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River modelling and flood mitigation in a Belgian catchment

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This paper describes the steps followed to propose solutions to recurring flooding problems in a Belgian catchment. First, the hydraulic capacity (maximum discharge before bank overflow) of the cross-sections was computed along the entire river by an iterative one-dimensional steady-state approach. In order to carry out these simulations, cross-sections from site surveys of the river were integrated into the model, as well as hydraulic structures such as culverts, footbridges, and pipes. Second, the flooding problem was analysed with a time-dependent approach consisting of simulating floods following extreme rainfall events. The hydrological aspect was studied in a spatially distributed way using a multi-layer hydrological model. The available data on the basin such as the digital elevation model, the land-use and the pedology were exploited to identify the basic modelling parameters. The hydrological contribution was routed by a one-dimensional network resulting from the merging of the digital elevation model-based and the cross-section-based river networks. According to the results of the aforementioned steps, various local and catchment-wide solutions against flooding were proposed and analysed. The comparison of simulated situations before and after these improvements allowed the effectiveness of the proposed solutions to be checked.

1. INTRODUCTION

As extreme rainfall events are occurring increasingly often, efficient management of river basins is necessary. This implies careful studies of the catchments, including both the hydrological and hydraulic aspects of the flood production mechanisms. A detailed hydraulic study of the river is necessary to estimate the acceptable discharges and to point out the problems leading to local overflowing. The hydrological part is required to compute the runoff production during a particular rainfall event, which is necessary to assess the efficiency of solutions such as water storage in storm basins.

In the present study, an application of a complete modelling system developed at the University of Liege (WOLF) for the 'Rieu des Barges' river catchment is described. The modelling steps are detailed, showing how the available data [such as the digital elevation model (DEM), land-use maps, river cross-sections and hydraulic structure descriptions] were exploited to derive potential solutions to mitigate the floods.

2. THE 'WOLF' MODELLING SYSTEM

The present study was completed using the 'WOLF' modelling system, developed at the University of Liege. WOLF includes a set of complementary and interconnected modules for simulating free surface flows: process-oriented hydrology, one-dimensional (1-D) and two-dimensional (2-D) hydrodynamics (Dewals *et al.*, 2008b; Erpicum *et al.*, 2009b; Roger *et al.*, 2009), sediment (Dewals *et al.*, 2008a) or pollutant transport, air entrainment, as well as an optimisation tool based on genetic algorithms. Other functionalities of WOLF 2-D include the use of moment of momentum equations, the application of the cut-cell method, as well as computations considering vertical curvature effects by means of curvilinear coordinates in the vertical plane (Dewals *et al.*, 2006).

The hydrological component of the WOLF modelling system is physically-based and spatially distributed. It computes the main hydrological processes using a multi-layer model with depth-integrated equations (Figure 1). The overland flow is computed using the diffusion wave equation, which is obtained by ignoring the inertia terms compared with the gravitational ones, friction and pressure heads in the well-known shallow water equations (Archambeau *et al.*, 2004). The velocities are linked to the friction slope using the Manning–Strickler friction law.

The infiltration is calculated using a Green–Ampt infiltration law (Chow *et al.*, 1988) with the impact of the land-use taken into account by using effective values for the infiltration coefficients, as proposed by Nearing (Nearing *et al.*, 1996). The
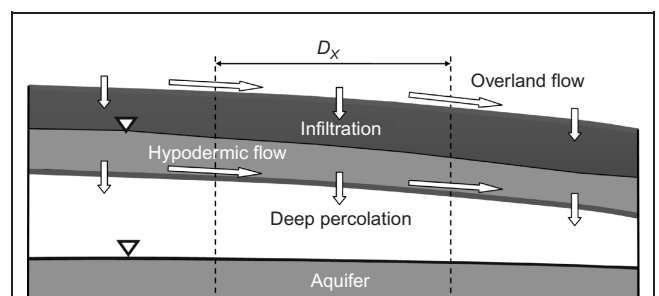

Figure 1. Flow layers computed in the hydrological model

subsurface flow is computed with the depth-integrated Darcy equations[2] and is therefore modelled with a diffusive wave equation similar to the surface flow equation.

