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Water Retention and Localisation

Theoretical background: Continuity equation in a reservoir

The attenuation represents the reduction in peak flow following the storage of water in a reservoir. It may be represented by the following differential equation, which is the equation of continuity applied to a reservoir:

$$dQ = Q(t) - q(t) = \frac{dV}{dt} \tag{1}$$

It states that the inflow *Q(t)* in the reservoir during an interval of time minus the outflow *q(t)* during the same interval is equal to the change in storage.

In the following, the time horizon is discretised into intervals of duration Δt, indexed by *i* The discretisation of the equation (1) means the replacement of differentials by finite differences:

$$\Delta Q = \frac{\Delta V}{\Delta t} \tag{2}$$

Basic data

To undertake the flood wave attenuation in a reservoir the following data are necessary:

a) The flood discharges ***Q*(*t*)**

The input can be a registered flood or a synthetic flood characterised by the probability of exceedance of the maximum discharge. The registered floods are processed in order to derive synthetic hydrographs, used for different hydraulic computations: deriving the flooded areas for medium or low probabilities of exceedance, designing the outlets of the dam and or establishing operation rules during exceptional floods.

A synthetic flood is usually defined by the following parameters (Figure 1):

$Q_{p\%}$ – maximum discharge corresponding to the probability of exceedance P%; it is obtained by statistical processing of the partial series of annual maximum discharges

T_{incr} – the increasing time of the flood hydrograph, from the beginning to the flood peak (as the average value of the most significative registered floods)

T_{tot} – total duration of the flood hydrograph (again as the average value of the most significative registered floods)

γ – the compactness (the shape) coefficient, computed also as the average value of the most significative registered floods:

https://doi.org/10.36250/01052_05

$$\gamma = \frac{W}{Q_{P\%}^{max}\ T_{tot}} \tag{3}$$

where W is the flood volume:

$$W = \int_0^{T_{tot}} Q\ (t)\ dt\ \approx \sum_{i=1}^{N} Q_i\ \Delta t \tag{4}$$

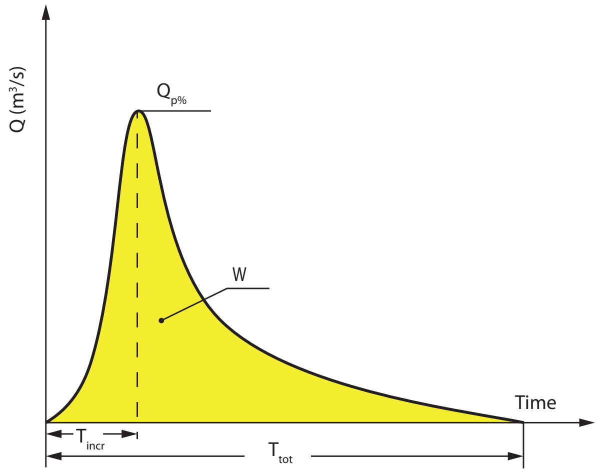


Figure 1. Parameters of the synthetic floods [5; 10]

Q_i represents discretised values of the discharges and Δt is the time step.

b) The elevation-storage ***V*(*H*)** relationship

The relationship $V(H)$ is derived by planimetering a topographic map at a convenient scale. If A_i and A_{i+1} are the areas delineated by the contour lines H_i and H_{i+1} of equal elevation above the mean sea level, the corresponding volume of water between these contour lines is:

$$\Delta V_{i,i+1} = \frac{1}{3}\ \left(A_i +\ A_{i+1} + \sqrt{A_i\ A_{i+1}}\right) (H_{i+1} -\ H_i) \tag{5}$$

The volume in the reservoir at the elevation H_{i+1} is obtained with the relationship:

$$V_{i+1}\ \ =\ \ V_i\ \ + \Delta V_{i,i+1} \tag{6}$$

A graphical representation of the elevation-storage curve is presented in Figure 2.

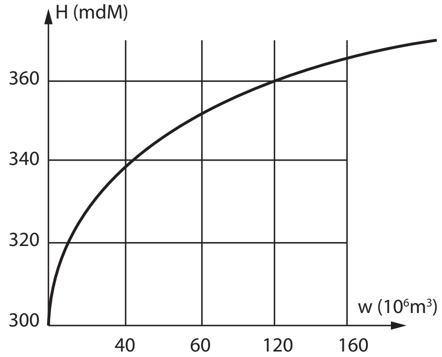


Figure 2. Elevation-storage curve [11]

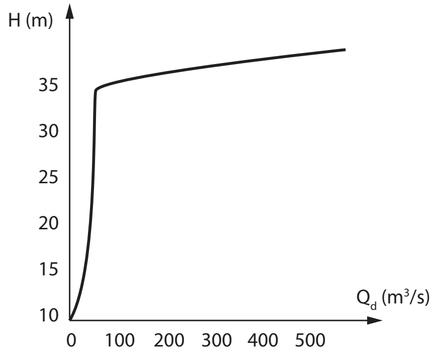


Figure 3. Elevation-outflow curve [11]

c) The elevation-outflow $\boldsymbol{q(H)}$ relationship

The elevation-outflow relationship (Figure 3) is obtained from hydraulic equations relating head and discharge. The discharge value for the bottom gates is differently computed according to the type of flow: free surface flow or pressure flow.

For the free surface flow through the bottom gates, the discharge computation is based on the well-known Chézy formula:

$$v = C\sqrt{RI} \tag{7}$$

$$Q = A\,C\sqrt{RI}$$

Where $C = \frac{1}{n} R^{1/6}$ – is the Chézy coefficient;

R – hydraulic radius
I – friction slope or hydraulic gradient
n – Manning's coefficient of roughness
A – area of the bottom gates

In case of pressure flow, the relationships used for velocity and discharge computation are:

$$v = \varphi\sqrt{2g\,H} \tag{8}$$

$$Q = A\,\varphi\sqrt{2g\,H}$$

Where H is the distance between the water level in the reservoir and the geometric centre of the gate. The increase of the discharge versus H is not very important due to the relatively low influence of $H^{1/2}$.

The flow from the reservoir above the spillway crest is computed with the following relationship:

$$Q = m\, b_c\, \sqrt{2g}\, h^{\frac{3}{2}} \tag{9}$$

Where m is a variable coefficient of discharge;
b_c – effective length of the crest (fluid contraction being considered)
h – total head on the crest including velocity of approach head

As opposed to the bottom gates, the increase of the water level in the reservoir above the spillway crest has a strong influence on the output discharges due to the power 3/2 affecting the head h.

Simulation models of flood wave attenuation in a reservoir

Using the values of the discharge at the beginning of each step of computation:

a) Equation (2) can be written in the following way:

$$Q_i - q_i = \frac{V_{i+1} - V_i}{\Delta t} \tag{10}$$

Where Q_i is the input flow in the reservoir at the time i while q_i is the output at the same moment; V_{i+1} and H_i are the water volumes in the reservoir at the time $i + 1$ and i respectively.

Consequently, the volume in the reservoir at the time i is:

$$V_{i+1} = V_i + (Q_i - q_i)\, \Delta t \tag{11}$$

The discharge $q_i = q(H_i)$, where H_i is the water level in the reservoir at the time i.

This approach can be used only when the time step Δt is small, being of the order of minutes. Otherwise, the errors of approximation of the flood volume are unacceptable.

b) Using the values of the discharge at the beginning and the end of each step of computation:

$$\frac{Q_i + Q_{i+1}}{2} - \frac{q_i + q_{i+1}}{2} = \frac{V_{i+1} - V_i}{\Delta t} \tag{12}$$

In this case, the volume V_{i+1} is:

$$V_{i+1} = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right) \Delta t - \frac{q_i}{2}\, \Delta t - \frac{q_{i+1}}{2} \Delta t \tag{13}$$

Which can be written:

$$V_{i+1} = \widetilde{V_i} - \frac{q_{i+1}}{2} \Delta t \tag{14}$$

Where

$$\widetilde{V_i} = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right) \Delta t - \frac{q_i}{2} \Delta t \quad (15)$$

The left term $\widetilde{V_i}$ is a constant for each computation step and can be obtained based on the known values of the input Q_i and Q_{i+1} (prespecified values) and the output q_i (calculated in the previous step). Thus, in the equation (14) there are 2 unknowns: V_{i+1} and q_{i+1} An iterative procedure can be used to evaluate the outflow q_{i+1} based on elevation-storage $V(H)$ and elevation-outflow $q(H)$ relationships. The iterative procedure for computing the outflow q_{i+1} consists of the following steps:

First iteration:

$$q_{i+1}^{(0)} = q_i \quad (16)$$

$$V_{i+1}^{(0)} = \widetilde{V_i} - \frac{q_{i+1}^{(0)}}{2} \Delta t$$

$$H_{i+1}^{(0)} = H^{-1}\left(V_{i+1}^{(0)}\right)$$

$$q_{i+1}^{(1)} = q\left(H_{i+1}^{(0)}\right)$$

Second iteration:

$$V_{i+1}^{(1)} = \widetilde{V_i} - \frac{q_{i+1}^{(1)}}{2} \Delta t \quad (16')$$

$$H_{i+1}^{(1)} = H^{-1}\left(V_{i+1}^{(1)}\right)$$

$$q_{i+1}^{(2)} = q\left(H_{i+1}^{(1)}\right)$$

The iterations continue until one or all criteria of stop are verified:

$$\left|V_{i+1}^{(k)} - V_{i+1}^{(k-1)}\right| \le \varepsilon_V \quad (17)$$

$$\left|H_{i+1}^{(k)} - H_{i+1}^{(k-1)}\right| \le \varepsilon_H$$

$$\left|q_{i+1}^{(k)} - q_{i+1}^{(k-1)}\right| \le \varepsilon_q$$

Where ε_V, ε_H and ε_q represent the desired precision of computation. The values obtained during the last iteration represent the values of the output q_{i+1}, volume V_{i+1} and water level in the reservoir H_{i+1}:

$$q_{i+1} = q_{i+1}^{(k)};\ V_{i+1} = V_{i+1}^{(k)};\ H_{i+1} = H_{i+1}^{(k)} \quad (18)$$

This approach allows large time steps, as opposed to the first approach based on the approximation of the water volume and input values at the time *i*.

c) Deriving a storage-outflow function ***E(H)***.

For this purpose, equation (13) is written in the following form:

$$V_{i+1} + \frac{q_{i+1}}{2}\Delta t = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right)\Delta t - \frac{q_i}{2}\Delta t \tag{19}$$

The left side of the relation (19) is denoted by $E(H)$ and can be derived based on elevation-storage $V(H)$ and elevation-outflow $q(H)$ relationships. Thus, for different values of H the corresponding values of $E(H)$ are computed (Figure 4):

$$E(H) = V(H) + \frac{q(H)}{2}\Delta t \tag{20}$$

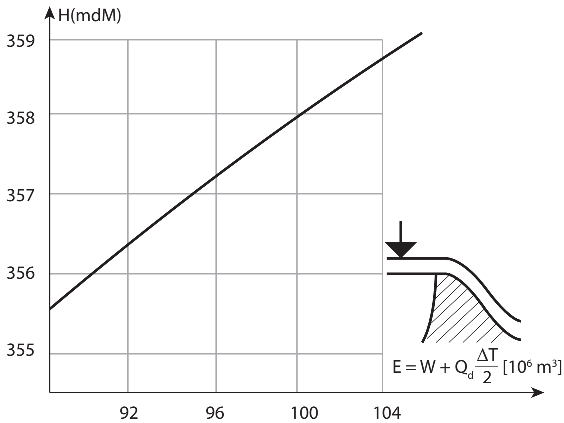


Figure 4. Elevation-outflow curve [11]

It has to be mentioned that the function elevation-storage $E(H)$ depends on the hypothesis of outlets operation rules. Consequently, corresponding to a proposed operation scenario of the outlets a specific graph $E(H)$ is obtained. The outflow q_{i+1} is derived as follows.

