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Validating a simplified model for flood hazard downstream levees

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ABSTRACT: For assessing the risk caused by the flooding due to a breach across a levee along a river, one should simulate both the process of breaching and the process of propagation in the flood plain. If the breach location is unknown, a lot of locations should be investigated to estimate the peak water depth, the time of arrival of the wave and the peak velocity in any point. To reach this last aim, a simplified method was defined; it includes a simplified erosion model of the levee and a simplified propagation model. To evaluate the latter model, results were compared to results of a 2-D reference model, which permits to assess the error or uncertainty in case a simplified model is applied for typical situations. Although the development of the simplified model is based on non-dimensional variables, high discrepancy appeared when the comparison concerns an experiment performed in laboratory with much lower water depths. Thus, use of the simplified model should be limited to a specific range of field cases.

Keywords: Levee breaching, Wave propagation, Flood risk, Numerical model

1 INTRODUCTION

Extreme flood events provide a large set of examples in which levees supposed to maintain the flow in the main channel breach causing high damages in the protected areas. In France, the more recent examples are the floods of 2002 and 2003 along the Rhône, Gard and Vidourle Rivers. In such events, the location of the breaches is unknown before the flood and the damages; then, casualties are often due to the absence of suitable local mitigation measures. However, even if some studies are performed to determine the most sensitive areas for breaching, the length of levees that can be breached remains high. Thus, assessing the consequences of various types of breaching in various locations for various hydrological events requires a lot of hydraulic calculations. In order to obtain accurate results of the flow features downstream from the breach, models solving 2-D shallow water equations can be used (Jaffe and Sanders (2001), Harms et al. (2004)) although sensitivity analysis are always necessary because parameters such as roughness coefficients cannot be calibrated easily (because a similar event never occurred). Then, calculation time for one single breach location and one hydrological event amounts to several days, which generally results in a study limited to a few locations of breaches.

To avoid this unsatisfactory situation, Cemagref started to develop a simplified model named CastorDigue in order to enable the study of a large range of events and locations because of a much shorter calculation time. Then, more accurate calculations will be limited to the more complex situations and the higher stakes. But it is essential that the simplified calculation does not provide too optimistic results, which would lead to a false safety. Thus, we studied the accuracy of CastorDigue in various situations to evaluate if the safety margin is enough.

In the paper, first, we detail the structure of the software and the methods used in the various components insisting on the calculation of the breach discharge hydrograph and the propagation in the near field downstream the breach. Second, we compare propagation results in the reference situations for which CastorDigue was built; these situations are idealized cases built to compare results of various types of numerical calculations. Finally, the application to a laboratory experiment is shown to illustrate the limitations of the method.

2 PRINCIPLES OF THE SIMPLIFIED METHOD

2.1 *General organization*

The calculation method couples four modules, every one representing the processes in one specific area. To simplify, it is supposed that one breach only will appear on one reach during one flood event.

The first module describes the flow inside the main channel that can be represented in a classical 1-D way, I.E. distance along the flow axis and a series of cross sections. The input to this module will be the discharge hydrograph at the upstream end of the main channel. If observations or a previous calculation do not provide the relations between discharge and water level in the cross section of the channel, the Manning equation is used to obtain water level.

The second module (details in §2.2) describes the breaching process and provides the discharge hydrograph at the breach site.

The third module (details in §2.3) describes the propagation immediately downstream from the breach. In this "near field", the flow will expand in all the directions and cannot be considered as one-dimensional. The inputs are the discharge hydrograph at breach site and the parameters of the "near field": the friction coefficient and the slope of the plane on which water is supposed to propagate.

The fourth module describes the propagation in the far field. Far enough from the breach, the flow is supposed to follow the main topographical features and thus becomes one-dimensional again. The inputs are the cross sections of the flood plain and the discharge hydrograph at breach site. The outputs are the peak water level, discharge, velocity and the time of peak in any of the cross section of the flood plain. The same method as in Castor software (Paquier and Robin (1995), Paquier and Robin (1997)) is used: the peak discharge is reduced along the valley by a coefficient depending on the slope, the friction coefficient and a non-dimensional distance to the breach; then the Manning equation is used to obtain water depth and finally peak velocity and time of wave arrival are calculated by elementary equations. The non-dimensional distance to the breach is calculated from the water level above the ground level downstream from the breach and the volume of the discharge hydrograph at the breach site.

