



HAL
open science

Uncertainties of a 1D Hydraulic Model with Levee Breaches: The Benchmark Garonne

Nathalie Bertrand, Maxime Liquet, Denis Moiriat, Lise Bardet, Claire-Marie Duluc

► **To cite this version:**

Nathalie Bertrand, Maxime Liquet, Denis Moiriat, Lise Bardet, Claire-Marie Duluc. Uncertainties of a 1D Hydraulic Model with Levee Breaches: The Benchmark Garonne. SimHydro 2017 - Choosing The Right Model in Applied Hydraulics, Jun 2017, Nice, France. pp.189-204, 10.1007/978-981-10-7218-5_13. irsn-04215779

HAL Id: irsn-04215779

<https://irsn.hal.science/irsn-04215779>

Submitted on 10 Oct 2023

HAL is a multi-disciplinary open access archive for the deposit and dissemination of scientific research documents, whether they are published or not. The documents may come from teaching and research institutions in France or abroad, or from public or private research centers.

L'archive ouverte pluridisciplinaire **HAL**, est destinée au dépôt et à la diffusion de documents scientifiques de niveau recherche, publiés ou non, émanant des établissements d'enseignement et de recherche français ou étrangers, des laboratoires publics ou privés.



Distributed under a Creative Commons Attribution - NonCommercial - NoDerivatives 4.0 International License

UNCERTAINTIES OF A 1D HYDRAULIC MODEL WITH LEVEE BREACHES: THE BENCHMARK GARONNE

Nathalie Bertrand¹

Institut de Radioprotection et de Sûreté Nucléaire, PRP-DGE / SCAN / BEHRIG, Fontenay aux Roses,
France
nathalie.bertrand@irsn.fr

Maxime Liquet

GFZ German Research Centre for Geosciences, Section 5.4: Hydrology, Telegrafenberg, 14473 Potsdam
Germany
maximel@gfz-potsdam.de

Denis Moiriat, Lise Bardet, Claire-Marie Duluc

Institut de Radioprotection et de Sûreté Nucléaire, PRP-DGE / SCAN / BEHRIG, Fontenay aux Roses,
France
denis.moiriat@irsn.fr, lise.bardet@irsn.fr, claire-marie.duluc@irsn.fr

KEY WORDS

Levee breach, flood modelling, sensitivity analysis.

ABSTRACT

In a fluvial environment, the main role of levees is to canalize water downstream of rivers and to reduce the risk of flooding to nearby areas. Levee failure can be either structural or hydraulic. Structural failure occurs where a breach in a flood defence system leads to the inundation of the protected area whereas hydraulic failure refers to flooding before the designed protection level is attained and without prior damage to the flood defence system. Nowadays, hydrodynamic modelling codes are able to perform hydraulic failure such as overflowing by means of any appropriate weir equation, however, only a few allow to simulate structural failure. HEC-RAS can do both and enables to model levee breaches with a simple but flexible parametric module. The aim of our study is to evaluate the capacity of a 1D hydraulic model to represent levees breaches and subsequent flooding. To do so, a 1D storage area model is built with HEC-RAS and calibrated using data provided by the “Benchmark Garonne” project initiated by EDF. The study case is based on the 1981 historical flood event of the Garonne River between Tonneins and La Réole (section 2). The model is introduced and compared to two other hydraulic models used in the benchmark (section 3). Two sensitivity analyses with respect to sets of hydraulic parameters and levee breach parameters are carried out (section 4). Results expressed as maximum water levels show that the main channel roughness coefficient and the final breach width are the most influencing model parameters, respectively. Levee breaches appear to be a non-negligible source of uncertainty in hydraulic modelling, comparable to uncertainties arising from model structure or model calibration. In order to improve our modelling approach, a ground survey and a literature survey is conducted to collect data about the breaches that occurred in the study area, in particular during the 1981 flood (section 5). Historical evidence shows that a significant number of breaches occurred since 1875.

1. INTRODUCTION

In a fluvial environment, the main role of levees is to canalize water downstream of rivers and to reduce the risk of flooding to nearby areas. Levees can be altered by slow and rapid damaging processes during their life, such as erosion, scour and slippage [1]. The breach initiation and its progression into the levee result from a sequence of physical mechanisms, such as piping, land-side erosion due to overflowing or slope instability, which decrease the resistance of the levee towards the hydrological load and can lead to a full levee break. Levee failure can be either structural or hydraulic failure. Structural failure occurs where a

¹ Corresponding author: Nathalie Bertrand (nathalie.bertrand@irsn.fr)

breach in a flood defence system leads to the inundation of the protected area whereas hydraulic failure refers to flooding before the designed protection level is attained and without prior damage to the flood defence system. Structural failure can induce hydraulic failure and vice versa. Therefore, breaching processes involve strong interactions between levee materials and flow conditions, leading to complex phenomena to be understood. As a result, it is difficult to predict the exact locations of levee breaches and to identify the triggering factors of the breach's initiations.