Based on known values of V_i and q_i (the volume in the reservoir and the outflow respectively at the beginning of the computation step) and on the inflow Q_i and Q_{i+1} at the time i and $i + 1$ the right term of the relation (10) is computed; the result represents in fact the value of $E(H_{i+1})$:

$$E(H_{i+1}) = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right)\Delta t - \frac{q_i}{2}\Delta t \tag{21}$$

Using the function $E(H)$, the value H_{i+1} representing the water level in the reservoir at the end of the computation step (time i +1) is obtained either graphically or numerically.

Based on the elevation-storage $V(H)$ and elevation-outflow $q(H)$ relationships, the values V_{i+1} and q_{i+1}, representing initial values for the next step are derived.

The results of the flood attenuation in a reservoir are the time series: $\{V_i\}$, $\{H_i\}$ and $\{q_i\}$which are graphically represented for visualisation purposes.

Considering individual operation rules for the outlets

Till now it was supposed that the output q_i is obtained considering all outlets open, which is not the case in the current practice. During the reservoir exploitation, depending on the water level in the reservoir some outlets (like the bottom gates) can be open, then closed and open again. Others will become active only when the water level exceeds the crest of the spillways. Consequently, the outlets are grouped into classes of identical operation, all outlets in a class having the same operation rules. The output q_i can be expressed as:

$$q_i = \sum_{j=1}^{n} s_j\ (H_i)\ q_j(H_i) \tag{22}$$

Where: n is the number of the classes of identical operation of the outlets. The outlets are differentiated not only by their structural but also by functional (operational) characteristics, outlets of the same structural type belonging to different classes. For example, if a dam has 4 bottom gates, 2 of them can belong to a class according to the operation rules while the other 2 can belong to another class.

H_i – is the water level at time i

$q_i(H_i)$– the outflow delivered by only one outlet from the class j at the time i

$s_j(H_i)$ – a state variable indicating how many outlets are in operation at the level Q_i

$s_i(H_i) = 0$ – if no outlet from the class Q_i is in operation

$s_j(H_i) = 1$ – if one outlet from the class Q_i is in operation, etc.

Considering the relation (22) for the output q_i, the equation (12) becomes:

$$\frac{Q_i + Q_{i+1}}{2} - \sum_{j=1}^{n} \frac{s_{i,j}\, q_{i,j} + s_{i+1,j}\, q_{i+1,j}}{2} = \frac{V_{i+1} - V_i}{\Delta t} \tag{23}$$

In this case, the volume V_{i+1} is:

$$V_{i+1} = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right) \Delta t - \sum_{j=1}^{n} \frac{s_{i,j}\, q_{i,j}}{2} \Delta t - \sum_{j=1}^{n} \frac{s_{i+1,j}\, q_{i+1,j}}{2} \Delta t \tag{24}$$

Which can be written:

$$V_{i+1} = \widetilde{V_i} - \sum_{j=1}^{n} \frac{s_{i+1,j}\, q_{i+1,j}}{2} \Delta t \tag{25}$$

Where

$$\widetilde{V_i} = V_i + \left(\frac{Q_i + Q_{i+1}}{2}\right) \Delta t - \sum_{j=1}^{n} \frac{s_{i,j}\, q_{i,j}}{2} \Delta t \tag{26}$$

In this simulation model, having the operation rules of the outlets previously defined, at each time *i* one obtains the state variables of the system: the volumes $\{V_i\}$ and the water levels $\{H_i\}$ in the reservoir, as well as the outlets $\{q_{ij}\}$ of each class and the total output discharges $\{q_i\}$.

Simulation-optimisation model

A simulation-optimisation model contains:

a) The objective function *Z* to be optimised.

The maximum value of the outflow should be limited at the discharge capacity of the river bed q_{adm}, without inundating the floodplain. At the same time, the flood wave should transit the reservoir as quickly as possible to allow the attenuation of successive floods. Consequently, the output discharges should be as close as possible to the value of q_{adm} and the objective function of the model is:

$$(\min)\ Z = \sum_{i=1}^{N}\left(\sum_{j=1}^{n} s_{i,j}\, q_{i,j} - q_{adm}\right)^2 \tag{27}$$

b) Equation (24) of the dynamic of the attenuation process, which during the optimisation allows the computation of the term $s_{i,j} q_{i,j}$.

c) Constraints concerning the state variables or the output variables, as follows:

- limitation of the water level in the reservoir to a maximum value H_{max}:

$$H_i < H_{max} \tag{28}$$

- limitation of the output fluctuations:

$$|q_i - q_{i-1}| < \varepsilon_q \tag{29}$$

where ε_q is the maximum allowed difference between two successive values of the outflow

- limitation of the hydraulic gradient of water level decrease or increase in the reservoir in order to prevent slope landslides:

$$\frac{\Delta H}{\Delta t} < G_{max} \tag{30}$$

here G_{max} is the maximum allowed gradient established during the design phase

- prescribing a target volume or level in the reservoir at the end of attenuation process:

$$|V_N - V_f| < \varepsilon_V \tag{31}$$

$$\left|H_N - H_f\right| < \varepsilon_H \tag{32}$$

where N is the number of computation steps, V_N and H_N are the volume and the water level at the end of the computation, while V_f and H_f are the target values in order to satisfy both the water users and to assure the flood protection in case of successive floods.

The decision variables are represented by the water levels between which the outlets are active. For instance, in Figure 5 a possible scenario of outlets operation is presented in the case of a dam with uncontrolled spillways. The bottom gates become active being open at the level Ht1, which can be the Normal Retention Level or a lower level. These gates are closed at the level Ht2, in order to protect the downstream area from flooding by avoiding the superposition of the outflows discharged simultaneously by the bottom gates and the spillways. Finally, the bottom gates are open again at the level Ht3, despite the large outflows downstream the dam in order to protect the dam of overflowing.

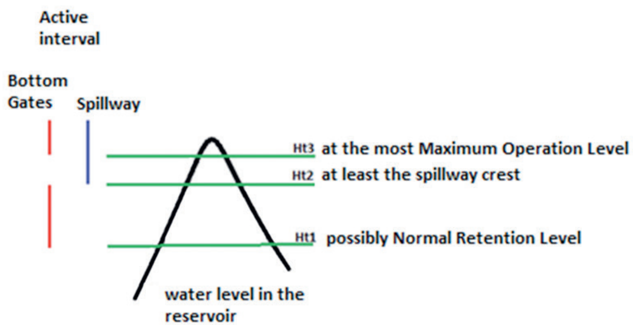


Figure 5. Outlets operation rules [8]

The decision variables are obtained by optimisation. Because the objective function (27) cannot be expressed analytically, the optimisation process is based on algorithms like Nelder-Mead method (downhill method) or on genetic algorithms which do not need the computation of the derivatives. Although fast, Nelder-Mead can stop the search in a local optimum. On the contrary, Genetic algorithms need more computational time but they find the global optimum.

Case studies of flood attenuation

Operation of Dridu Dam outlets

The Dridu dam is located in a low area of the Ialomiţa river basin, 800 m upstream the confluence with Prahova River. The Dridu retaining wall is represented by a concrete spillway dam and a front earth dam that continues with lateral dams called inadequately dykes. The height of the dam is 20 m, and the volume of the water reservoir is of 45 hm^3.

During the 24 years of operation some atypical phenomena and incidents were observed. Consequently, the Normal Pool Level was decreased from 69.20 m to 68 m above the Black Sea level. The minimum operation level to guarantee water supply to the population is 63 m, while the minimum retention level to produce hydropower is 65 m. The spillways are controlled by radial gates. The gates are partially lifted in order to regulate the rate of flow. A family of rating curves is derived depending on the opening *e* between the crest of the spillway and the lower part of the gate (Figure 6).

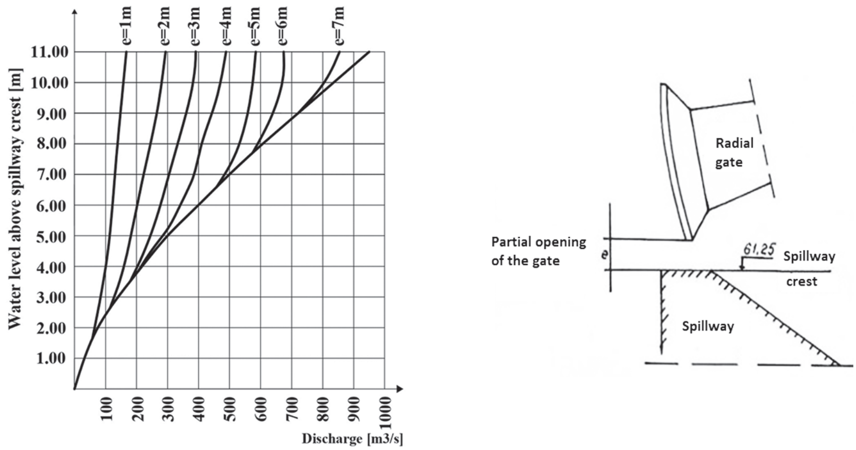
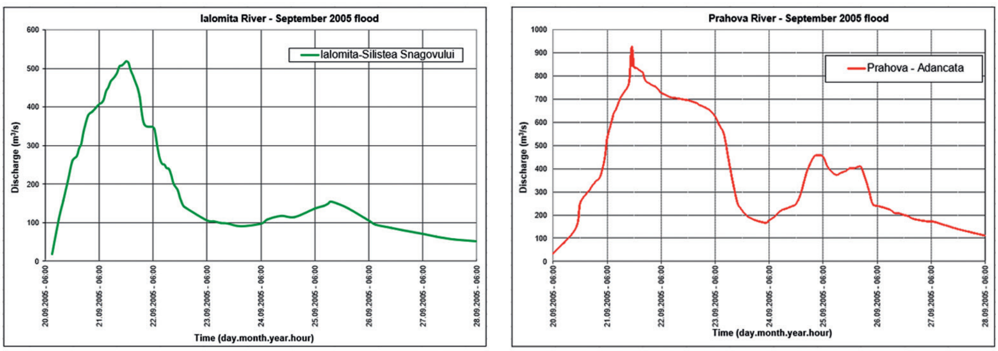


Figure 6. Rating curve for a radial gate [13]

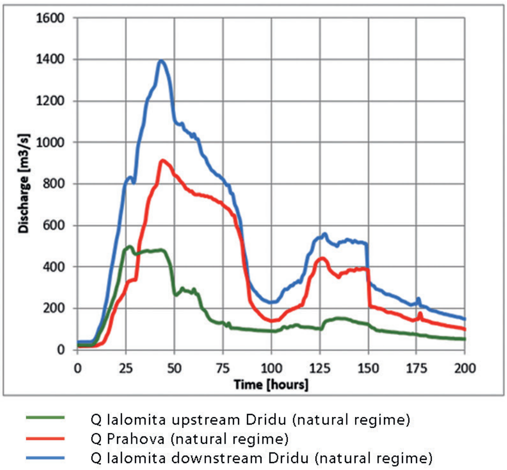
In 2005, floods occurred in the whole country. Between 20–30 of September, the flood produced on the Ialomița River reached a maximum discharge of about 500 m^3/s, while the flood on its tributary had a maximum value of about 900 m^3/s (Figure 7). As previously mentioned, the dam is located on the Ialomița River, while the flood on the Prahova River cannot be controlled.



