Numerical simulation of a dam-break flood wave propagation in Belgium

M.J.J. Pirotton, A.G.H. Lejeune
(1) Associate Professor
(2) Professor
(1), (2) Members of the Belgian ICOLD Commitee

Department of Hydraulic Constructions, University of Liège,
6 Quai Banning, B - 4000 Liège, Belgium

ABSTRACT : The assumptions of the 1D Euler equations don't prevent a suitable software from reproducing all transient flows occurring downstream of a dam, from a daily exploitation to a dam collapse. However, the coexistence of several flow rates with transient jumps in ramified natural nets requires the development of suitable capturing methods. Numerical experimentations with finite element methods revealed that the dissipation has to take the flow conditions themselves into account. A conservative formulation, based on dissymetric continuous functions, was therefore investigated. An excellent fit between computed and analytic or gauged values assessed the adequacy of the code to simulate all transient flows. It was therefore applied to the worst conditions of a dam-break flood wave propagation. The computation was performed with a real topography of the reservoir and of the network in order to evaluate the measures to be taken for the population in the near downstream of the site.

1. INTRODUCTION

Water represents such a fundamental concern for Man that it has inspired at all times suitable creative capacities for the challenges to take up. Beyond of the philosophical inspirations of the biggest poets and superstitions that inflected quantity of ancestral practices for a long time, numerous proofs subsist in the hydraulic field that suggests the biggest humility considering so much talent and ability.

The accumulation of expertise, advances in fluid mechanics as well as the evolution of he technology lead today to a more complex resolution of the same fundamental hydraulic problems. Hydraulicians deal now with many antagonistic constraints and objectives and synthesize them in projects that are more oriented towards environmental aspects, submitted to sterner security criterias, working on sites more difficult to handle. Otherwise, the complete conception of a project implies to forecast the consequences of the structure dysfunction in order to anticipate and to minimize their effects.

The use of hydraulic softwares to manage hydraulic plants helps to tackle more scientifically the crucial question of optimal decisions.

These induce not only fluctuations on the variables of the system but influence its neighborings, from the inputs upstream to the floods that these decisions cause downstream by a routine or accidental exploitation. A complete numeric modeling has today to handle all these situations in order to help the manager, giving him a clear view of the hydraulic process generated by the possible decisions.

In this scope, we develop at the University of Liège a global approach to reproduce and forecast most of the hydrologic cycle flows. The proposed hydraulic models must be able to predict the inputs and outputs of any managed plant; in other words, they have to reflect the impact of normal or hazardous exploitation in the vicinity of a plant, from the very low water level situations up to dam-break flood wave propagations.

Privileging a systematic reasoning based on the physics of any free surface flows, three specific levels of modelization were considered, the first one devoted to the runoff processes of the raindrop after its fall on the ground, the second one focusing on the wave propagation in any natural network (propagation of lateral hydrographs poured in the drainage path, wave propagation in the managed plant), the last one forecasting all consequences in the downstream vicinity. Thus can be followed and explained the whole story of a flood formation, from the thin stream of water rushing down the hillside to the temporal and spatial evolution of water depth and river flow threatening riverside residents.

This global approach in the theoretical and numerical study of this kind of transient flows passes beyond large characteristic scale differences. They are especially closely bound by the common presence of hydraulic jumps in every type of out-flow.

The benefits of considering first common physical features of each flow, then of discerning numerical shortcomings also as cautions aimed to ensure the best numerical efficiency are highlighted in this application based on a very general flood routing.

2. OPEN-CHANNEL COMPUTATIONS WITH MIXED FLOW RATES

One-dimensional computations of unsteady free surface flows induced in nets of natural rivers are classically based on the Euler equations. However, the main assumptions that lead to this set don't prevent from reproducing all transient flows occurring downstream of the dam after its partial or complete, sudden or gradual, collapse.