The river flow inputs generated from the hydrology module are routed in the river network by way of the 1-D module, which solves the conservative form of the 1-D Saint-Venant equations. The hydrological inflows are treated as lateral inputs (source terms). The hydrological and the river flow equations are therefore uncoupled. The spatial discretisation of the 1-D equations is performed by a widely used finite volume method. Flux treatment is based on an original flux-vector splitting technique developed for WOLF. Fluxes are split according to the sign of the flow velocity, requiring a suitable downstream or upstream reconstruction for both parts of the convective term according to a stability analysis (Erpicum *et al.*, 2009a). Efficiency, simplicity and low computational cost are the main advantages of this scheme. An explicit Runge–Kutta scheme or an implicit algorithm (based on the GMRES) is applied to solve the ordinary differential equation operator, and an original treatment of the confluences based on Lagrange multipliers allows the modelling of large river networks in a single way. Both free-surface and pressurised flows can be modelled simultaneously using the same set of equations thanks to the Preissmann Slot artifice (Preissmann, 1961). Indeed, it is well known that the only difference between the Saint-Venant equations for open channel flow and the incompressible pressurised flow set of equations lies in the pressure gradient term. Analytical developments, initially presented by Preismann, show that this difference is overcome by adding a narrow slot at the top of the pressurised flows. In this way, pressurised flow can be calculated through the free-surface set of equations.

A number of other sophisticated computational rainfall–runoff models have been developed and implemented in the last two decades, including Mike Basin and Mike SHE (Graham and Butts, 2005), HEC HMS (Feldman, 2000) or SWMM (Rossman, 2004) to name just a few. In contrast to Mike Basin, which is based on the lumped conceptual hydrological model NAM (DHI, 2000), the present WOLF modelling system relies on a spatially distributed and process-oriented approach as in MIKE SHE. In addition, similar to Mike SHE, fully dynamic flow modelling is used in WOLF to compute flood routing in rivers, whereas simplified approaches such as the Muskingum–Cunge or kinematic wave approximation are used in Mike Basin, HEC HMS and SWMM. However, the later models enable continuous simulations, whereas WOLF applies mainly to event-based simulations.

3. THE 'RIEU DES BARGES' BASIN

The 'Rieu des Barges' is a river located in Belgium. Following some important rainfall events in recent years, the river basin has suffered from numerous damage events from overland flow and river bank overflows. The flood frequency and subsequent damage brought about the need for a study to propose mitigation solutions. The total surface of the river basin is 38·6 km^2. The basin slopes are variable, with a mean value of 2%. The land cover is mainly composed of crops (77%) and meadows (14%). The urban areas cover 3·5% of the basin.

4. METHODOLOGY

The study was completed by following four main steps. The first step covered the pre-processing of the hydrological data (for the runoff computation) and of the hydraulic data (for the river flow computation). In the second step, the 'Rieu des Barges' was studied from a hydrodynamic point of view, in order to draw conclusions on the acceptable discharges, sensitive areas and overflowing zones. In the third step, a coupled hydrology–hydrodynamics approach was used to study a flood event. The final step consisted of the analysis of the preceding results and an assessment of mitigation solutions.

5. DATA PROCESSING

The necessary data were prepared using the geographical information system (GIS) interface of WOLF, using pre-processing tools to convert raw data. The soil properties were extracted from pedologic maps using pedotransfer functions (Rawls and Brakensiek, 1989). The DEM was processed in order to remove depressions, using an algorithm proposed by Martz and Garbrecht (Martz and Garbrecht, 1999), and a 'stream burning' method (Callow, 2007; Saunders, 1999) was applied in order to make the DEM-based flowpaths coherent with the real ones obtained from site surveys. Specific engineering structures which significantly modified the flowpaths were taken into account in this process. For example, a high-speed railway crosses the catchment, and the flows are therefore re-routed through drainage channels along the railway.

Site measurements of the cross-sections were available on the two main rivers of the catchment ('Rieu des Barges' and 'Rieu de Taintignies'), as well as a description of existing hydraulic structures along these rivers (such as culverts and pipes). A few pre-processing steps were necessary to prepare this data for the 1-D simulations.