Ialomița River – Siliștea Snagovului gauge station *Prahova River – Adâncata gauge station*

Figure 7. Floods on 20–30 September 2005 on Ialomița and Prahova Rivers [13]

In natural regime (no dam), the flood hydrograph downstream the confluence of the Prahova River with the Ialomița River, would look like in Figure 8, and the maximum discharge would have been almost 1,400 m^3/s. The operation of the dam outlets should be done in such a way to avoid the superposition of the maximum discharges. The solution is to put into operation the bottom gates at the very beginning of the flood on the Ialomița River. In this way, the additional storage volume created in the reservoir will be used to retain the flood peak on the Ialomița River, when the discharges on the Prahova River are at maximum. For this purpose, all the radial gates are gradually elevated to 1.5 m, then to 3 m. During the peak on the Prahova River the gates are let down to 1.75 m and finally are closed for 15 hours (Figure 9). In the following, only one gate is open at 1.75 m on the recession limb of the flood. This manoeuvre is made to prevent the increase of the water level in the reservoir over 68.00 m, representing the safe level in operation. At the end of the attenuation process, the water level in the reservoir is brought at 68.00 m in order to assure the necessary reserve for water users. The maximum discharge downstream the confluence is about 980 m^3/s (Figure 10).



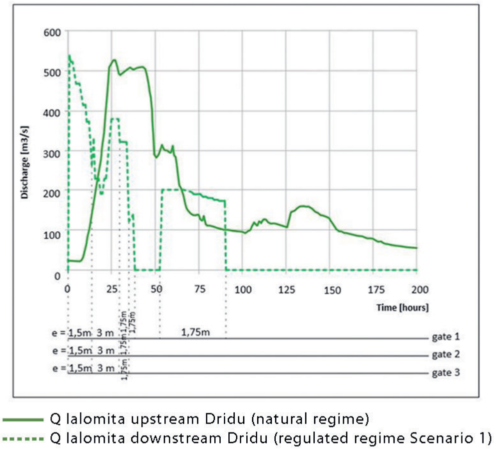


Figure 8. Superposition of the floods downstream the confluence in natural regime (no Dridu dam) [13]
Figure 9. Gates operation during the flood on the Ialomița River (Scenario 1) [13]

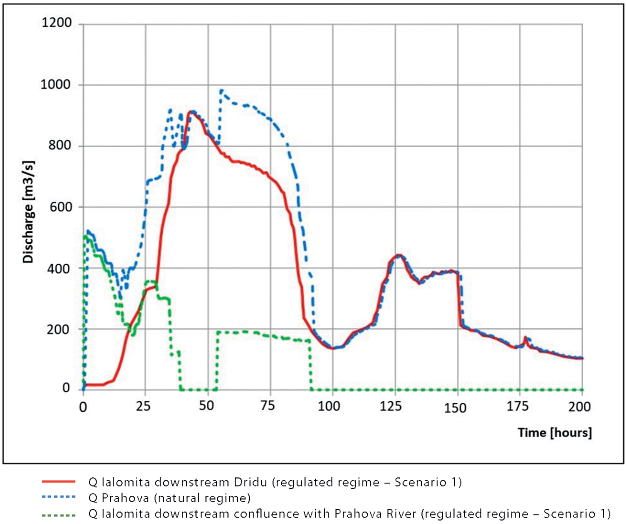
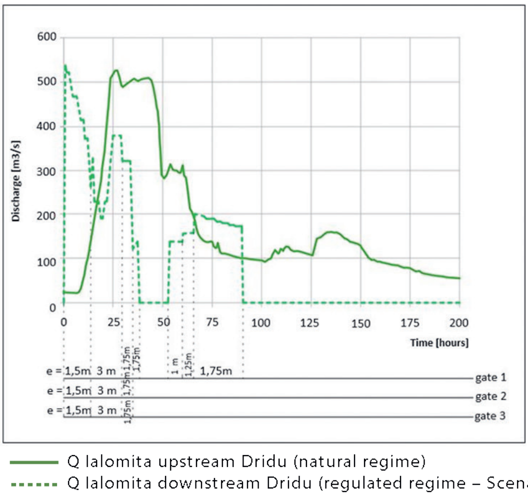


Figure 10. Superposition of the floods downstream the confluence in regulated regime (Scenario 1 of operation) [13]

A more sophisticated operation of the gates is needed to obtain a flat shape of the maximum discharges downstream the confluence (Figures 11 and 12). In this Scenario the safe water level in the reservoir is exceeded for a short period, which is however acceptable.



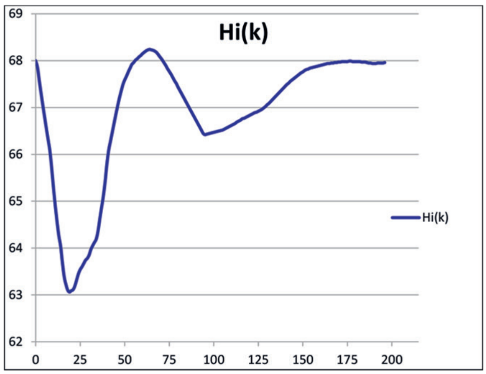


Figure 11. Gates operation during the flood on the Ialomița River (Scenario 2) [13]

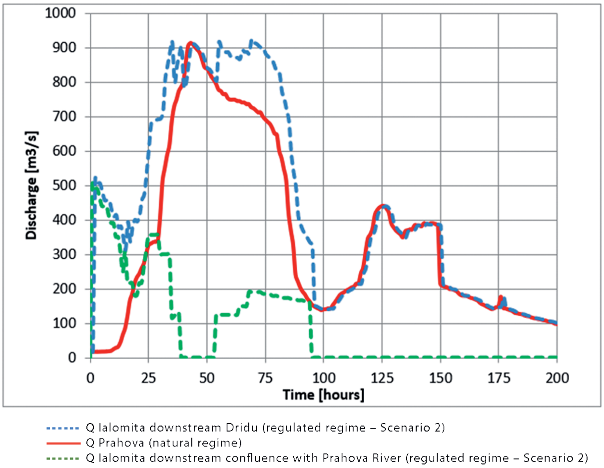


Figure 12. Superposition of the floods downstream the confluence in regulated regime (Scenario 2 of operation) [13]

The maximum discharge downstream the confluence in Scenario 2 is practically flat, being reduced from 1,400 m^3/s to 920 m^3/s (Figure 13).

The small height of the dam as well as the constraints imposed to the maximum water level by safety reasons limits an advanced attenuation of the flood downstream the confluence.

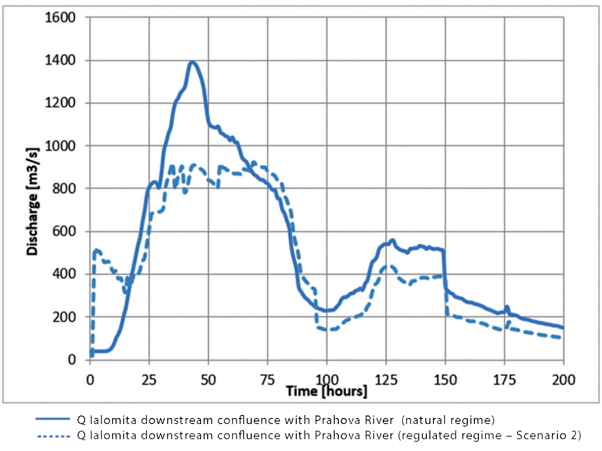


Figure 13. Superposition of the floods downstream the confluence in natural regime (Scenario 2) versus superposition in natural regime [13]

Flood management in the Jijia River basin

Joint operation of the reservoirs

The hydrotechnical development of the Jijia River (Figure 14) includes 4 permanent frontal reservoirs, from which the Ezer reservoir is situated on the Jijia River, Cătămărăşti and Drăcşani reservoirs on the Sitna River, and Hălceni reservoir on the Miletin River. Câmpeni reservoir, located on the Miletin River upstream Hălceni reservoir is a frontal non-permanent reservoir. The Hălceni polder is just upstream Hălceni reservoir. At the same time, in the lower part of the Jijia River 6 polders at Ţigănaşi were designed as the ultimate control structures for flood attenuation. The polders are symmetrically located, 3 polders being placed on the left side of the Jijia River and the other 3 polders in mirror on the right side of the river.

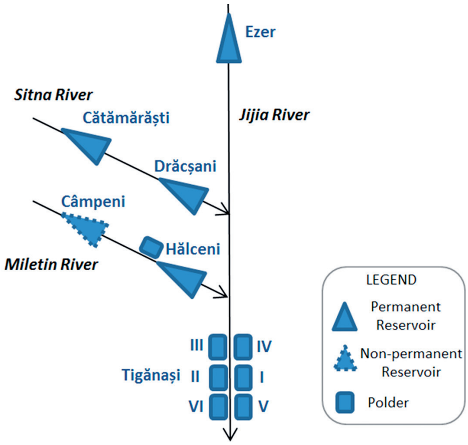


Figure 14. Jijia hydrotechnical development [12]

The maximum flood control volume of the reservoirs (4 x frontal permanent, 1 x frontal non-permanent, 1 x lateral non-permanent) is of about 77.35 mil. m^3, while the volume of the 6 polders at Ţigănaşi is approximately 80.3 mil. m^3.