The coexistence of several flow rates with shocks and bores in ramified nets of variable cross section arms requires the development of suitable capturing methods. They have to simulate sharp transitions without excessive smearing on several meshes or excessive growing of dissipative processes. The knowledge of factors affecting the intrinsic behaviours of the discretizations was devoted to the development of an original finite element method approach .

$$\frac{\partial}{\partial t}\begin{bmatrix}\omega\\ q\end{bmatrix} + \frac{\partial}{\partial x}\begin{bmatrix} q \\ q\,u + g\,p_\omega\end{bmatrix} + \begin{bmatrix} -q_L \\ -\,g\,\omega \sin\theta + n^2 \dfrac{g\,u|q|}{R_m^{4/3}} + g\,p_x + 2\left(\nu\,\omega \dfrac{\partial u}{\partial x}\right)\end{bmatrix} = \begin{bmatrix}0\\0\end{bmatrix} \tag{1}$$

with $$p_\omega(h) = \int_0^h (h-\xi)\, l(x,\xi)\, d\xi \qquad p_x(h) = \int_0^h (h-\xi)\,\frac{\partial l(x,\xi)}{\partial x}\, d\xi \qquad (2,\,3)$$

where F = the friction term ; ω = the wet cross-section ; u, q, q_L = the average speed in the section, the flow and the lateral inflow ; Z = the elevation of free surface ; ν = the kinematic viscosity ; ρ_ω = the parameter of uneven distribution of the axial speed in the section

In a first stage, the earlier formulation was fitted to the complete quasi two-dimensional set of equations (1). Nevertheless, numerical experimentations including shocks revealed an uneven improvement depending more particularly of the Froude number. This essential stage suggested that the dissipation has to take the flow conditions themselves into account.

The following formulation, based on dissymetric continuous test functions P was therefore investigated, which, in simplified conditions, agrees with schemes well known for their shock capturing capabilities :

$$[\omega\ q]^T = N'^T \,.\, Y \tag{4}$$

$$P^T = N'^T + \alpha_{N+1}\, A^T\, W^T \tag{5}$$

where α_{N+1} = decreasing function of the mesh size, including a parameter to be optimized, setting the degree of dissipation ; A = the convective matrix, W = the matrix including one-dimensional functions with degree (N+1), by reference to the classic interpolation functions expressed in N'.

The resolution method proceeds with uncoupling the equations by using a method of predictor-correctors. A first step consists in inverting the “continuity system” to obtain an upgraded approximation of wetted sections. The second one deals with the “momentum system” to give new discharge values. The resolution of transient flows in any networks is achieved through the introduction of Lagrange multipliers.

In the field of the hydraulic ressources management, the excellent fit between computed water depths and gauged elevations confirm the ability to simulate classical transient flows. Moreover, transient modifications of flow rates and bore propagations were tested to assess the accuracy of the code in handling subcritical and supercritical flows to numerically coexist at any moment in different locations of the discretization.

With the assurance of reliable simulations of unsteady appearances, movement and disappearances of discontinuities in the most various conditions of hydraulics, the code was applied successfully to real networks to accurately reproduce the flood waves propagation resulting from a sudden complete collapse of a large Belgian gravity dam.

3. SIMULATION OF THE EUPEN DAM_BREAK FLOOD WAVE PROPAGATION

If anticipation is a fundamental notion of any optimal management of hydraulic resources, it becomes vital when considering the exceptional circumstances that the International Committee for Large Dams recommends for a long time to consider : outcomes of sudden or progressive collapses of dams.

We will use the finite element software to simulate the first moments of such a "full size situation" in a real topography of a valleys network in Belgium.

The dam on the Vesdre river is a concrete gravity dam. It is 53 m heigh and 410 m long for a total storage capacity of 25 Mm3. It was completed after the second world war and was devoted to the water supply of Eupen situated in its close vicinity.

Besides, industrial purposes, especially the wool activity in Verviers and its neighborings, had induced to erect the first dam in Belgium on a tributary (Gileppe) to supply the Vesdre river just downstream of Eupen.

The dam situation on the main course of the river, upstream of the dam of La Gileppe, its vicinity from the Eupen city bring us to consider the hypothesis of its rupture.

