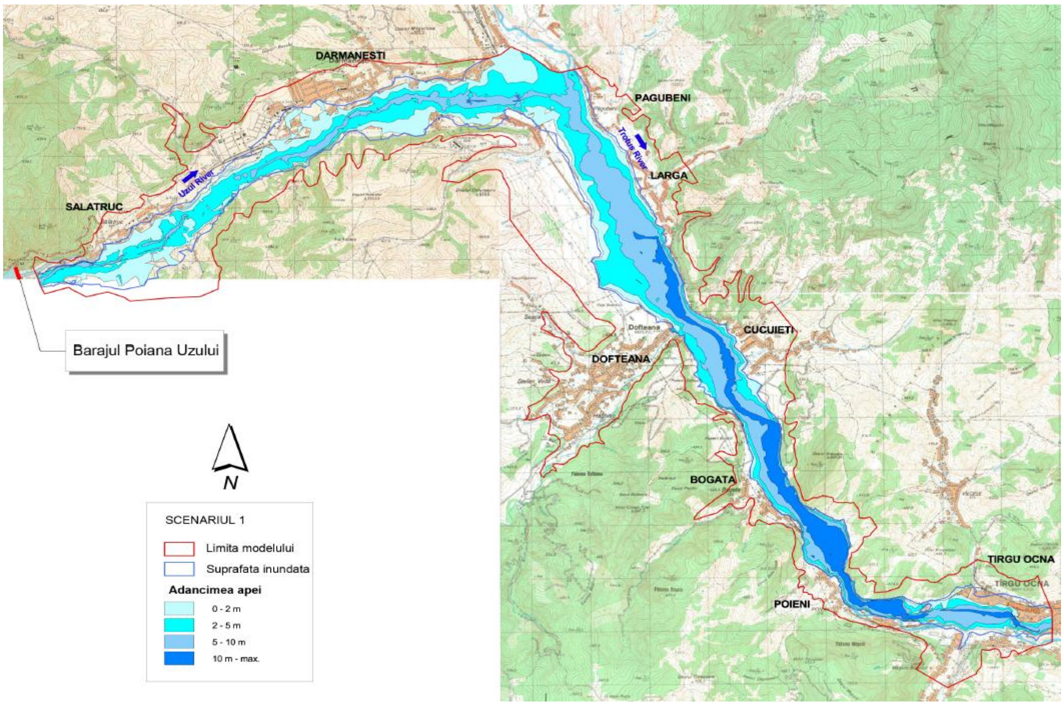


Fig. 19. Flood wave at 120 min after dam breaks.

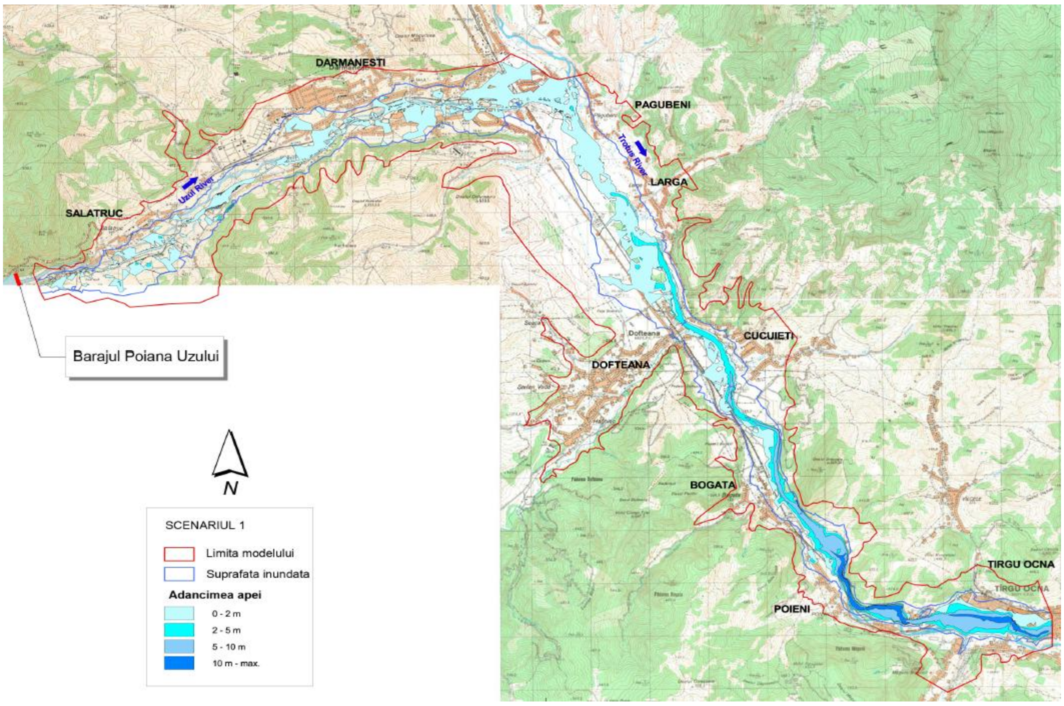


Fig. 20. Flood wave at 240 min after dam breaks.

Conclusions

To know the propagation times in case of a breaking accident at Poiana Uzului dam, two breaking scenarios were modelled. In such case a hydrodynamic two-dimensional calculation was necessary and it was done with SMS/HYDRO_AS-2D computer programs on the base of topographical surveys.

The two scenarios were elaborated with the help of Prof. Ph.D. eng Dan Stematiu from UTCB, the result being entered in the model as breaking hydrographs.

After calculations have been done, it can be observed that water depth reaches about 18 m and velocity takes values from 0 m/s to 9 m/s. Such an event could modify river bed morphology through significant erosions and deposits of alluvial material. The force of such event could destroy all the bridges and other constructions existing in the river bed.

Propagation times are very short, especially for localities that are near the dam (ex. Salatruc). Even for Targu Ocna city, located at 25 km downstream the dam, propagation times are insufficient to take measures to evacuate people in case of an accident at the dam simultaneously with alarm system failure.

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