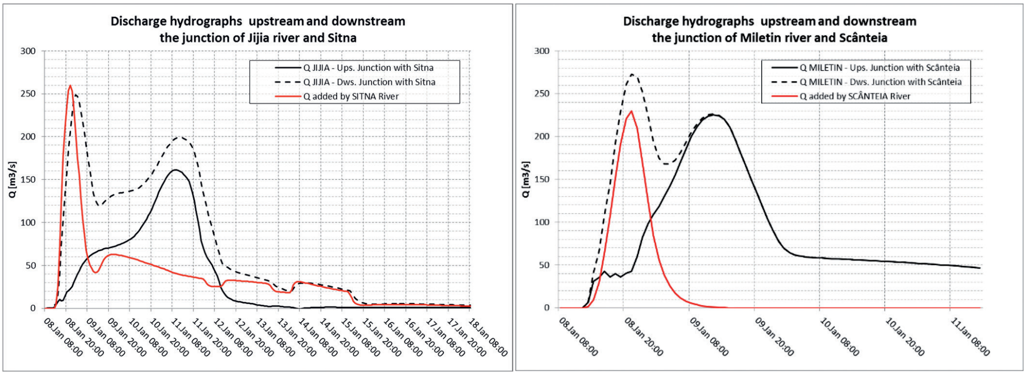
The software used for numerical simulations is Mike 11 by DHI. The specific numerical model is able to consider as input:

– either the precipitation on the entire river basin followed by a coupled hydrological-hydraulic modelling – in this case the main modules used are: Rainfall-Runoff module (RR), Hydrodynamic module (HD) and Structural Operation module (SO)
– or directly the input hydrographs due to small tributaries of the Jijia River and its main tributaries: Sitna River and Miletin River as upper boundary conditions – in this case only the following main modules are used: Hydrodynamic module (HD) and Structural Operation module (SO)

The main steps in mathematical modelling are:

- schematisation of the river network, by creating the topologic model of the network
- integration of the Digital Terrain Model (DTM), of both the floodplain area and the river bed
- database creation, which should include detailed information related to the river basin, hydraulic network, hydraulic structures and their operation, meteorological and hydrological data
- setting up the hydraulic model and calibration of the hydraulic parameters of the river bed using steady state simulations
- choosing the most significant floods
- analysing the meteorological data which generated significant floods
- calibration of the hydrological parameters, based on physiographical characteristics of the river basin and previous values of hydraulic parameters for the river bed – the hydraulic parameters for the floodplain were proposed according to Landcover information concerning the land use
- validation of the hydrological and hydraulic parameters using other registered floods
- statistical processing of the maximum annual precipitation for different duration obtained from the meteorological and/or hydrometric stations
- evaluation of the synthetic floods components, by keeping the same probability of exceedance along the river stretch between two successive hydrometric stations
- flood propagation in the modified hydraulic regime due to existing hydraulic structures
- assessment of the efficiency of the existing operation rules of the reservoirs using synthetic floods corresponding to the following probabilities of exceedance of the maximum discharge: 10%, 1% and 0.1%
- improvement of the coordinated operating rules by a trial and error procedure

The concentration time of the tributaries is smaller than the concentration time of the main river basin, leading to a different moment of the peak discharges. As a result, the flood downstream the confluence has two peaks and the maximum discharge is smaller than in the case of superposition of the maximum discharges (Figure 15). In other cases, due to the spatial variability of the precipitation it is not possible to avoid the superposition of the flood downstream the reservoirs with the floods due to tributaries.



a) Jijia River and Sitna tributary *b) Miletin River and Scânteia tributary*

Figure 15. Discharge hydrographs upstream and downstream the confluence of the main river with a tributary [8]

The simulations showed that the effect of the reservoirs is important, due to large flood protection volumes. Still, the bottom gates are able to evacuate relative low discharges and the reservoirs spillways have no gates. Under these conditions the possibility to improve the operation rules of the reservoirs is quite limited, the only chance to increase the role of the reservoirs being the release of the water through the bottom gates immediately after the early warning of an imminent flood. In Figure 16, some examples of the flood waves upstream the Cătămărăşti reservoir and the flood hydrographs downstream reservoirs for different timing of bottom gates opening are presented. The maximum discharge is decreased from 176 m^3/s to 25–80 m^3/s depending on when the bottom gates are open.

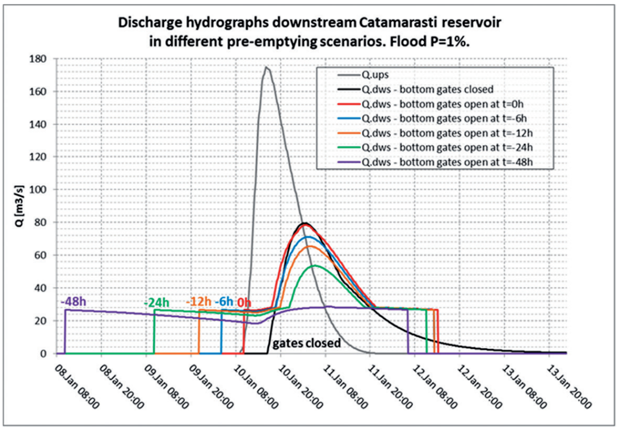


Figure 16. Effect of bottom gates opening based on early warning of a flood [8]

In order to understand how important it is to jointly operate both reservoirs on the Sitna River, other scenarios were imagined, taking into account all the possible combinations of operating each reservoir. The following scenarios of operating the bottom gates of both reservoirs, Cătămărăşti and Drăcşani, were examined: 1. Keeping permanently the gates closed; 2. Opening the gates at the time of arrival of the flood into the reservoir; 3. Opening the gates 24 hours before the arrival of the flood; and 4. Opening the gates 48 hours before the arrival of the flood.

Some tributaries (Morişca, Dresleuca, Burla) of the Sitna River bring important contributions during the flood period. However, the attenuation on the Sitna River due to the significant retention capacity of the Cătămărăşti and Drăcşani reservoirs is important (Table 1), with a reduction for the flood 1% at the confluence with Jijia River from 380 m^3/s (natural regime) to 152–182 m^3/s (regulated regime). Still, the potential to optimise the operation rules in order to minimise those values is quite limited due to the absence of flap gates at spillways.

The operation rules of the Cătămărăşti and Drăcşani reservoirs are presented in the lower part of Figure 17, where the yellow strip means the gates are open, while the grey strip corresponds to closed gates. On the same figure, the input and output hydrographs from the reservoirs are shown, as well as the water level evolution in the reservoirs.

Table 1. Maximum discharges along the Sitna River – Flood 1% (compiled by the author)

No.	Cross-section	Q Max [m^3/s]
1	Q upstream Catamarasti reservoir	181
2	Q downstream Catamarasi reservoir	42
3	Q gauge station Catamarasti	54
4	Q upstream Morisca tributary	58
5	Q downstream Morisca tributary	123
6	Q upstream Dresleuca tributary	125
7	Q downstream Dresleuca tributary	181
8	Q upstream Dracsani reservoir	180
9	Q downstream Dracsani reservoir	100
10	Q upstream Burla tributary	100
11	Q downstream Burla tributary	150
12	Q gauge station Dracsani	150
13	Q gauge stationTodireni	182

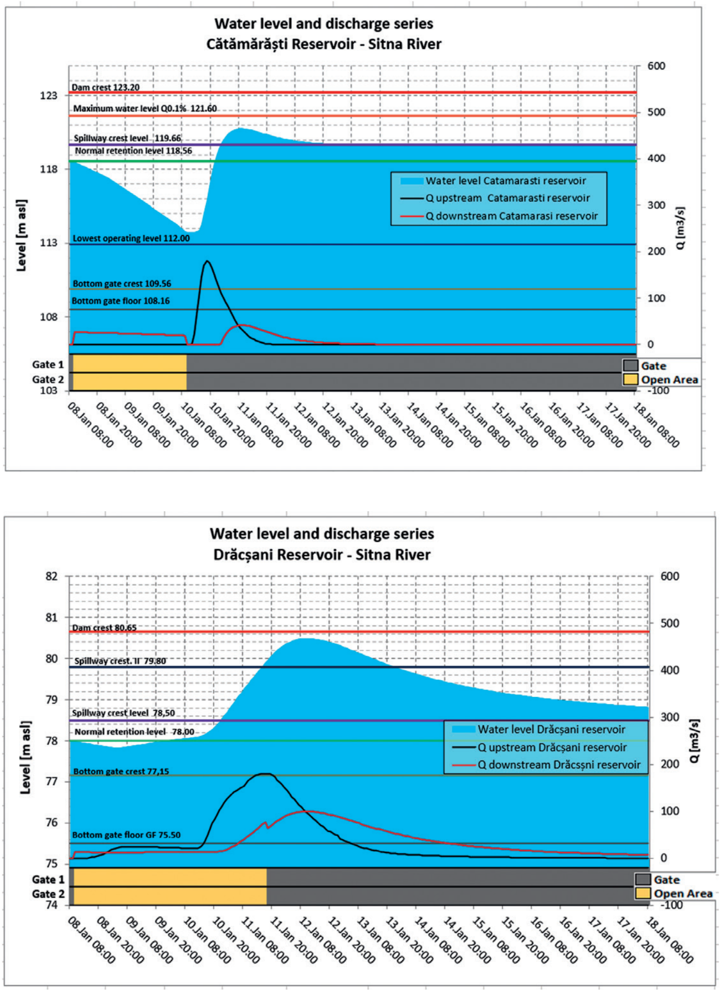


Figure 17. Flood attenuation in the Cătămărăşti and Drăcşani reservoirs [8]

As it can be seen in Figure 18, the best joint operation rule corresponds to the situation when the bottom gates are kept closed at Cătămărăşti, while decreasing the water level in the Drăcşani reservoir starts 48 hours before the flood arrival by opening the bottom gates. A common pattern identified in the simulations is: the best efficiency in terms of maximum discharge at the Sitna River upstream the confluence with the Jijia River (values between 152–155 m^3/s) is obtained when the Drăcşani reservoir water level starts to be lowered 48 hours earlier of the flood occurrence in the reservoir, whatever rules are chosen for Cătămărăşti.

The worst solution is to open the bottom gates at the Cătămărăşti reservoir 48 hours before the flood arrival, opening the Drăcşani reservoir only at the arrival time of the flood and keeping the bottom gates open during the entire flood event. The maximum discharge in this case is 182 m^3/s at the confluence of the Sitna River with the Jijia River.

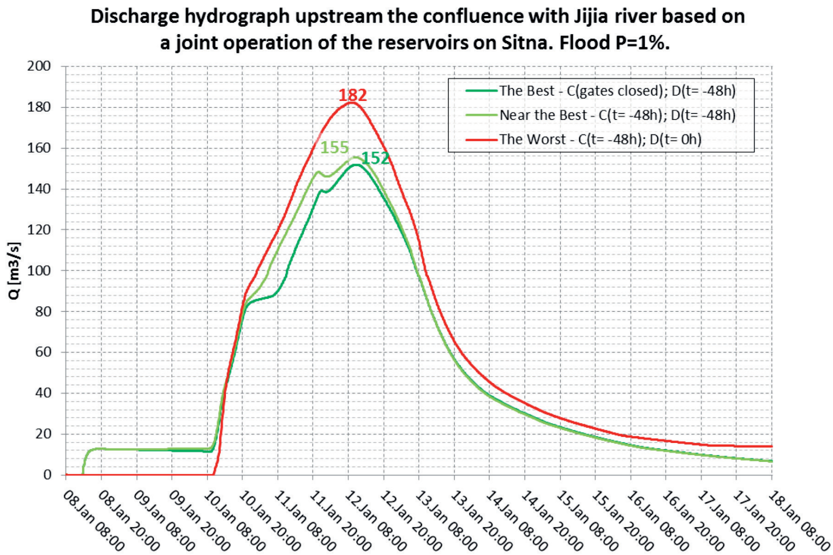


Figure 18. Discharge hydrograph upstream the confluence with the Jijia River for different operation scenario of the bottom gates [8]

A similar approach was used for the operation of the reservoirs on the Miletin River. The evolution of the maximum discharges along the Miletin River, considering the attenuation in the reservoirs as well as the influence of the floods on the tributaries is presented in Table 2. On this important tributary of the Jijia, there are two reservoirs: Câmpeni and Hălceni. The last one is, in fact, a system of two reservoirs, compound by a permanent reservoir and a polder, situated immediately upstream the reservoir, on the left bank of the river. Câmpeni reservoir is non-permanent, and therefore there is not possible to operate the bottom gates. In this case, a joint operation on the Miletin River is not possible. Hălceni is situated only at 6 km upstream the confluence with the Jijia River, thus, the discharge immediately downstream this reservoir can be considered the contribution of the Miletin River to the Jijia River.