2.2 *Breach discharge hydrograph calculation*

Two kinds of erosion processes are supported: piping and overflow. A simplified calculation of flow and erosion provides the discharge hydrograph though the breach. The discharge crossing the levee is calculated as the sum of the flow over the levee and the flow through the breach. The first term is calculated from a weir-type equation in which the elevation of the weir crest is the one of the levee crest. Generally, this discharge is equal to zero because the water elevation in the main channel remains below the crest of the levee; a general overflow of the levees cannot be considered because only one inlet into the floodplain is considered. The second term results from a simplified calculation of the breach erosion. The levee is guessed a trapezoidal shape embankment built using one single material that can be described mainly by grain diameter and porosity. The breach is summed up by one single cross section, the shape of which is circular for beginning of piping and rectangular for the other cases. The flow is computed from the Manning equation considering the slope between the water level in the main channel upstream from the breach and the critical depth downstream from the breach (or a higher water level if the calculation of water depth in the "near field" provides it). At every time step, the eroded volume is calculated from the sediment discharge obtained using the Meyer-Peter and Müller (1948) equation. The initiation of the breach comes from either the opening of a small pipe inside the levee or from the lowering of part of the crest of the levee (initial rectangular breach on the top of the levee). This type of simplified evolution of the breach was previously developed in the software Rupro by Cemagref, see Paquier (2001). An extensive validation of the method against results from experiments and results from calculations using other methods was performed during CADAM and IMPACT European projects as reported in Paquier and Recking (2004). Further improvements described in Paquier (2007) lead to an accurate calculation of both peak discharge and final breach width (error below 30%).

Although the method was developed and used for dam breaching, it can be easily extended to levee breaching considering that a lot of the software validation tests were performed using an upstream water elevation that evolves slowly. Thus, the uncertainty of the results of this calculation can be estimated from the previous studies. Paquier and Recking (2004) summing up the studies performed during Impact project indicate that an error in the estimate of the parameters of the model can lead to an error less than 30 % for peak discharge (interval of 90% confidence); thus summing up this latter error and the error of the method, we can estimate that peak discharge is situated between 60 % and 170 % of the calculated peak discharge. Then, the software user to determine the range of uncertainty in the propagation downstream can enter this range of value.

2.3 *Equations for propagation in the near field*

The equations are built on non-dimensional variables (noted by *) such as:

$$h^* = \frac{h}{h_0} \quad \text{or} \quad t^* = \frac{t - t_0}{\sqrt{h_0 / g}} \tag{1}$$

which means that the average upstream water depth upstream h_0 during the period starting from the breaching time t_0 is the basis for this non - dimensioning (notations: h water depth, t time, g gravity acceleration).

The flow is shared between a front zone and a backward zone. The velocity of the front V_f in the direction of the slope is determined by equation (2) in which C and η are coefficients calculated using respectively equations (3) and (4), n is the Manning coefficient, B the breach width, I is the slope of the plane surface considered for propagation, a is the distance between the breach and the front calculated using numerical integration of V_f.

$$V_f^*(t) = C\eta n^{*-0,4} B^{*0,2} h^{*0,5} t^{*(\eta-1)} \tag{2}$$

$$\begin{aligned} C &= 1{,}2/(7\eta - 2{,}5) \quad \text{if} \quad I \geq 0 \\ C &= 1{,}2/(0{,}2 + 3{,}2\eta^2) \quad \text{if} \quad I \leq 0 \end{aligned} \tag{3}$$

$$\begin{aligned} \eta &= 1 - 0{,}5/(1 + 25\, I^{0,7}) \quad \text{if} \quad I \geq 0 \\ \eta &= 0{,}5 \exp\,(-200\, I^2 a^* / h^*) \quad \text{if} \quad I \leq 0 \end{aligned} \tag{4}$$

Empirical equation (2) was obtained fitting the coefficients on about 50 reference cases described by Monier (2009), similar to the ones presented in §3.