The mode of collapse was chosen in accordance with the features of the studied work. By reference to the French principles on the subject for all concrete dams, we consider the extreme hypothesis of an instantaneous breaking, as blown by an explosion (Benoist). Then, the process is initiated with an almost-vertical fluid wall of 57 m height at rest.

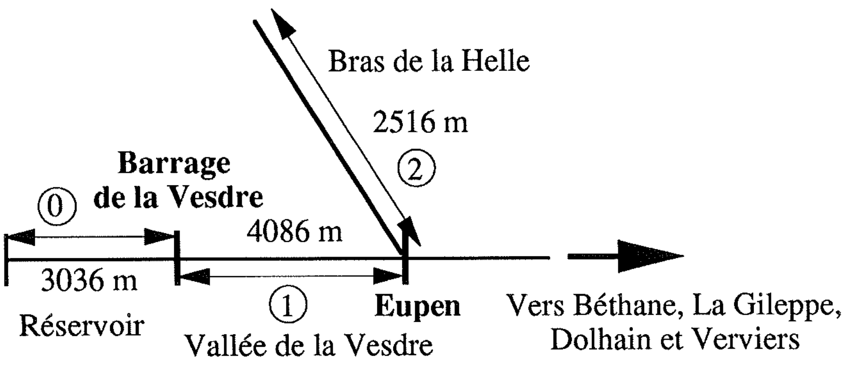

Figure 1 - Network considered for the modelling

This hypothesis, extremely severe regarding the cubages (450.000 m3 of concrete for the main structure), has to be analyzed in the context of instantaneous flood discharge foreseen at the dam site, that, we will see , exceeds 100.000 m3/s.

As the dam doesn't act as a control section, we include reservoir and valleys, with their real topography, in a complete modelling handling cross-sections generated from the digitalized maps of I.G.N (Institut Géographique National de Belgique) (figure 1).

3.1 Temporal evolution of the water level in the reservoir (arm 0)

Three significant points are to note when analyzing this first aspect of the computation :

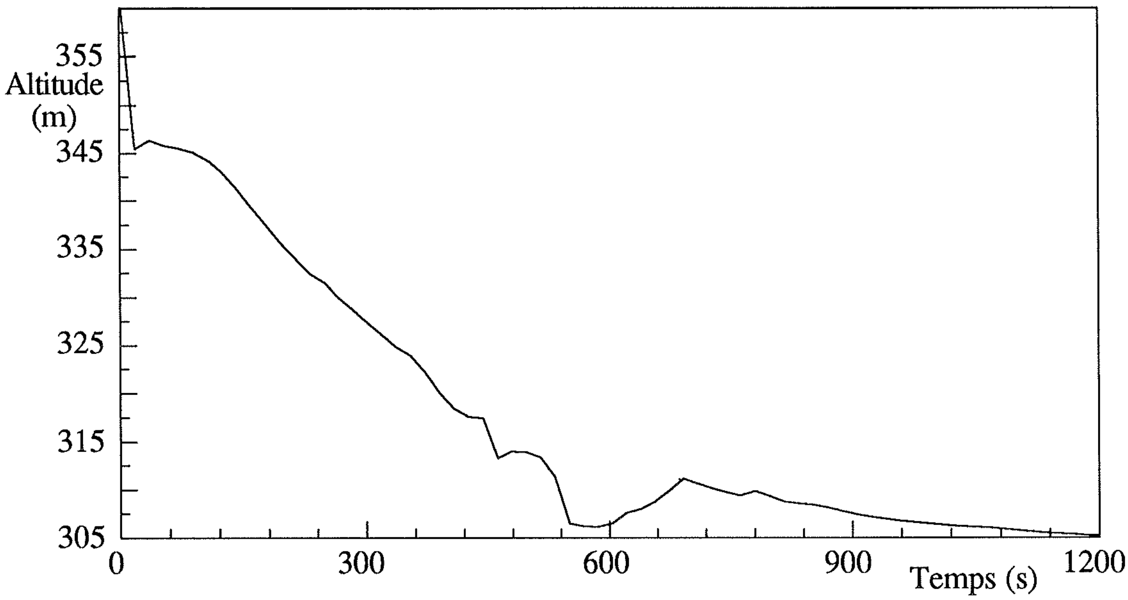

Figure 2 - Temporal evolution of the free surface altitude at the dam site

- Firstly, the negative wave initiated at the dam site comes to reflect on the upstream moving condition of the reservoir after 270 s. The celerity clearly decreases according to the depths into which the waves propagates.