Table 2. Maximum discharges along the Miletin River – Flood 1% (compiled by the author)

No.	Cross-section	Q Max [m^3/s]
1	Q upstream Campeni reservoir	307.7
2	Q downstream Campeni resevoir	253.3
3	Q upstream Scanteia tributary	246.9
4	Q downsteam Scanteia tributary	269.9
5	Q upstream Recea tributary	308.6
6	Q downstream Recea tributary	327.4
7	Q upstream Halceni reservoir	179.6
8	Q downstream Halceni reservoir	119.8
9	Q upstream Confluence, Ui@. River	119.5

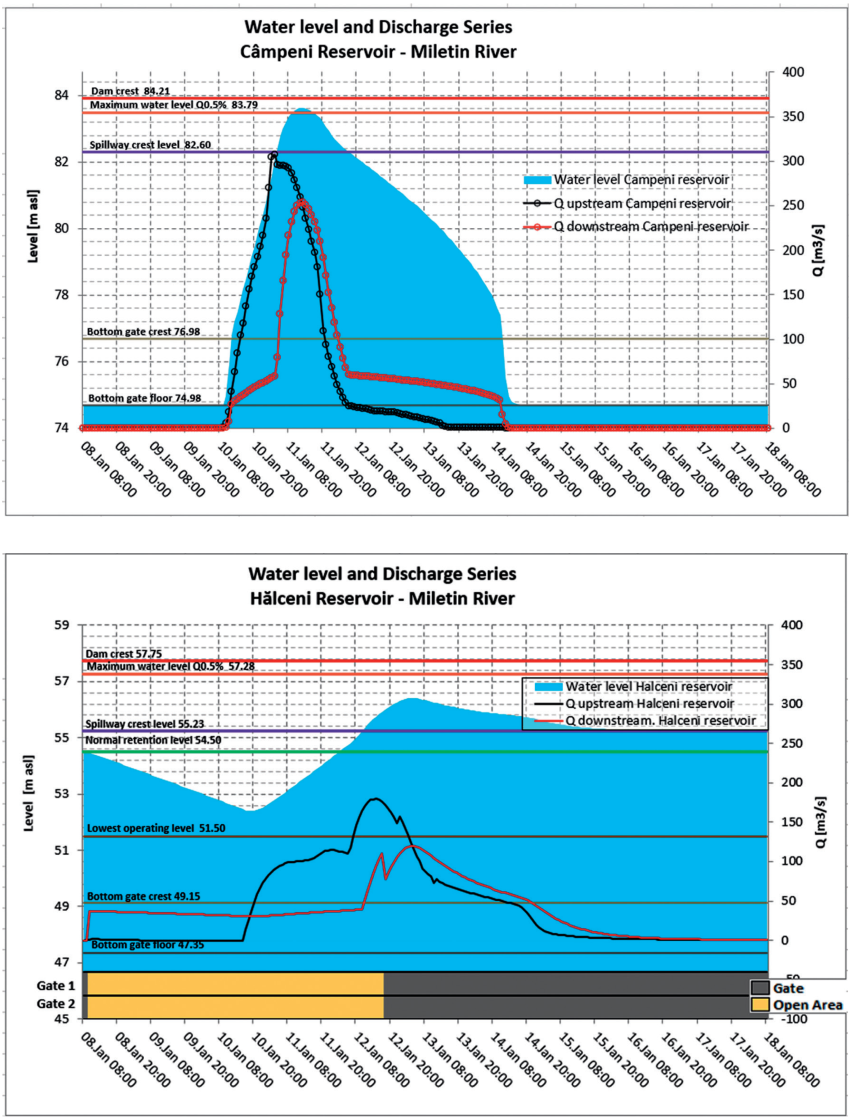


Figure 19. Flood attenuation in the Câmpeni and Hălceni reservoirs [8]

The operation rules of the Hălceni reservoir are presented in the lower part of Figure 19, with the same meaning: the yellow strip means the gates are open, while the grey strip corresponds to closed gates. On the same figure, the input and output hydrographs from the reservoirs are shown, as well as the water level evolution in the reservoirs. Câmpeni reservoir is a non-permanent storage reservoir and the bottom gates are always open.

Hălceni polder and Hălceni reservoir (see Figure 14) have an important role in flood attenuation: the polder diminishes the maximum discharge from 194 m^3/s to 108 m^3/s, and depending on how efficient the bottom gates of the frontal dam are operated, the downstream maximum discharge can vary from 54 m^3/s to 95 m^3/s (Figure 20). The difference between the best operation rule of the bottom gates and the worst one is only 40 m^3/s in this case.

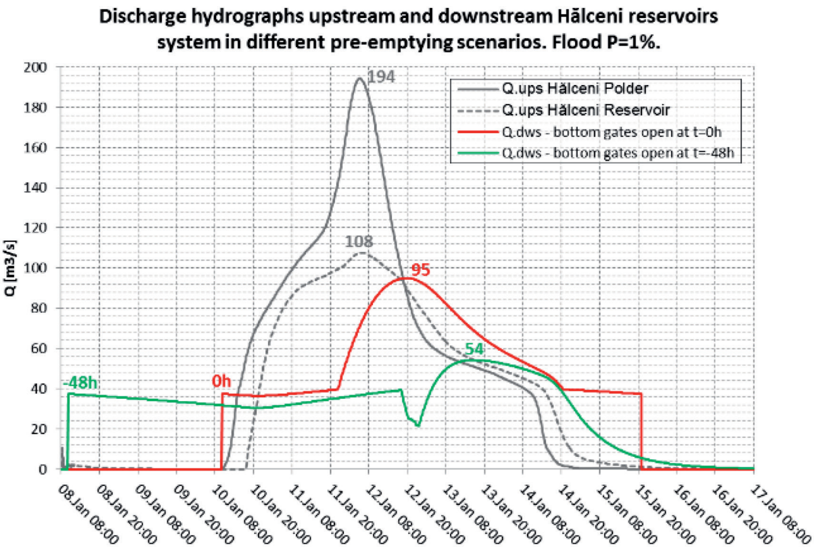


Figure 20. Discharge hydrograph upstream and downstream of the Hălceni reservoirs system [8]

The estimated maximum discharge in natural regime at the confluence of the Miletin River with the Jijia River for the flood 1% is 360 m^3/s. The attenuation due to the Câmpeni and Hălceni reservoirs is significant, the maximum discharge being reduced from 360 m^3/s to 54–95 m^3/s, depending on the bottom gates operation of the Hălceni reservoir.

Modelling polders effect

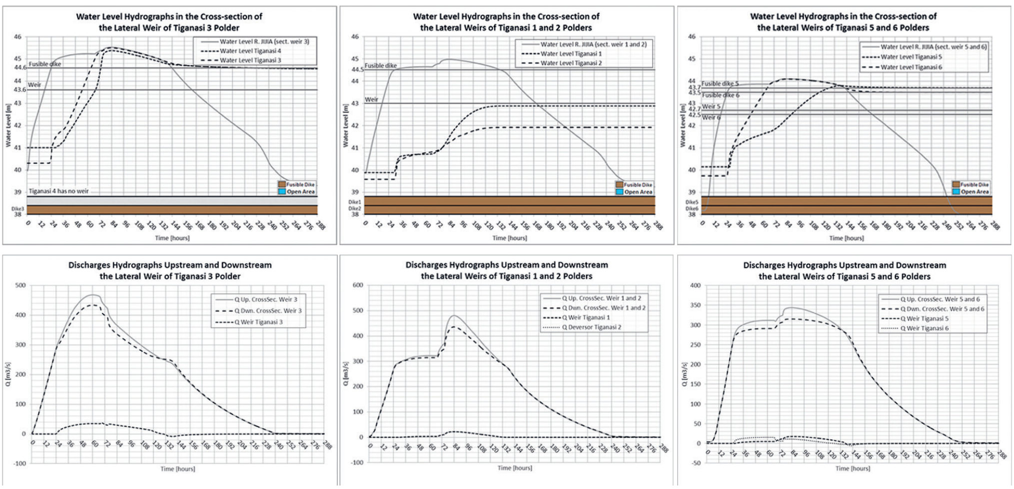
According to the common agreements signed between water authorities in Romania and the Republic of Moldova, the maximum discharge on the Jijia River upstream the confluence with the Prut River should be limited to a maximum of 220 m^3/s. Consequently the maximum discharge on the Jijia River upstream the confluence with the Bahlui River is limited to a maximum of 220 m^3/s. It has to be mentioned that this discharge is a little bit higher than the maximum discharge corresponding to 5% probability of exceedance at the Victoria gauge station, which is the last station upstream the confluence with the Bahlui river.

The catchment area at the Victoria gauge station is 3,643 km^2, the peak time of the floods in natural regime is 60 hours, the total flood duration is 240 hours, while the shape coefficient is 0.45. The maximum discharge corresponding to 1% probability of exceedance in natural regime is 350 m^3/s, and for 0.1% probability of exceedance is 575 m^3/s. According to the above-mentioned data, the threshold discharge of 220 m^3/s can only be achieved by the attenuation of the flood waves in reservoirs or polders. In the lower part of the Jijia River 6 polders at Țigănași were designed as the ultimate control structures to maintain the maximum discharge under the threshold value of 220 m^3/s. The polders are symmetrically located, 3 polders being placed on the left side of the Jijia River and the other 3 polders in mirror on the right side of the river.

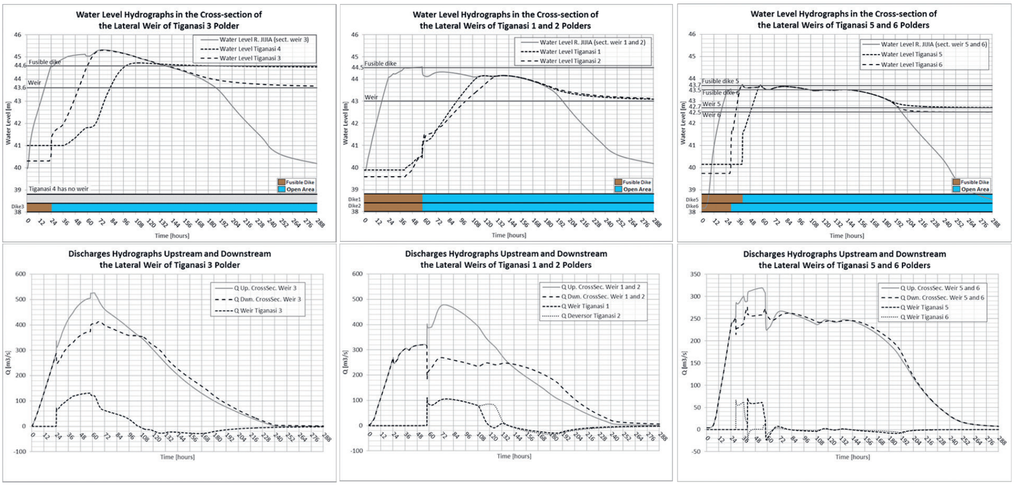
The maximum flood control volume of the reservoirs (4 x frontal permanent, 1 x frontal non-permanent, 1 x lateral non-permanent) is of about 77.35 mil. m^3, while the volume of the 6 polders is approximately 80.3 mil. m^3. The software used for simulations is Mike 11 by DHI.