Some cross-sections were only composed of three points (corresponding approximately to the lower point of the bed and to the tops of the two banks). Linear interpolation between these points would have produced a triangular section which would not have been realistic (Figure 2, dotted line). Therefore, additional points were added to these sections using fixed values for the bank angles (30° from the vertical axis) and the bed angle (5° from the horizontal axis). These values are based on visual site estimations. Moreover, as most of the cross-section data were limited to the top of the banks, an enlargement was added to every section to represent the floodplains (Figure 2). This enlargement of the section width at the top of the banks was arbitrarily fixed at 10 m. As the aim of the study is a situation without any bank overflowing, this value does not have any impact on the simulations of the final
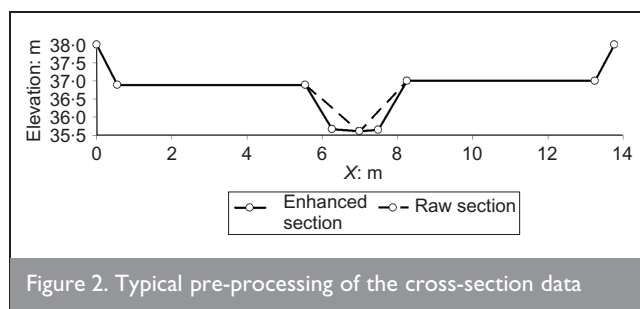
Figure 2. Typical pre-processing of the cross-section data

state (river with various improvements for the flood mitigation). Therefore, the purpose of this enlargement is only to improve the analysis of the initial situation, and it was not worth carrying out additional site surveys to refine this estimation.

The river sections were then interpolated on a regular mesh (5 m cells), and the hydraulic structures were added. The closed sections were treated the same way as open sections, except that an artificial slot was added at the top of the section (Figures 3(a) and (b)). In the classical Preissmann theory, the slot width reflects the pipe dilation and the water compressibility under a pressure fluctuation, and has therefore very small orders of magnitude (about 10^{-5} m for the pipes existing in the 'Rieu des Barges' river) (Kerger *et al.*, 2009). Using this value would have led to extremely small time steps. However, in hydrological simulations, it is not necessary to compute highly transitive phenomena such as a water hammer. As the evolution of the flow over time is much more gradual, a much larger Preissmann slot can be used to compute the pressurised flows (a 0·1 m width was used in this application).

Using these pre-processing steps to prepare the data from site surveys, the main river could be modelled with the 1-D model. In order to cover the whole catchment, the river network had to be completed using other data inputs. An automatic process was developed in order to combine the 1-D network created previously (using the cross-section data and including the hydraulic structures) with a second 1-D river network generated on the basis of the DEM (modified as described above). Both networks were merged using the following steps.

(*a*) The river branches were split into multiple parts at each characteristic point (confluences and ends) of both networks.
(*b*) The DEM-based river parts were replaced by the corresponding parts from the other (more accurate) network, where available.
(*c*) Special treatments were applied to deal with the inconsistencies between both networks, such as bed level discontinuities at the junctions.
(*d*) The split river parts were merged back to form the complete river network.

The resulting network therefore covered the whole catchment, and included detailed data such as cross-sections and hydraulic structures where they were available.
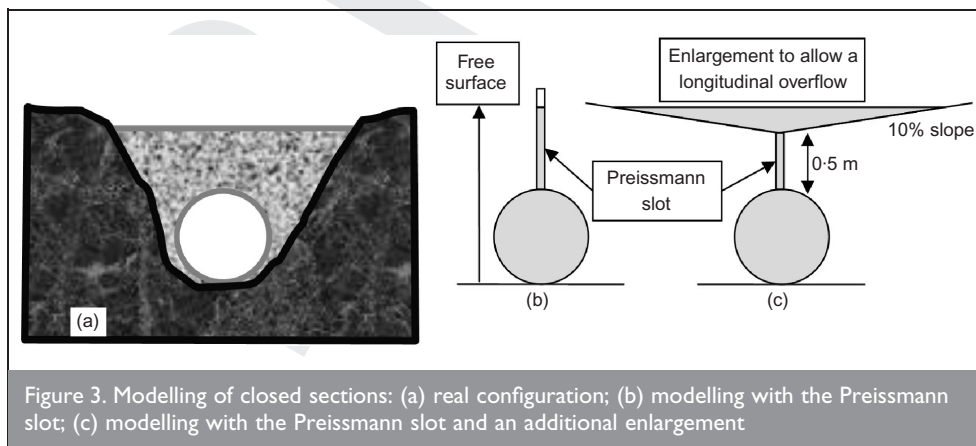
Existing and planned storm basins were also included in the simulations. In particular, two storm basins have a special operating mode. The first one, located on a tributary of the main river (Figure 4 – basin no. 1), collects the water from the drainage system of a part of the railway (which crosses the tributary), and the river discharge exceeding 1 m³/s. These inputs are routed to a buffer tank and are then pumped to the main reservoir. The stored water is evacuated through an opening, with a discharge function of the water depth in the basin. The real water depth–volume relations were therefore implemented for the simulations.