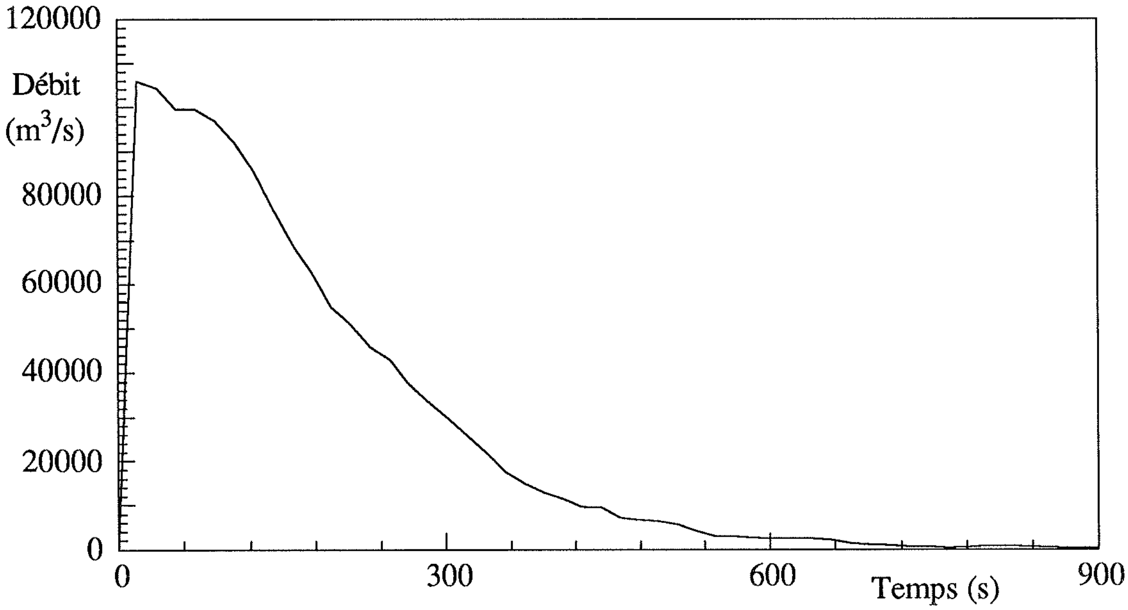

Figure 3 - Temporal evolution of the Vesdre discharge at the dam site

- The free surface altitude at the dam site, (see figure 2), is nearly steady during the first instants of the phenomenon. It goes instantaneously toward the value after the breaking. This result perfectly corroborates dam-break flood wave propagation theories in simple conditions, with a water height of 41 m. The discharge evolution at the dam site, (see figure 3), confirms that the peak of discharge is almost got instantaneously after the rupture to culminate to 108.000 m3/s. The nearly steady value of the 100.000 m3/s during 100 s is obvious, before a decrease that ends after 500 s.

- The instantaneous state of the water level after this period confirms that the reservoir is then nearly empty, with 97% of the initial volume already discharged. 1/4 hour after the disaster, the free surface flow characterizes the discharge upstream condition, representative of the inputs in the reservoir.