The effect of the operation rules of the reservoirs is important for flood control mainly on the tributaries (Sitna and Miletin). After the confluence with the Jijia River, their effect is counterbalanced by the floods produced on the Jijia River or its tributaries. The floods from upper Jijia could be important due to the limited effect of the Ezer reservoir (which is very upstream) and the significant input of the tributaries and inter-basins downstream this reservoir. It means that the permanent reservoirs on the Sitna and Miletin rivers cannot mitigate the floods at the level of the whole basin in such a way to maintain the maximum discharge at Victoria station under the threshold value.

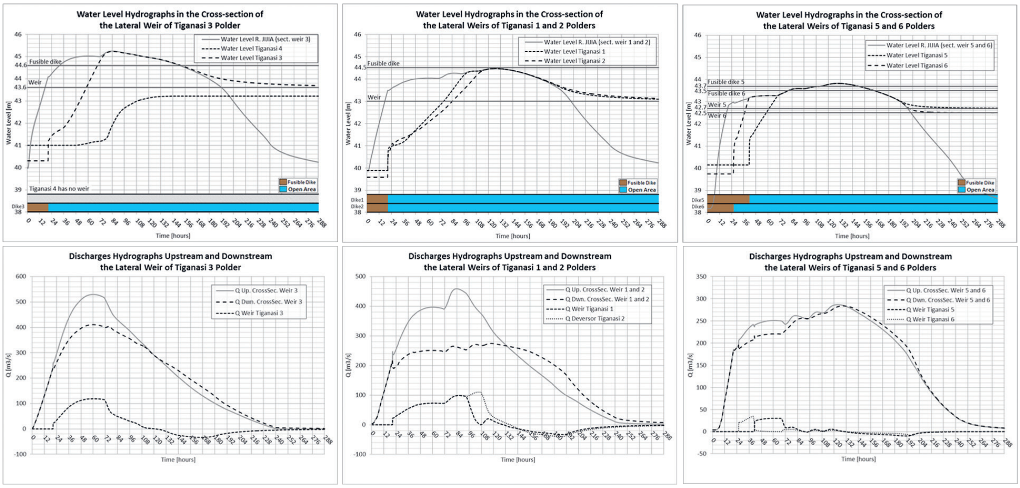
Under these conditions, the Țigănași polders are the most downstream hydrotechnical works able to modify the flood waves coming from upstream and to observe the threshold value imposed by the international convention between Romania and the Republic of Moldova.



a)



b)



c)

Figure 21. Water levels and discharges according to the analysed scenarios [8]

a) Scenario no. 1; b) Scenario no. 2; c) Scenario no. 3

The simulations reproduced the possible behaviour of the polders, which are protected by fuse dikes. The fuse dike is an erodible layer of 1 m for the polders 3, 5 and 6, and 1.5 m for the polders 1 and 2 respectively. The following scenarios were analysed:

Scenario no. 1 – The fuse dikes do not breach, even overtopped.

Scenario no. 2 – The fuse dikes breach after being overtopped by a water depth of min. 5 cm.

Scenario no. 3 – The fuse dikes breach by internal erosion when the water level in the river is 50 cm higher than the spillway crest.

The obtained results are presented only for the flood corresponding to the maximum discharge 0.1% (Figure 21). This flood is characterised by a maximum discharge of 575 m^3/s and a volume of 223 mil. m^3 upstream the polders in natural regime.

By representing on the same graph the discharge hydrographs upstream and downstream Țigănași polders one can notice that the most significant attenuation is produced in Scenario no. 2 when the fuse dike breaches by external erosion, after overtopping. In this case, the maximum discharge (271 m^3/s) is reached on the increasing limb of the flood wave (Figure 22). In Scenarios no. 1 and 3, the maximum discharge is reached on the recession limb (284 m^3/s in Scenario no. 3, and 325 m^3/s in Scenario no. 1).

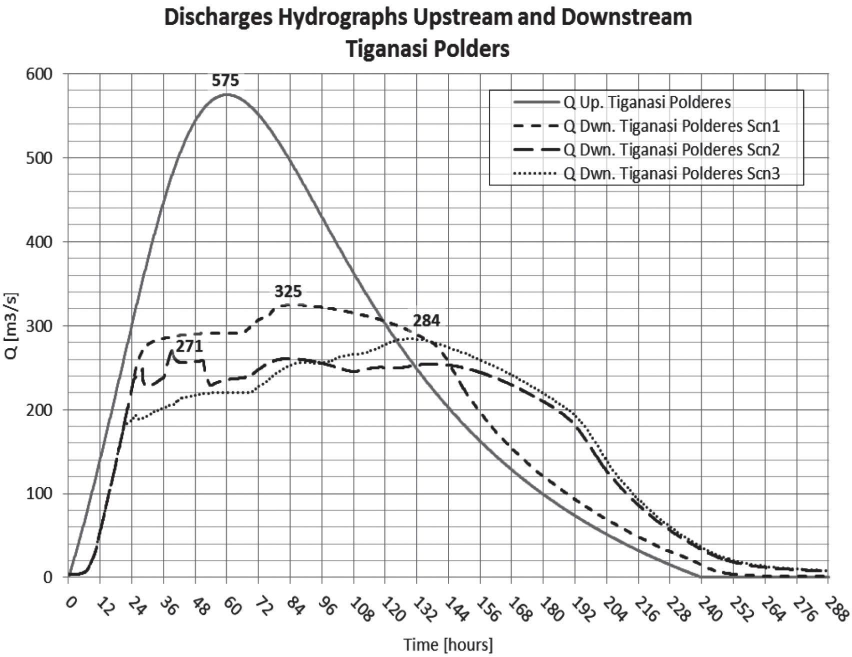


Figure 22. Discharge hydrographs upstream and downstream polders [8]

These simulations put into evidence the major role played by the polders. By the storage of about 51 mil. m^3, the maximum discharge upstream the polders is reduced from 575 m^3/s to 271–325 m^3/s downstream the polders, depending on the breach scenario.

Polder no. 4 (Țigănași 4) has no fusible dike because its flooding should be avoided as much as possible. Still, in the case of the flood 0.1% even this polder is flooded no matter the breach scenario.

Since the increasing time of the flood is about 60 hours, representing the minimum anticipation time, the water management authority has enough time to examine the behaviour of the whole hydraulic system, to run mathematical simulations and to adapt, if it is the case, the framework operation rules to the real time evolution of the flood.

DDS for operation of the lateral reservoirs during flood periods

At the end of the 19th and beginning of the 20th century, land reclamation works started in the Danube floodplain and especially in the Danube Delta. The first embankments were realised at Mahmudia (467 ha) in 1895 and Chirnogi (1,058 ha) in 1904. The latest reclamation works date from 1985, the total protected area reaching almost 395,000 ha. On the lower Danube (downstream Iron Gates) the river banks on the Romanian side are protected by dykes on a length of 1,100 km. The floodplain is divided by transverse dykes into agricultural zones which could be used as polders for storing water during high floods. A number of 34 enclosures were realised all along the Danube (Figure 23).

In natural regime, the floodplain had a retention capacity evaluated at 20.3 billion m^3. After the embankment's realisation, an increase of more than 1 m of the water level compared to the natural regime occurs in the towns of Brăila and Galaţi during floods close to 1% probability of exceedance. Besides Brăila and Galaţi, there are other towns along the Danube River, like Giurgiu, Olteniţa and Călăraşi which are threatened by floods close to or higher than 1% probability of exceedance.

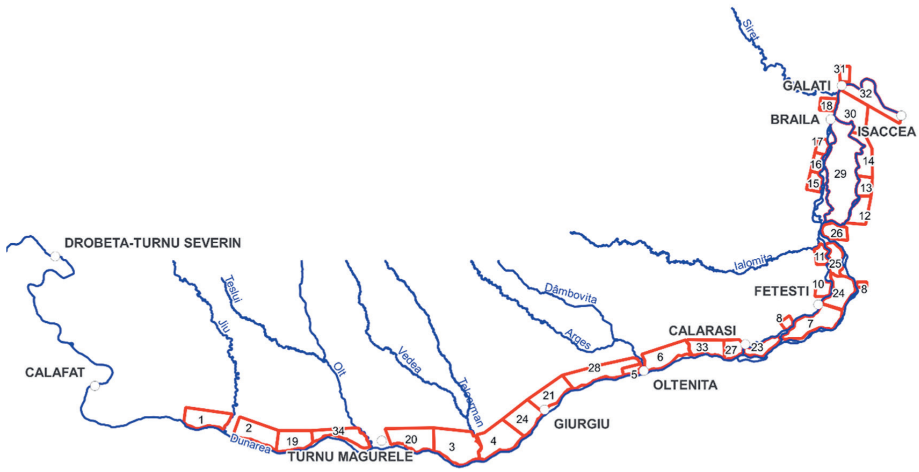


Figure 23. Enclosures along the Danube River on the Romanian territory [9]

Galaţi area has a special situation, being located between the confluences of two important tributaries: Siret River (upstream Galaţi town) and Prut River (downstream). The maximum discharges on the Prut River are controlled by the Stânca-Costeşti reservoir, being lower than 600–700 m^3/s in the section Oancea. On the contrary, the maximum discharges on the Siret River can reach 4,000–4,200 m^3/s in the Lungoci section, 65 km upstream the confluence with the Danube.

The flooding in Galaţi town occurs due to the upstream floods as happened in 2006 (15,800 m^3/s on the Danube River at Brăila gauging station upstream Galaţi town, 1,375 m^3/s on the Siret River and 627 m^3/s on the Prut River) or can be aggravated because of the high floods on both tributaries as happened in 2010 (15,480 m^3/s at Brăila, 2,460 m^3/s on the Siret River and 700 m^3/s on the Prut River). In 2006, the maximum discharge at Grindu on the Danube River, just downstream Galaţi was 16,200 m^3/s, which is quite close to the discharge corresponding to 100 years return period, while in 2010, the maximum discharge at Grindu was 16,780 m^3/s.

The flood occurred in April–May 2006 had at Baziaş, at the entrance of the Danube in Romania, a maximum discharge of about 15,800 m^3/s, being the highest registered discharge in the interval 1840–2006. The maximum water levels of this flood were up to 60 cm higher than the highest maximum levels of the floods occurred after the Danube embankment. At the same time, the design levels were exceeded up to 127 cm (Figure 24), for more than 20–35 days.