The second basin (Figure 4 – basin no. 2), receives the inputs from another portion of the railway drainage system. The water simply enters the basin by gravity. The water is evacuated through two constant discharge pumps. Each one starts when the water surface in the basin reaches a specific elevation (43·45 and 44·16 m above sea level), and stops when the reservoir is nearly empty (water surface elevation = 42·24 m). This operating mode implies a discontinuous outflow. Moreover, as the first pump only activates when the level reaches 43·45 m, the initial water surface elevation (at the beginning of the storm) was fixed to this value in order to represent the worst case.

6. SIMULATIONS AND RESULTS
As mentioned in the methodology, the river was first studied using a purely hydrodynamic approach. A steady-state discharge was set in the river as a linear function of the drained basin. This discharge was progressively increased to a maximum of 0·35 m³/s per km² (over this value, most of the river overflows), and, for each point of the river, the value corresponding to the first overflow was noted. This method therefore links an 'acceptable discharge' to each point of the river and shows the limiting cross-sections. The river sections that did not overflow at the maximum simulated discharge (and have therefore an acceptable discharge over 0·35 m³/s per km²) are not represented in Figure 5.

In Figure 5 and subsequent similar graphs, the distance along the river is measured from the most upstream point for which cross-section data was available (this point has a drainage basin area of 2·2 km²). The large dots show the cells where cross-section data were available from site surveys, whreas the small ones correspond to the cells with interpolated cross-sections. As can be seen in this figure, the acceptable discharge increases towards the downstream end of the river, but many restrictive areas present overflows at relatively low discharges.

The free surface corresponding to a fixed discharge can also be plotted all along the river. It allows a better understanding of the flow dynamics and emphasises the parts of the river where an important head loss exists.

Each sensitive area was

Figure 3. Modelling of closed sections: (a) real configuration; (b) modelling with the Preissmann slot; (c) modelling with the Preissmann slot and an additional enlargement
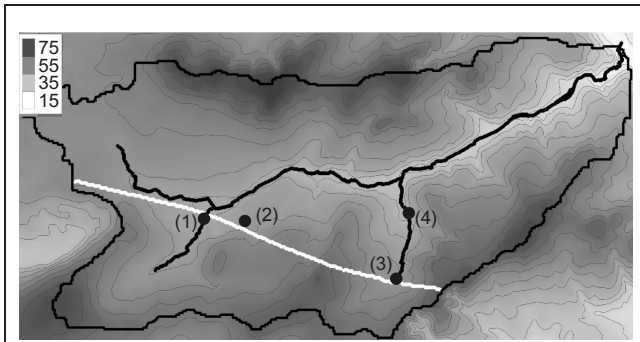
Figure 4. Position of existing and planned storm basins. The black lines represent the river and the catchment limits, and the white line stands for the drainage paths along the railway
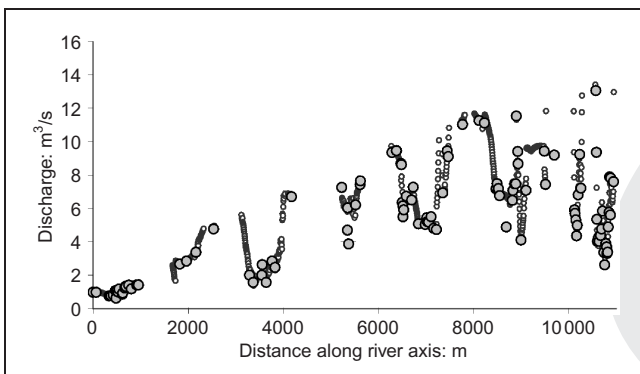

Figure 5. Acceptable discharge in the river before bank overflow

studied to find the causes of the overflow. Following this analysis, various local solutions (such as the enlargement of some pipes and culverts, or the removal of some obstacles) were proposed to decrease the risks of flooding. It was found that these local improvements should be combined with other catchment-wide solutions (such as the installation of storm reservoirs) for optimal efficiency. However, due the presence of uninhabited woods along the river, some areas can be flooded harmlessly and do not need any specific modification.