Nowadays, hydrodynamic modelling codes are able to perform hydraulic failure such as overflowing by means of any appropriate weir equation. However, only a few incorporate a structural failure module. Three types of main breach models have been developed by researchers and engineers: empirical, semi-physically based and physically based model [2]. Empirical models are methods usually based upon data collected from a series of documented breach events. The main advantage of this model is that it is rather simple. However, this is also one of their main weaknesses, as there can be highly uncertainty within the predictions. Semi-physically based model improves the prediction capability by adding some of the physical process or processes involved in the failure without being too demanding regarding computation procedure. Physically based numerical models simulate the failure of embankments based on the processes observed during failure, such as the flow regimes, erosion and instability processes. In the last four decades many models have been developed to simulate the failure of embankments. These models differ in their complexity, assumptions involved and techniques used.

The three different types of models presented show that there is a large number of methods and tools available for practitioners to model the development of a breach. Thus, whatever the approach chosen, the assessment of the uncertainty associated with the results obtained, such as the peak flow, is required [3]. Therefore, recent studies assessed the uncertainties associated with breach models. For example, [4] analysed the dispersion of the water level-discharge relationship according to the variation of hydraulic parameters such as breach width, duration and time. Their study concluded that uncertainty propagation can help to identify and quantify results according to the variation of one or more parameters at once. Other studies evaluated the uncertainties of predicted peak flows and water levels downstream from breaches levees [5, 6].

The choice between one of the models introduced depends mostly on the study purpose, data availability, and computational power. Due to the interactions between the floodplain and the river in a fluvial context, non-physically based models are usually not suitable for modelling dike breach. Physically based models are interesting tools however they may be expensive in data collection, model development and computational time. Semi-physically based models, owing to their flexibility and limited complexity, seem to be preferred today. An example of such models is the one integrated in HEC-RAS [7, 8]. It gives the possibility to model a breach triggered either by piping or overflowing. A set of parameters has to be defined a priori, such as the triggering water level or the weir coefficient, then, the breach develops according to these parameters and the basic underlying equations, i.e. a weir equation for overflowing and an orifice equation for piping. In [9] for example, HEC-RAS is used for 1D flood wave routing on the river Reno in Italy and to account for the uncertainty associated with the location and characteristics of a potential levee breach, varying the depth of the breach, the time of formation and the width of the breach.

The aim of our study is to compare the performance of different hydraulic models and evaluate the capacity of a 1D hydraulic model to represent levee breaches and the associated flooding. To do so, a 1D storage area model was built with HEC-RAS and calibrated using data provided by the "Benchmark Garonne" project initiated by "Electricité de France" (EDF). The case study is based on the 1981 historical flood event of the Garonne River between Tonneins and La Réole (section 2). Then, the model is introduced and compared to two other hydraulic models used in the benchmark (section 3). Two sensitivity analyses with respect to hydraulic parameters and levee breach parameters have been carried out and will be presented in section 4. Section 5 introduces perspectives to enhance our levee breach modelling approach.

2. CASE STUDY

The study area is a 50 km long reach on the Garonne river between Tonneins and La Réole (Figure 1). The area has been settled to protect floodplains by organizing flooding and flood storage, between 1760 and 1850, where many earthen levees were built to protect the harvest against spring floods [10, 11]. The river

was canalized to protect local resident against flooding after the historical 1875 flood event [10]. Therefore, this segment of the Garonne river is characterised by successive storage areas that control flooding in the floodplain. IRSN has been taking part of the “Benchmark Garonne” project induced by EDF. The aim of the project is the uncertainty quantification in hydraulic modelling on a stretch of the Garonne. Earlier works, carried out by Besnard and Goutal [12] and Bozzi et al. [13], have investigated discharge and roughness uncertainty in 1D and 2D hydraulic models. In order to contribute to the project, a version of MASCARET 1D as well as hydraulic and hydrological data required to build a model were provided to us. Within [12], models’ capacities to represent a major flood event were compared. The three following models were used: two 1D models (MASCARET) with different ways to represent the floodplain areas with or without storage areas and one 2D model (TELEMAC). The comparison mainly highlights a convergence between the 1D storage area model and the 2D model.