To calculate the water depth at a distance x from the breach along the main slope direction, equation (5) from Whitham (1955) is used in the front zone and, near the breach, equation (6) in which exponent 2 comes from Ritter (1892). As soon as the value obtained by equation (5) is up the value of equation (6), this latter value is kept.

$$h^* = \left(\frac{7}{3} V_f^{*2}\, n^{*2}\, (a^* - x^*)\right)^{3/7} \tag{5}$$

$$h^* = h_b^* \frac{(a^* - x^*)^2}{a^{*2}} \tag{6}$$

To calculate the flow velocity v, simple empirical equation (7) is used, coefficient β being calculated by equation (8) and subscript b referring to the breach location.

$$v^* = V_f^* + \frac{(a^* - x^*)^\beta}{(a^*)^\beta}\left(v_b^* - V_f^*\right) \tag{7}$$

$$\beta = 0.148\, t^{*\,0,46} \tag{8}$$

Then, to obtain the flow parameters all over the surface, we consider that the lines of same values respectively of water depth and velocity are ellipses of which the main axis is in the direction of the slope. On the same set of reference cases, we fitted the ratio Γ between the lengths of the two axes (equation (9)).

$$\begin{aligned} \Gamma &= 1 + 50I + 0.00732\, n^{*-0,85} I\, t^* \quad \text{if} \quad I \geq 0 \\ \Gamma &= \exp(-0.214\, |I|^{0,43} t^*) \quad \text{if} \quad I \leq 0 \end{aligned} \tag{9}$$

The set of equations presented here above permits to obtain the water depth and velocity at any time step in any point of the plane surface. Then, at any point, CastorDigue calculates the higher water depth and the higher velocity. Only, these latter values and the time of the wave arrival are further validated because they are the main hazard parameters to be used in a risk analysis. In the present stage of development of CastorDigue, only slope in the direction normal to the levee is considered although the above method can be applied for any slope direction.

3 REFERENCE CASES

3.1 *Presentation*

The reference cases are simplified cases that are thought to permit comparison between various numerical models. They are built on the same schematization of the river, levee and flood plain. Only the dimensions are changed from one case to another case. Here below, we consider only changes in the slope of flood plain (near field).

The river is a straight symmetrical trapezoidal channel of which the bottom is 90 m wide and the top 100 m wide at the levee crest elevation 10 m up. The levee lies along the river and the floodplain is 1500 m long in the river direction and 2700 m long in the normal direction. The levee foot in the floodplain side is 5 m down and it is considered that the breach develops exactly down to this elevation. The crest of the levee is 6 m wide and the downstream slope of the levee is 100%. The slope of the river is 0.1 %, the discharge is constant at 2800 m^3/s and the breach width 13.5 m. Before breaching, the water elevation above the flood plain at the foot of the levee is 3.25 m. Breaching is considered as instantaneous in order to simplify the question of validating the propagation in the near field. The friction in the floodplain corresponds to a Manning coefficient of 0.04.

First case corresponds to zero slope in the flood plain. Later, we consider various slopes of the flood plain in the direction normal to the levee.

The reference calculation are carried out using Rubar 20 that solves 2D shallow water equations and includes a breaching module similar to the one included in CastorDigue. Here below, the calculations using Rubar 20 starts from a steady state in the river without breach and instantaneously, a part of the levee is removed. Because Rubar 20 uses an explicit Godunov type scheme as described in Mignot et al. (2006), such an instantaneous failure can be quite well reproduced as shown in Paquier (2001), which explains that such a calculation can be used as a reference. The grid is constituted of rectangles about 13 m long in the river direction and 1 m to 20 m wide depending of the distance from the breach. A control with denser grid in Beraud (2009) proved that there is no bias in the results due to the grid, particularly near the breach.

3.2 *Comparisons for zero slope*

Comparison takes place at 2400 s after the breach opens. Figures 1 to 3 shows the results along the direction normal to the levee. For CastorDigue, ten points equally spaced are shown. For Rubar 20, all the calculation cells are used but results are stored using a quite long time step.