3.2 *Propagation of the wave between the dam and Eupen (arm 1)*

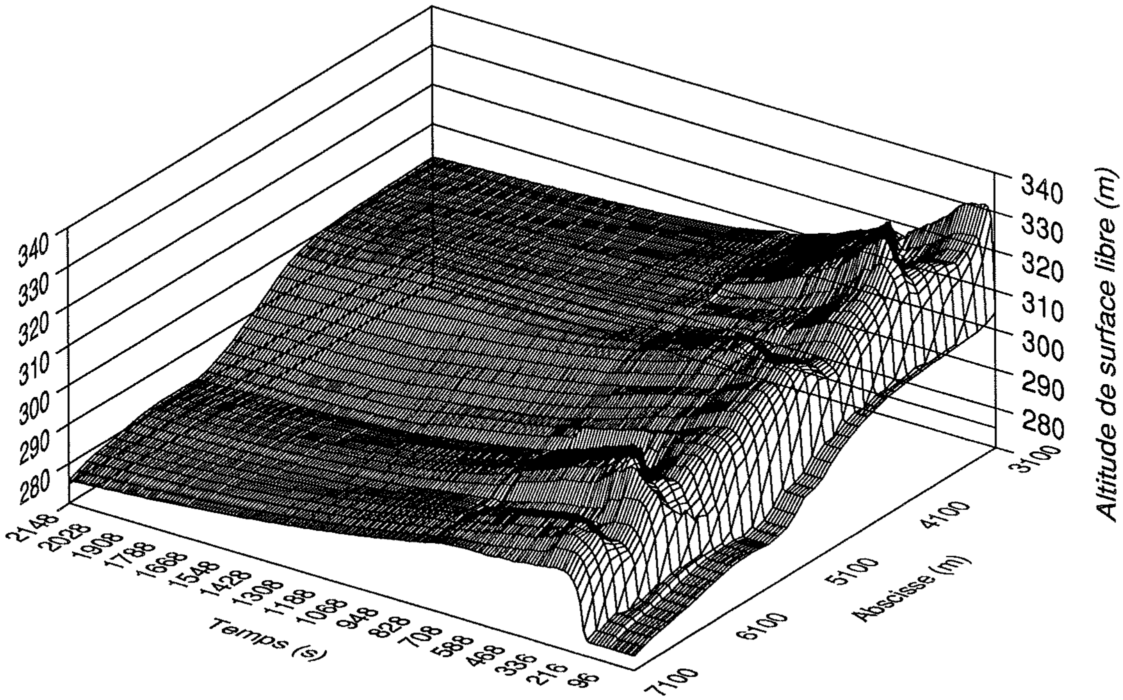

Figure 4 - Temporal evolution of the free surface altitude in the arm 1

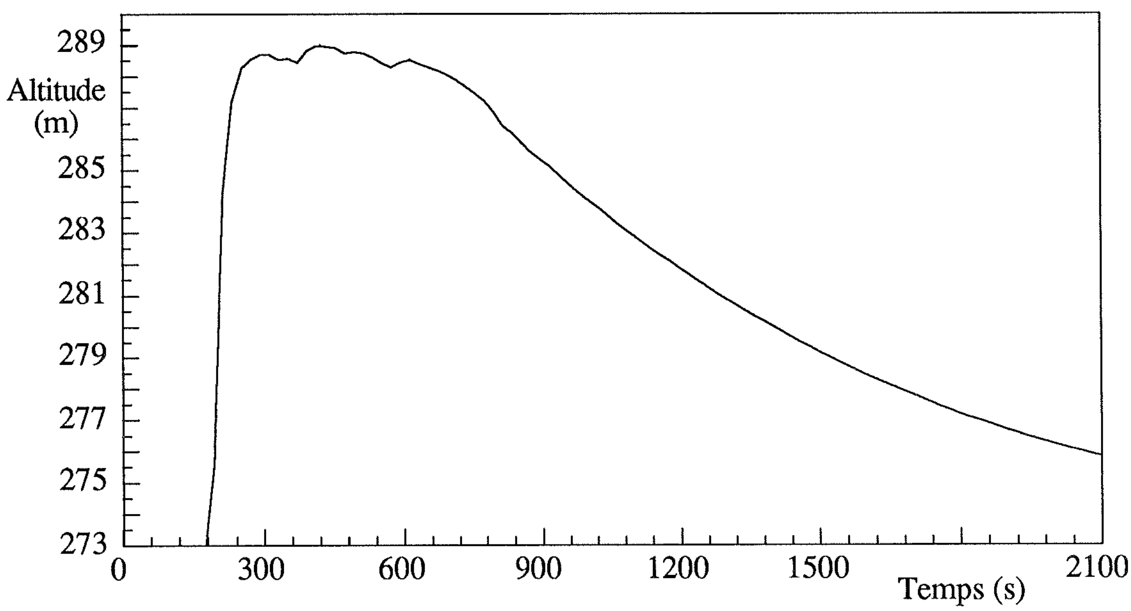

Figure 5 - Temporal evolution of the free surface altitude at Eupen

One of the fundamental purposes of such simulations consists in determining the lag time for the city of Eupen. The simulation doesn't introduce any favorable circumstance for a fast propagation since the Manning coefficients reflect the presence of wooded areas just downstream of the dam, as well as the presence of houses quite upstream of the center of the city.

Nevertheless, as highlighted by the temporal evolution of water levels on this section (Figure 4), the propagation is extremely fast. A sharp wave front reaches the confluence with the Helle after only 200 s, ie an average speed superior to 20 m/s.