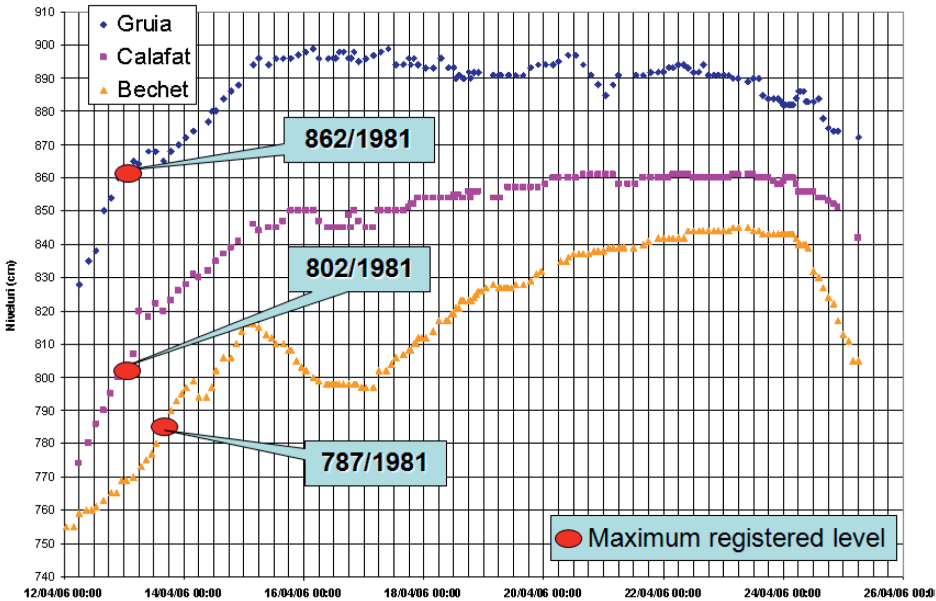


Figure 24. Water levels during the April 2006 flood and the maximum registered levels in the past [3]

The historic flood on April–May 2006 on the Danube River was at the origin of the accidental or voluntary breaches in the dykes. A surface of 73,144 ha mainly used for agricultural purposes, in 10 enclosures protected by dykes, was flooded. Some settlements were also flooded, 16,530 inhabitants being evacuated. Important towns on the river bank were also threatened by floods. Despite the damages registered in the upstream part of the river, the enclosures flooding had also beneficial effects on the downstream water levels.

Depending on the breaches' location and the failure time, the maximum level drawdown was about 28 cm. Thus, a possibility to lower the water levels during floods and to save from inundation important assets is to flood deliberately less important areas, with lower damages. In order to cut the peak and to mitigate the flood effects during the April 2006 event, 3 controlled breaches were set up at Rast, Călăraşi-Răul and Făcăieni-Vlădeni.

Table 3. Water volumes accumulated in the flooded enclosures during the April–May 2006 flood [3]

No.	Breaches	Area (ha)	Volume (mil. m^3)
	Uncontrolled breaches		
1	Ghidici–Rast–Bistreț	11,120	350
2	Bistreț–Nedeia–Jiu	15,000	285
3	Jiu–Bechet–Dăbuleni	6,000	120
4	Potelu–Corabia	11,500	230
5	Oltenița–Surlari–Dorobanțu	8,000	213
6	Oltina	2,890	94
7	Ostrov–Pecineaga	1,491	10
8	Rasova	1,500	66
	Controlled breaches		
1	Călărași	10,748	195
2	Făcăieni–Vlădeni	4,895	53
	Total	73,144	1,616

The hydraulic simulations can be used to investigate the possibility of decreasing the water levels in the downstream sections by inundation of the upstream enclosures from the Danube floodplain. If the decision is taken to flood the upstream enclosures, other downstream towns will also benefit from this effect. In fact, the enclosures' inundation operates at least partially like the natural attenuation in the floodplain. The decrease of water level depends on the volume stored in the enclosures. Of course, the inundation should be controlled in such a way to diminish the total damages on the whole stretch of the Danube River on the Romanian sector. The flood risk management thus involves a good knowledge of the economic damages for different scenarios of flooding.

A number of 13 enclosures with individual volumes in the range 40–780 million m^3 were selected if necessary for deliberate flooding by the Danube Delta National Research Institute (Figure 25). Nevertheless, the total volume which could be stored in these areas is less than 4.5 billion m^3.

Figure 25. Proposed enclosures (in blue) for flooding [9]

Proposed enclosures for flooding [9]

No.	Enclosure name	Dike crest level (m asl)	Enclosure volume (mil. m^3)
1	Seaca_Vanatori_Suhaia_Zimnicea	24.00	496
2	Bujoru_Pietrosani	19.5	41
3	Vedea_Slobozia	20.00	170
4	Gostinu_Greaca_Arges	16	723
5	Oltenita_Surlari_Manastirea	15	213
6	Boianu_Sticleanu_Calarasi	14	777
7	Borcea de Sus 1	11	218
8	Borcea_de_Jos_III	8	228
9	Borcea_de_Jos_I_II	6	45
10	Macin_Zaclau	5	346
11	Zaclau_Isaccea	5	702
12	Ciobanu_Garliciu	7	79
13	Peceneaga_Turcoaia	7	122

The effect of flooding the enclosures should not be, however, over evaluated. If considering the total volume of about 4.5 billion m^3 which could be stored into the above-mentioned enclosures, the total drawdown in the most favourable conditions would be less than 62 cm at Brăila for the flood corresponding to 1% probability of exceedance (Figure 26).

The problem which should be solved is what enclosures have to be flooded in order to obtain the necessary water level decrease to avoid significant damages at the most important towns along the Danube River. At the same time, it is obvious that flooding some enclosures has economic and social consequences on the affected areas.

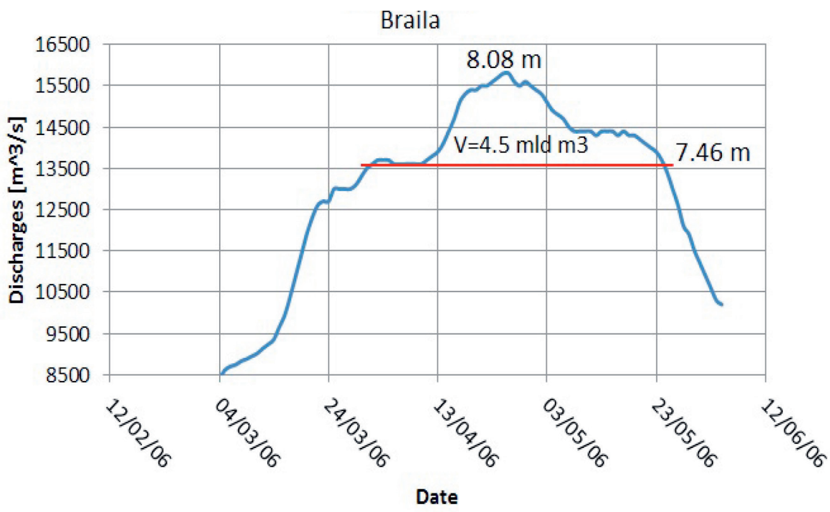


Figure 26. Maximum effect at Brăila after flooding the selected enclosures [6]

Choosing the enclosures that should be flooded involves decisions which cannot be taken under pressure during the event. A Decision Support Tool (DST) was developed by UTCB in order to evaluate the hydraulic consequences (drawdown of the water level and of the maximum discharge) for different scenarios of polders accidental or deliberate flooding. At the same time, a special sub-model for breach development (evolution in time of breach elevation and length) was set up.

Different scenarios of polders inundation were proposed and the corresponding hydraulic consequences were evaluated.

The background of the DSS tool is presented in Figure 27. On the left side of the picture one can notice the following elements:

– the Danube River and its main tributaries (marked with a blue line)
– the gauge stations on the Danube River and the most downstream gauge station of the tributaries upstream the junction with the Danube River (marked with red triangles)
– the location of all enclosures which presents favourable conditions for flooding (marked with a yellow circle with a blue arrow suggesting the water entering the enclosures)

The Danube stretch between Olteniţa and Călăraşi gauge station was analysed in the light of flood management. Two important enclosures are along this river stretch: Olteniţa–Surlari–Mănăstirea (213 mil. m^3) and Boianu–Sticleanu–Călăraşi (777 mil. m^3).

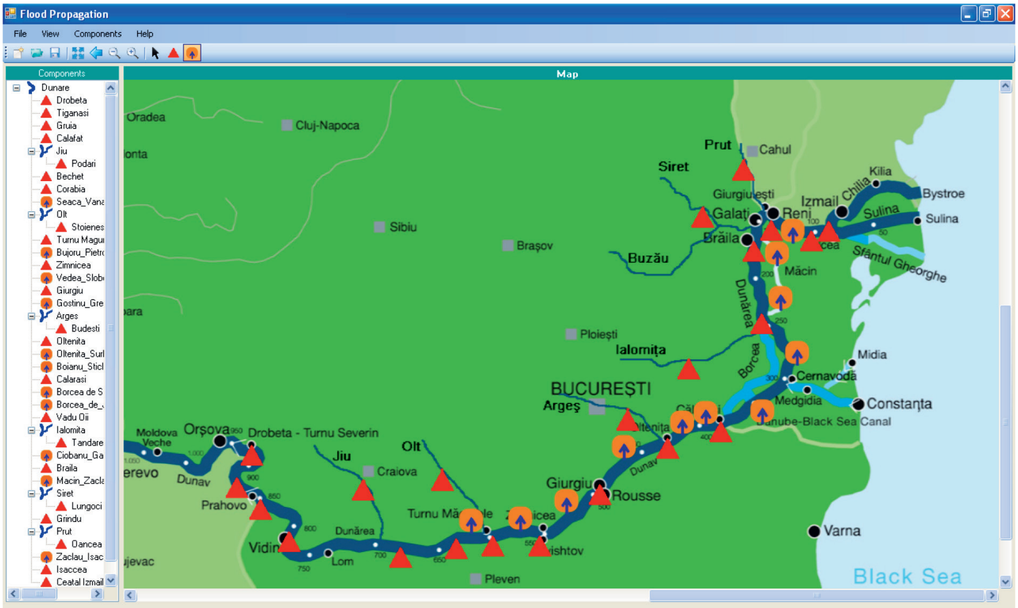
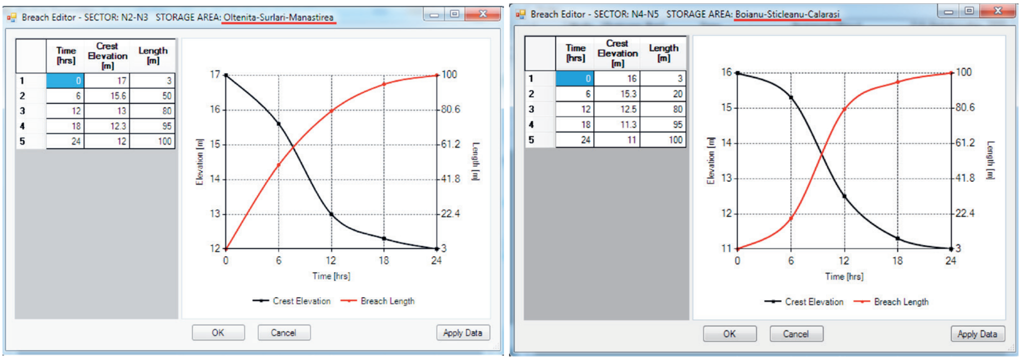


Figure 27. Background of the DSS tool [6]

The breach development involves the quantitative description of the breach crest elevation and breach length evolution. Thus, the crest elevation decreases from 17 m asl to 12 m asl in the case of Oltenița–Surlari–Mănăstirea enclosure, and from 16 to 11 m asl for Boianu–Sticleanu–Călărași enclosure (Figure 28). In both cases, the maximum breach length was supposed to be 100 m.