Some discrepancies can be observed:

- The wave arrival time is slightly (about 10%) overestimated using CastorDigue for the long distances (for which the 2-D calculation uses large cells, which explains the steps in the curve);

- The peak water depth is overestimated at breach site while velocity is strongly underestimated at the same location. Both problems that are linked come from the estimate of the breach parameters; in Castordigue, a progressive failure is modeled while 2-D calculation was set with instantaneous failure. A progressive failure in the 2-D model will reduce the deviation but this problem stresses the difficulty to control the dynamics of the breaching process;

- Far from the breach (more than 100 m), overestimate of peak water depth is reduced to less than 20% while peak velocity becomes overestimated.

Although, this case is part of the set of reference cases on which the coefficients of CastorDigue were calibrated, the error linked to the use of CastorDigue can be assessed to about 20 %.

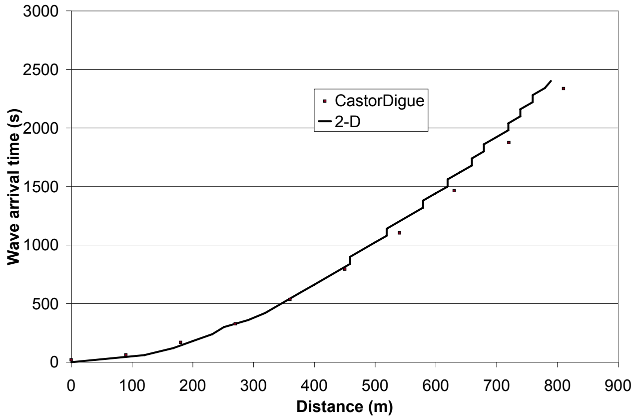

Figure 1. Wave arrival time in the direction normal to the levee.

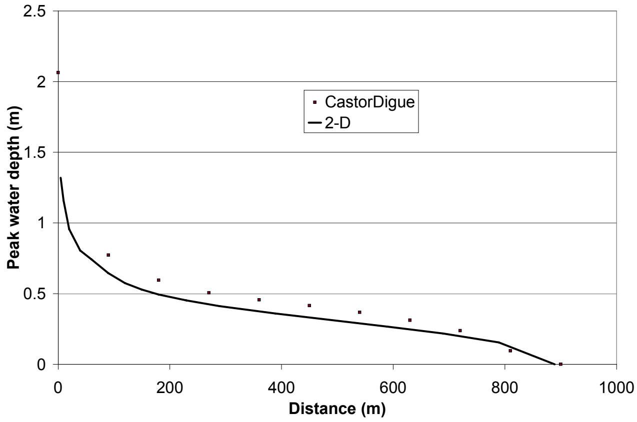

Figure 2. Peak water depth in the direction normal to the levee.

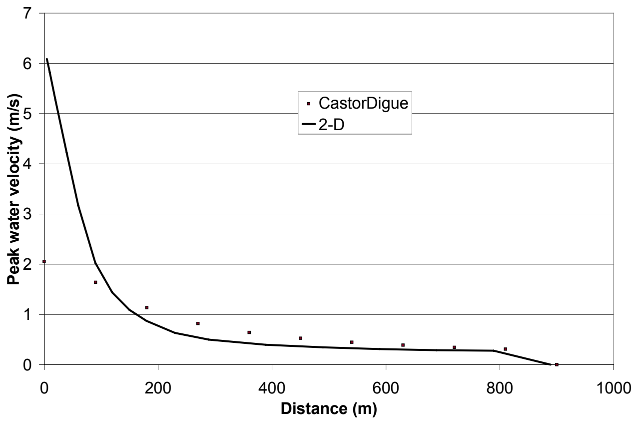

Figure 3. Peak water velocity in the direction normal to the levee.

If there is some discrepancy along the direction normal to the levee, it is expected a higher deviation else where for which the calculation results are inferred from the results along the normal direction. A first insight of this question is shown on figure 4. For zero slope, the shape of the flooded area at any time is half a circle in CastorDigue modeling. The 2-D calculation provides a slightly different scheme because the velocities in the river and in the breach create some non-symmetrical features; although, the flooded surface is similar, the overlap is not complete because in the river flow direction (marked by the arrow in figure 4), propagation is faster. In the direction showing higher error, a 10% additional error can be estimated. Similar results are also observed for the other variables but differences can be even higher very close to the levee because the propagation is slowed down by the levee itself; however, in such cases, CastorDigue rises and accelerates the flow, which means that it provides pessimistic results.