After these steady-state simulations, the river basin was studied as a whole, combining the hydrological and hydrodynamic approaches for the simulation of an extreme rainfall event (Figure 6). The rainfall distribution was generated for three
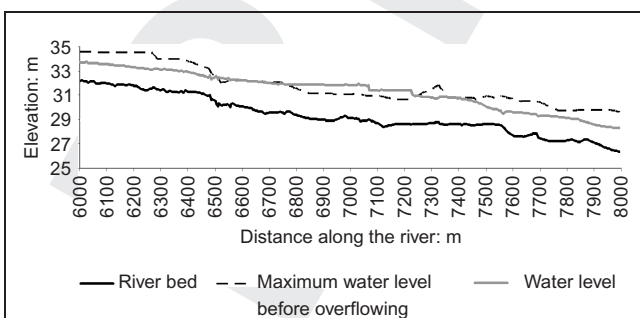

Figure 6. Water levels in the river for a linear input of 0.25 m³/s per km²

return periods (10, 25 and 100 years), using the alternative block method (Chow et al., 1988). However, the results presented in this paper correspond to the 25 years return period rainfall, which was contractually fixed as the design storm. The rainfall was specified as uniform over the catchment and its intensity was multiplied by an areal reduction factor of 0·75.

A fundamental question arose from these simulations about the modelling of the flow through structures such as pipes and culverts. When they are simply considered as closed sections, the water has no other choice but to pass through the structure. Therefore, when the discharge becomes significant, the water level upstream of the structure increases until there is a sufficient head to force the whole discharge through the structure. This effect can therefore cause an unrealistic water storage upstream of the structure. In real conditions, when the level exceeds a threshold, the water can overflow the structure. However, no data were available to identify the pipes and culverts which could be overflowed and the corresponding water level threshold. Therefore, two extreme cases were defined. In the first one, no longitudinal overflow was allowed. In the second one, there was an enlargement of the closed section starting 0·50 m above the top of the waterway, with a lateral slope of 10% (Figure 3(c)).

Figures 7 and 8 show the hydrographs at six locations along the river for a 25 year return period storm, for each situation.
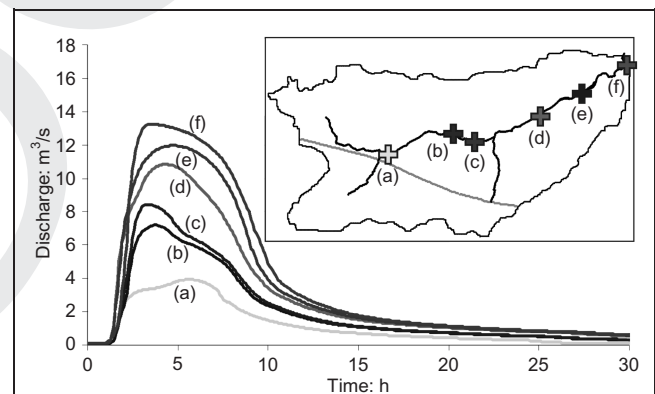

Figure 7. Hydrographs for a 25 year return period flood (no structure overflow allowed)
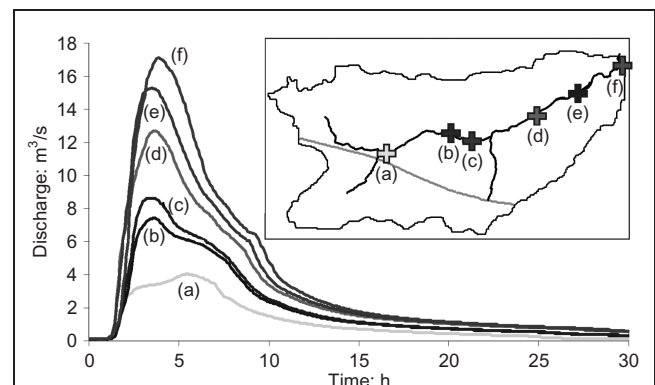

Figure 8. Hydrographs for a 25 year return period flood (structure overflow allowed)

In the first case, the water storage upstream of the structures considerably decreases the discharge peaks (attenuation effect) and increases the maximum water levels. In the second case, the water levels are lower, but the discharges are higher (no attenuation effect). Therefore, in a conservative approach, the first case was used to estimate the maximum water levels in the river, whereas the second case was preferred to compute the maximum discharges.