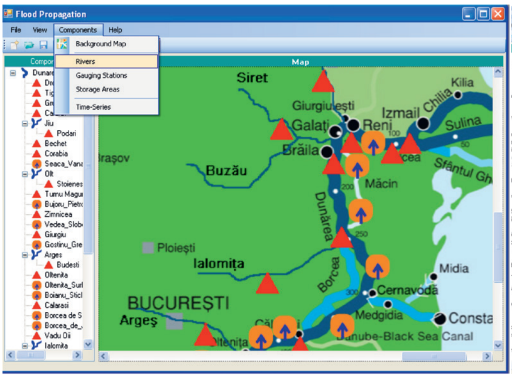


Oltenița–Surlari–Mănăstirea enclosure   Boianu–Sticleanu–Călărași enclosure

Figure 28. The breach development [6]

The necessary data for the DSS tool is the following:

– the river configuration: gauging stations with their attributes (Figure 29)
– the rating curves for each gauge station (Figure 30)
– the enclosures and their elevation-storage curves (Figure 31)
– discharge series along the Danube River (Figure 32)



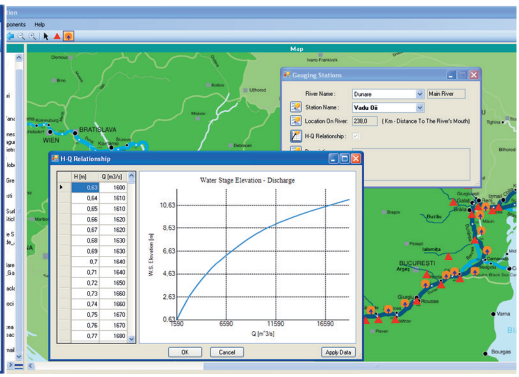


Figure 29. The river network [6]   *Figure 30. The rating curves [6]*

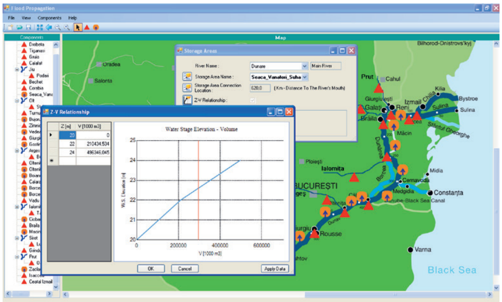


Figure 31. Enclosures [6]

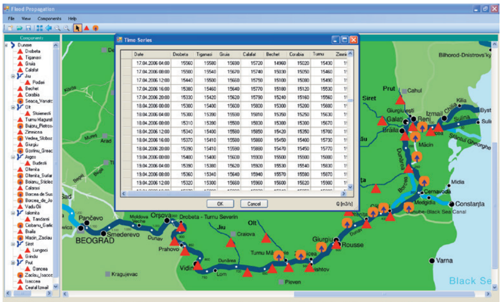


Figure 32. Discharge series [6]

Different scenarios of enclosures flooding were tested. For the beginning (Simulation no. 1) only the enclosure Oltenița–Surlari–Mănăstirea was flooded. The flooding effect is not significant (Figure 33): although a maximum discharge of about 1,000 m^3/s is entering the enclosure, due to its small retention capacity (213 mil. m^3), a temporary decrease of the Danube discharges can be noticed, but very soon the flood reaches the initial discharges. Anyway, the maximum discharges were not at all affected by the enclosure flooding.

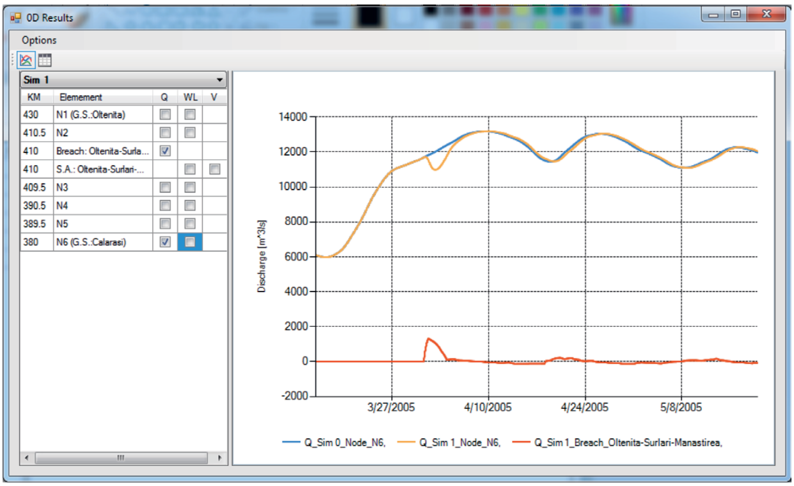
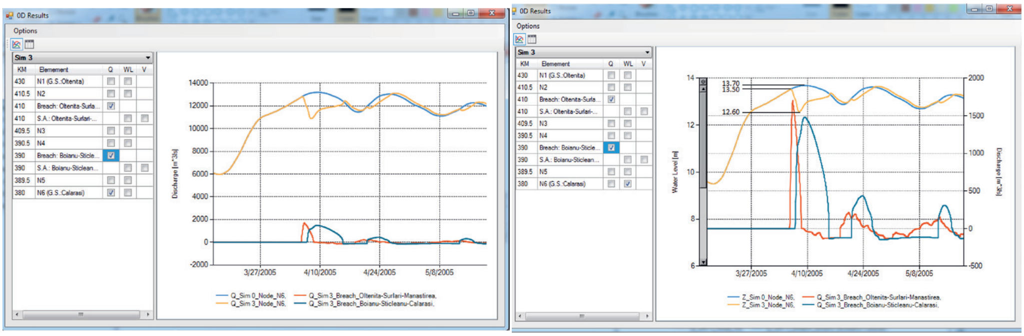


Figure 33. Hydraulic consequences of the Oltenița–Surlari–Mănăstirea enclosure flooding [6]

In order to increase the flooding effect, not only the Oltenița–Surlari–Mănăstirea enclosure, but also the Boianu–Sticleanu–Călărași enclosure (777 mil. m^3) were flooded. It can be noticed that together, these two enclosures have a total volume of about 1 billion m^3. This storage capacity is remarkable. However, the effect on downstream discharges and water level is again not significant. In Simulation no. 3, the Oltenița–Surlari–Mănăstirea enclosure was flooded first, followed by the flooding of the Boianu–Sticleanu–Călărași enclosure (Figure 34), while in Simulation no. 5, the order of flooding was reversed (Figure 35). The purpose was to find out if the order of enclosures flooding is important concerning both downstream discharges and water levels.

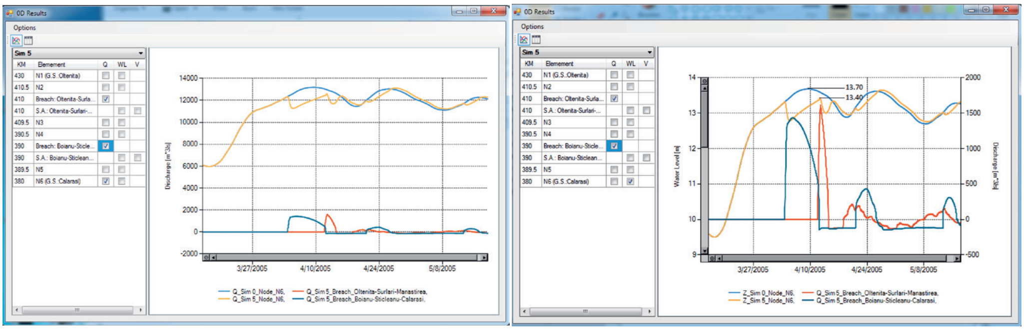


a) Discharges

b) Discharges through the breaches and water levels downstream

Figure 34. Results of Simulation no. 3 [6]

In Figure 34 a and 35 a, the discharges before and after the flooding are represented, while in Figure 34 b and 35 b, the discharges flowing into the two enclosures and the water levels on the Danube are put into evidence.



a) Discharges

b) Discharges through the breaches and water levels downstream

Figure 35. Results of Simulation no. 5 [6]

Apparently, the results are similar. However, comparing the water levels computed at Călăraşi in both cases (Figure 36), one notices that while the water level decrease in Simulation no. 3 is of 20 cm (from 13.70 m asl to 13.50 m asl), in Simulation no. 5, the effect is 50% higher, meaning 30 cm decrease (from 13.70 m asl to 13.40 m asl).

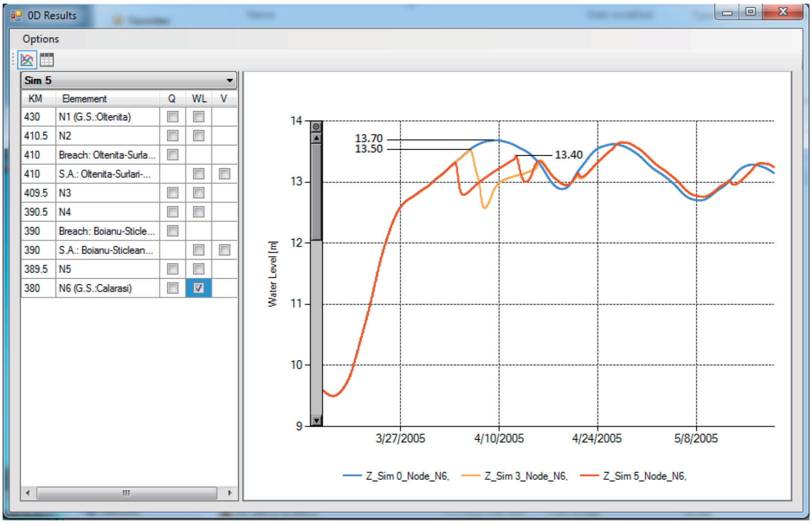


Figure 36. Comparison of water levels: Simulation no. 5 versus Simulation no. 3 [6]

However, this drawdown is not necessarily kept along the Danube River till Galaţi. The decrease of the water level will allow a supplementary volume from the tributaries to enter into the Danube River, by reducing the backwater effect of the high levels of the Danube upon the tributaries. Thus, the water levels in the Danube will continue to increase, especially due to the input of the Siret River.

By considering these facts, it can be concluded that local measures (like using mobile dykes) to protect the vulnerable areas of the towns along the Danube River are more preferable than flooding upstream enclosures.

The main conclusions of the model simulations are the following:

– The inundation of small or medium volume enclosures (less than 200 million m^3) has small effects on the water level decrease.
– The large enclosures should be inundated during floods peak, not before, in order to obtain the maximum effect downstream. Thus, a forecast of at least 7 days (including the forecast on the main tributaries: Olt and Siret) is necessary. Anyway, the water level decrease is quite small even when flooding large enclosures. If flooding all selected enclosures, with a total storage volume of 4.5 billion of m^3, the water level decrease at Galaţi in the most favourable conditions will be maximum 62 cm.
– The enclosures inundation is efficient only for large floods (more than 15,000 m^3/s) in order to use the enclosures storage volumes at the maximum extent.
– Local protection measures in Galaţi area should be initiated instead of expecting the decrease of water level by flooding the upstream enclosures.

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