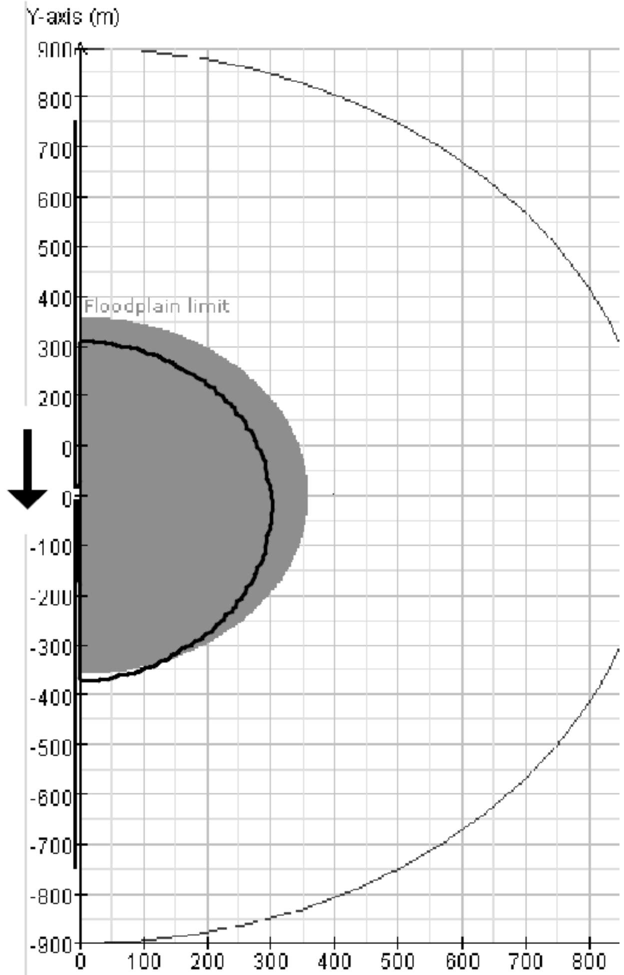

Figure 4. Flooded area at t=540 s. Grey area is result of CastorDigue (of which calculation was limited to 900 m), black line limit according to 2-D calculation.

3.3 *Comparisons for non-zero slope*

Same comparisons as for zero slope can be performed with various slopes. In case of negative slope, rapidly, the wave propagation is limited by gravity and CastorDigue was fitted to this situation.

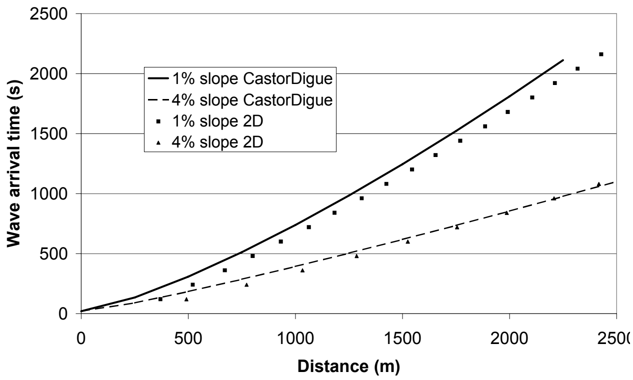

Figure 5. Wave arrival time in the direction normal to the levee.

More interesting is the case of strong positive slopes in which the flow can accelerate because of the slope. Figure 5 shows that CastorDigue reproduces this process in a quite suitable way with an error limited to about 10% in any case.

Finally, a more extensive comparison with the same basis leads to errors up to about 30 to 40% on any of the three main variables for the 50 reference cases. However, because such cases were used for coefficients calibration, it was necessary to use other references. Because field measurements were scarce and uncertain, laboratory measurements were preferred; they have also the advantage to integrate a scale change because the water head upstream the levee is generally much lower.

4 LABORATORY EXPERIMENT

4.1 *Presentation of the experiment*

The experiment selected for this paper was performed in Korea and described in Yoon (2007) and Yoon and Lee (2007). A 5 m wide and 30 m long rectangular channel (acting as a reservoir) was separated (by a wall) to a flat flood plain 28 m long and 24 m wide (with opened boundary). The breach that was in the middle of the side of the floodplain was 1 m wide (for the case considered) and the gate that closed the rectangular opening can be opened in about 5 seconds. The water depth in the channel was 0.5 m over the flood plain ground level before the gate opening. Manning coefficient was about 0.012. Gauges located in the flood plain measured the water depths along time.