The maximum water levels at each river section were computed and the resulting water levels in the river were found to remain similar to those computed in the steady approach. The lessons drawn from the previous analyses are therefore still applicable for the unsteady simulations.

Various solutions were proposed in order to mitigate the potential floods. In addition to some local improvements in areas where important head losses were identified (cf. the analysis with the steady flow), retention basins were pointed out as a relevant catchment-wide solution. After an analysis of the possible locations (depending on the potential sites identified by the local water authorities and their interest from a hydraulic point of view), three main sites were located to install retention basins. Three characteristics needed to be determined: the threshold river discharge from which the basin started to store the water (bypass discharge), the emptying discharge (evacuation from the basin), and the basin volume. The total of the two first characteristics (bypass and emptying discharge) depends on the acceptable river discharges (see Figure 5). However, due to the very low hydraulic capacity of some river parts, this approach would have led to excessive storage volumes. Therefore, in some areas, the discharge left in the river still exceeded the acceptable threshold, and additional local works and improvements had to be considered.

Figure 9 shows the simulated discharges in the 'Rieu des Barges' when adding the three storage basins. It can be seen that the effect of a reservoir decreases towards the downstream end of the river, and it is therefore necessary to distribute the basins along the whole river. The graph also shows the river sections where the acceptable discharge is exceeded.

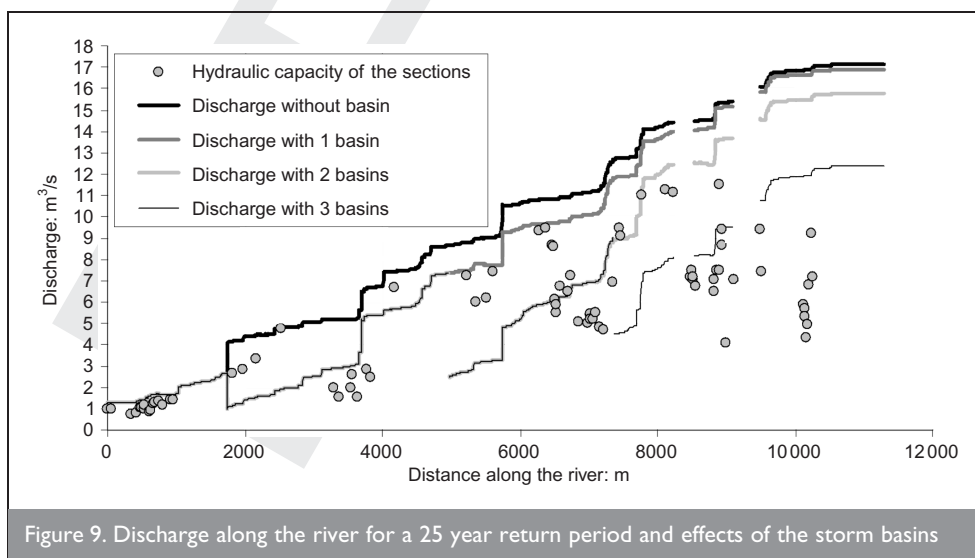The simulations also provided the change in stored water volume during the flood for each reservoir. The maximum volume could therefore be used for the sizing of the potential retention basins. For a 25 year return period, the following volumes were found (Table 1).

Even with three storm basins, the acceptable discharge was found to be still exceeded in some sections. A number of reasons can explain this.

(a) The basin volumes have to be limited due to ground occupation and cost limits.
(b) Unlicensed constructions have been erected in some sections of the river, resulting in significant local narrowing of the river.
(c) The acceptable discharge may be underestimated in some sections due to inaccuracy in data.

In some areas, while the 'acceptable discharge' (corresponding to a flow maintained in the main channel) was exceeded, local overflowing of the banks is harmless and can therefore still be acceptable. In contrast, in other areas, additional local solutions are necessary, such as the raising of the banks or modifications of the river course. For example, in an area in the downstream part of the basin, the river narrows without any possible enlargement due to the presence of neighbouring houses. An important reduction of the flood discharge is therefore needed, and the possible diversion of a part of the discharge through a culvert parallel to the river was analysed. The flow distribution was computed on the basis of the maximum allowed discharge in the river (Figure 9).