The Figure 5 describes the temporal evolution of the free surface altitude at Eupen. It shows indeed that more than 10 m of water already submerge the city after 210 s, while the maximum water height is got after 300s, with 16 m water depth. It must be pointed that a quasi steady stage maintains water depth values superior to 15 m during 500 s before starting the subsidence stage.

Thus, even if the friction influence is primordial, by comparison with analytical solutions in restrictive conditions, it is obvious that the city is exposed, especially since the nearly vertical front induces maxima of water height just a short time after its arrival in the city.

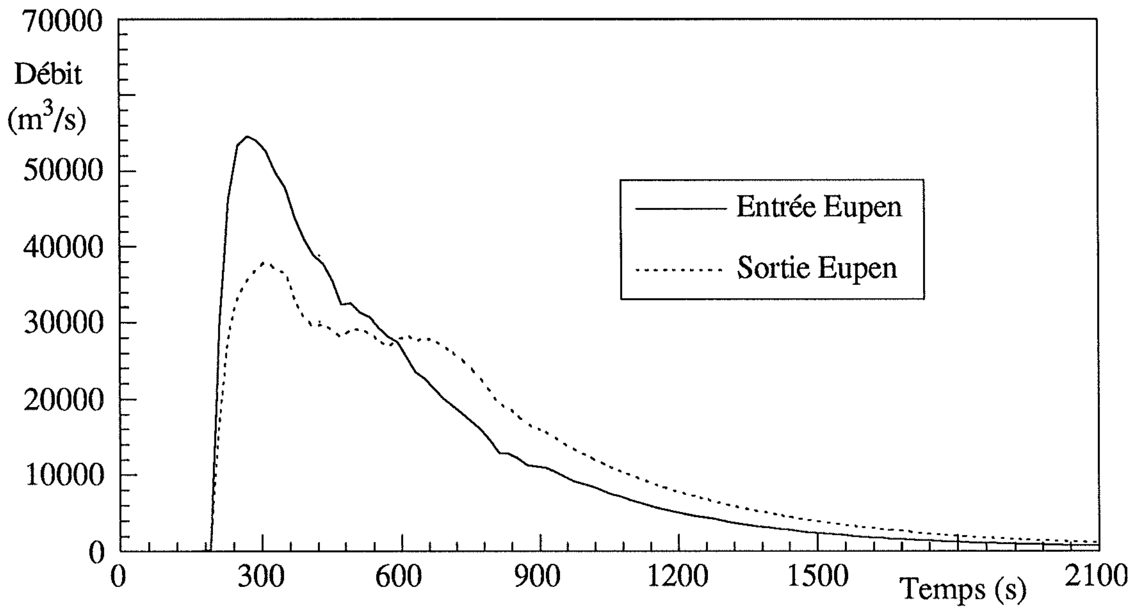

Figure 6 - Temporal evolution of the Vesdre discharge at Eupen

The temporal evolution of discharges at Eupen, illustrated on the Figure 6, exhibits a peak after 260 s with a maximal value of 54.800 m3/s.