4.2 *Comparison with experimental data*

The experimental measurements are compared to CastorDigue results and to results obtained by 2-D calculation. The latter modeling used Rubar 20 software on a grid made of rectangular cells with an average space step of 0.15 m (denser grid near the breach).

Figure 6 shows that CastorDigue overestimates the wave arrival time far from the breach and above all when compared to 2-D model. Because the main process is the emptying of the reservoir upstream from the breach, this discrepancy leads to a too high water depth at one time and also to a too high peak water depth. However, Figure 7 shows that the discrepancy on peak water depth is very high, which means that there is another cause.

We guess that the error is amplified by the equations (5) and (6) we used to calculate water depth. We selected equation (6) because it permitted to obtain a "safe" result but in some cases and particularly on experimental cases, it seems to provide a very high overestimate. Compared to real cases, this overestimate may be not so high because some sediments coming from the erosion of the levee can deposit and raise water level.

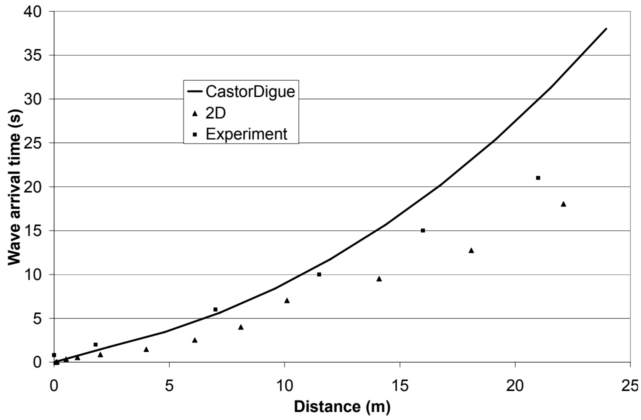

Figure 6. Wave arrival time in the direction normal to the channel.

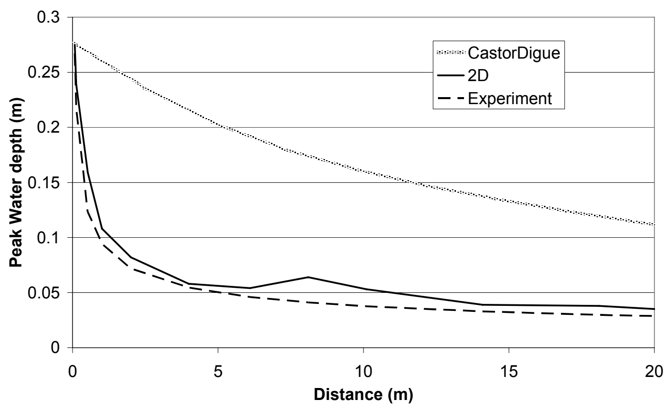

Figure 7. Peak water depths in the direction normal to the channel.

5 CONCLUSIONS

The development of the simplified model CastorDigue to simulate breaching of levees and wave propagation downstream leads to the development of a set of empirical equations describing the propagation in the near field of the flood plain. This set of equations and the calibrated coefficients included in CastorDigue lead to an error generally limited to about 40% on the key parameters that are peak water depth, peak velocity and wave arrival time. This error is equivalent to the one obtained on the breach discharge hydrograph using the method included in CastorDigue.

However, this result is limited to a set of reference cases selected to represent the field situations along the rivers when breaching can occur. The comparison to results of experiments at a much smaller scale permits to reveal some discrepancies; because the CastorDigue results are always too pessimistic in this latter cases, software can be used at a preliminary stage to identify if the risk exists or not; a positive answer and the presence of stakes will then lead to a more detailed hydraulic study.

Consequently, further research dedicated to near field propagation will be oriented to determining the range of use of CastorDigue I.E. the domain in which uncertainty on key results remains below 40%. A larger set of test cases will be used including as much as possible experimental and field measurements together with reference 2-D modeling.

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