The study pointed out the importance of the flow dividing device. A structure made of two weirs was proposed to divide the flow between the river and the culvert (Figure 10). This ensures a distribution of the flow which only depends on the total discharge, and is independent from the water level in the river and the culvert due to supercritical flow conditions on the weirs.

The dimensions of the structures were defined in order to obtain the desired flow division. The level of the weir diverting into the river was specified as lower than the other one, so that at low flows, the entire discharge stays in the river.

The flood event was then simulated in this new configuration and, as expected, the results indicate that the bank overflows along the downstream part of the river can be avoided by the proposed solution (Figure 11).

7. CONCLUSION

This paper presents a practical application of a complete modelling system including within a unique framework a pre-processing tool, a hydrological model and a module for the 1-D simulation of river flows. The

Figure 9. Discharge along the river for a 25 year return period and effects of the storm basins

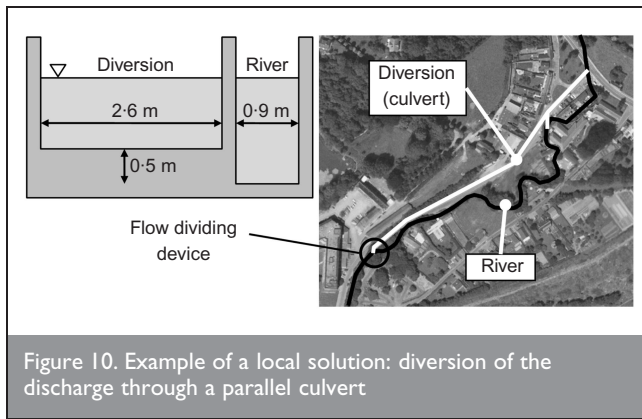|	Bypass discharge + emptying discharge: m^3/s	Storage volume: m^3
Basin no. 1	1	61 700
Basin no. 2	2·5	45 100
Basin no. 3	4·5	32 500

Table 1. Characteristics of the storm basins

Figure 10. Example of a local solution: diversion of the discharge through a parallel culvert
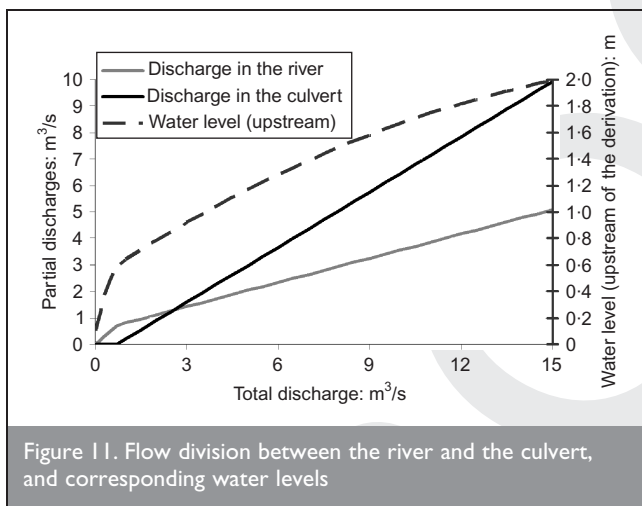

Figure 11. Flow division between the river and the culvert, and corresponding water levels

WOLF modelling system was used to find and assess solutions for the flood mitigation in the 'Rieu des Barges' catchment. The study was conducted by following four main steps. In the first one (data processing), the DEM was modified in order to be consistent for hydraulic numerical modelling, the distributed model parameters were generated from land-use and pedologic maps, and an automatic process was applied in order to generate a complete 1-D network by combining data from the digital elevation model, and from site surveys. Existing and projected storm basins were also included in the simulations with a specific implementation of the way they operate. In the second step, a study of the river hydrodynamics allowed problematic areas to be identified, the formulation of proposals for local solutions and assessment of the maximum acceptable river discharge. In the third step, a coupled hydrology–hydrodynamics approach was used to compute the hydrographs in the river for an extreme flood event generated using the alternative block method. Finally, various

improvements to mitigate the harmful effect of floods were modelled.

The study showed that catchment-wide solutions such as the implementation of storm basins are an interesting solution, but have to be combined with local improvements, such as the diversion of a part of the river discharge, the raising of the banks or the removal of obstructing structures.

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