In spite of the important part played, in this junction, by the lateral arm of the Helle (see the hydrograph modifications between the entrance and the exit of the city on the Figure 6), the comparison of this signal with the one computed at the dam site confirms a fast smoothing of discharge curves. Indeed, we only recover 50% of discharge extrema recorded at the dam after nearly more than 4 kilometers of propagation.

Beside these serious consequences for the population, already suspected a few years ago, when Eupen refused a first project on a near upstream site, we can focus on some flow features upstream of this strategic point.

The most interesting aspect lies in the apperance of jumps in several abscissas, moving then before disappearing very gradually during the subsidence. The most visible one on Figure 4 is the jump that appears approximately 900 m downstream of the dam. In this location we cumulate locally two topographic features : a widening of section and a decrease of bottom slope. Some instantaneous flow transitions are especially apparent with differences of water height that exceed 16 m on a mesh! This jump goes up again progressively toward the the dam site where it influences the outflow at the end, while disappearing, as suggested by the "bump" in the curve (Figure 2) after nearly 600 s.

If this detail resorts to numerical interest, since its impact on results for an uninhabited region is not significant, it is primordial to demonstrate that the software is able to handle them without any numerical noise or smearing on the computation.

3.3 Propagation of a secondary front in the lateral arm of the Helle (arm 2)

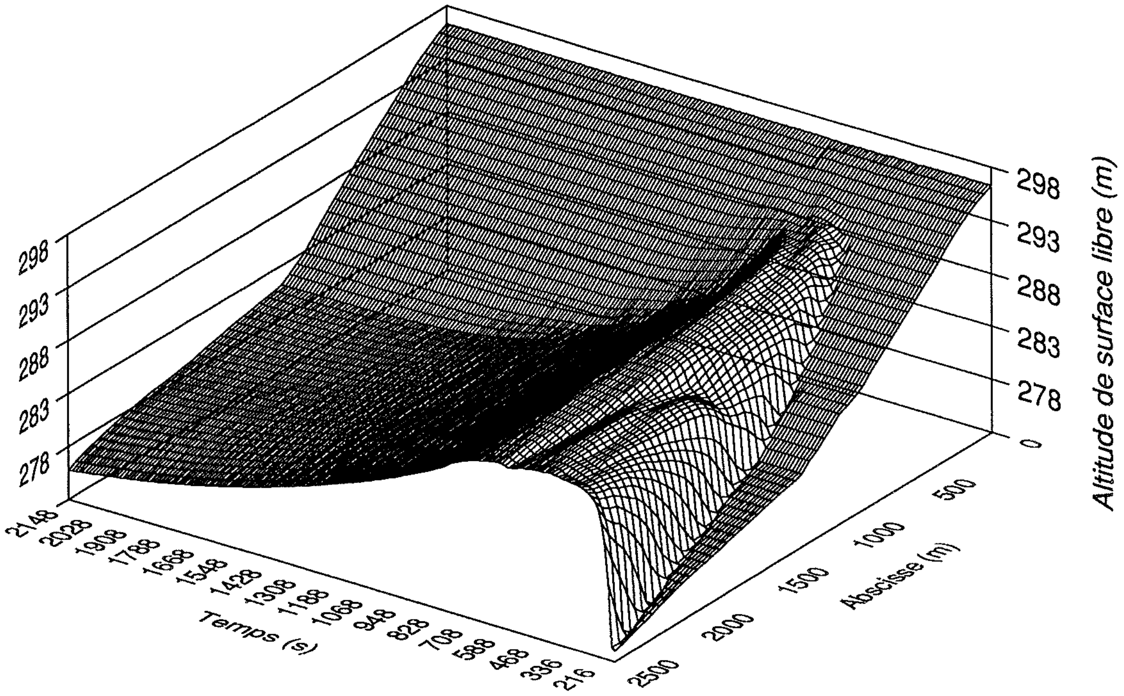

Figure 7 - Temporal evolution of the free surface altitude in the arm 2

One can admit intuitively that the storage capacity of the lateral arm of the Helle can only contribute to somewhat attenuate catastrophic effects of the wave on maximum water depths reached in the city.

Figure 7, with the temporal evolution of water levels of the Helle, shows effectively that a secondary wave goes up on more than 2100 m until to it dissapears on the progressive raising of the bottom. In this figure, the confluence (Eupen) is characterized by x = 2500 m, while the upstream boundary condition acts at x = 0 m.

The temporal evolution of discharge in the Helle river at Eupen (Figure 8) shows a maximum value of 20.000 m3/s as well as a reversing at t = 590 s. This reference allows to estimate the maximum volume stored in this ramification that is to say 16 % of the initial volume of the reservoir.

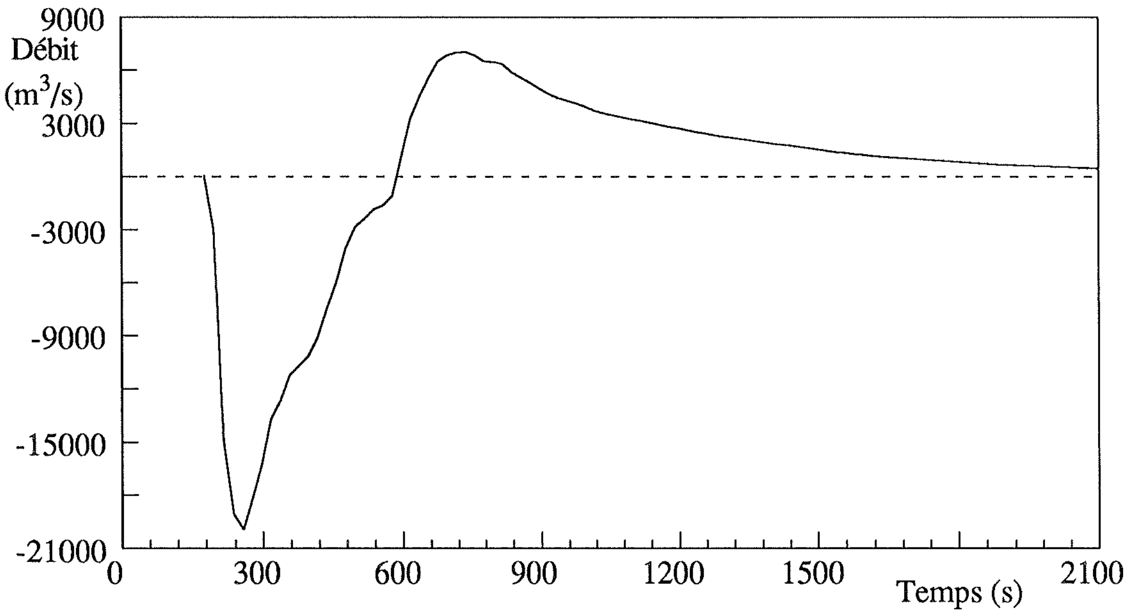

Figure 8 - Temporal evolution of the Helle discharge of at Eupen

4. CONCLUSION

This computation on the Vesdre topography was carried on beyond the limits presented in this paper. It must be putted in the context of prevention that incites many countries to undertake impact studies to integrate them in safety policies for people and goods.

Interesting perspectives open up thanks to the improvement of hardware and sofwares for the computation, in real conditions, of such phenomena for the setting up of efficient alert devices and emergency maps.

Apart from these fundamental preoccupations, this case study demonstrates that the reliable interpretation of the result lies on the numerical features of the software and especially on its ability to react adequately to the coexistence of regimes without inducing excessive smearing or numerical noise. Two essential constants underlies the obtention of good results : the importance of a good knowledge of the hypotheses and the limit of validity of theoretical sytem of equations and the adequate exploitation of the numeric behaviors of the software. This conclusion outlines the difficulties and the dangers to benefit from the present multiplication, in hydraulics, of sofwres usable as " black " boxes.

5. BIBLIOGRAPHY

PIROTTON M., Modélisation des discontinuités en écoulement instationnaire à surface libre (Du ruissellement hydrologique en fine lame à la propagation d'ondes consécutives aux ruptures de barrages), Editions LHCN, 479 pages, 1994.