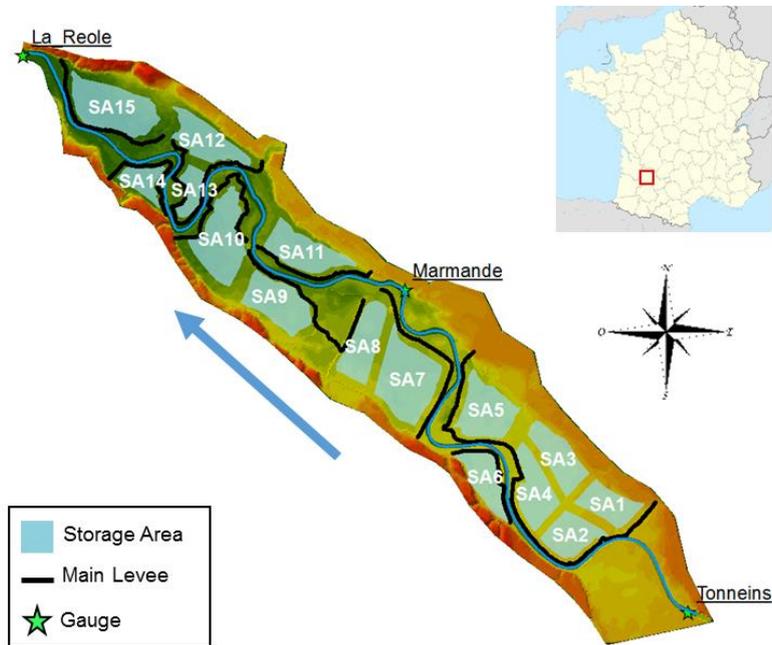


Figure 1: Map of the study area on the Garonne river including the location of the modelled storage areas and the main levees.

Successive storage areas give a particular configuration to the Garonne profiles. Figure 2 shows three flow characteristics on a typical cross-section of the Garonne to illustrate the flooding sequence: 1) base flow 1,100 m³/s, 2) bankful flow 2,400 m³/s and 3) flow before levees with the lowest protection level are overtopped 3,500 m³/s.

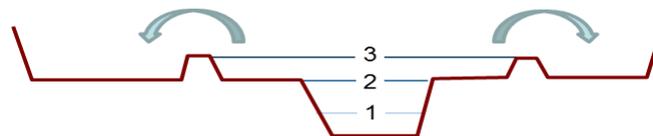


Figure 2: Typical cross-section of the Garonne between Tonneins and La Réole.

Flooding of the less protected areas between Tonneins and La Réole occurs for low return period, i.e. from the 10-year flood approximately. Only a few levees have a standard of protection higher than the 30-year flood. Due to a flat topography and the presence of steep floodplain lateral slope, the floodplains are largely inundated even for floods with high probability. December 1981 flood is one of the largest floods that occurred since the most severe recorded flood (1875), this flood event has been used as a reference for our study. During this 9-day event, the peak discharge measured at Tonneins reached 6,040 m³/s, corresponding approximately to a 20-year flood, and the floodplains were fully inundated. More than 25 levee breaches occurred within the study area, with lengths ranging from 10m to 100m [11].

3. HYDRAULIC MODELLING

3.1 Model Choice: HEC-RAS 1D

Although not an open-source code, we chose to use HEC-RAS in this study as it is commonly used (see e.g. [14]), well documented [15] and integrates a levee breach module. Furthermore, an Application Program Interface allows to remotely control HEC-RAS via another application [16], which is useful for conducting simulations when a high number of calculations are required (see section 5.2.). HEC-RAS solves the 1D Saint-Venant equations based on the Preissmann implicit scheme. Floodplains can be modelled as storage areas (“casiers” in French), linked to each other and to the reach with lateral structures (e.g. weirs, culverts, etc.). Both primary levees located along a river and secondary levees further protecting the floodplain can be represented as weirs, which is common practice.

Regarding levee breach modelling, the available approach is physically simplified. Erosion processes are not calculated in the code, instead, the user has to predefine the failure mode (overflowing or piping), and all the breach characteristics (e.g. final width, growth rate and duration), which are then used to calculate the breach outflow with a weir formula. Breach initiation can be set at a specific time, or conditioned to the hydraulic load by means of two available criteria, either for a given water level or for a given water level combined with a given duration above this level, checked dynamically during the simulation. Hence, this levee breach module is simple but also flexible, allowing various breach configurations to be simulated.

3.2 Model Setup

The HEC-RAS 1D model in this study is based on a MASCARET 1D model developed previously by EDF [12]. The main channel is defined by 83 cross-sections obtained from a bathymetric survey, further interpolated at an appropriate spatial spacing. Floodplain topography is represented by 15 storage areas whose stage-volume relationships are derived from photogrammetry data (downstream part of the study area) and national topographic map (upstream part). In the model 57 weirs, with lengths ranging from 300m to 1700m, are used to represent levees located along the river and in between storage areas. Levees are modelled as broad-crested weirs. This assumption holds if $C > 2H/3$ [17], where C is the levee crest width (m) and H the hydraulic head (m), which is the case when overflowing occurs on the levees in the study area. Weir flow coefficient values are based on expert judgments. The boundary conditions are represented by: i) a downstream rating curve established at La Réole, ii) the 1981 flood hydrograph measured at Tonneins.

3.3 Calibration

The model has not been calibrated against observations as it is usually the practice but calibrated through comparison with results obtained from previous simulations performed with MASCARET, owing to the benchmark procedure. A flood event untitled “17.2” in the project was used for the calibration of the HEC-RAS model, along with steady state flow data. The “17.2” event is characterised by a peak discharge value of 3,620 m³/s, therefore leading to limited flooding of the protected area (see Figure 2). Main channel and floodplain Strickler’s coefficients were used as “calibration” parameters. It should be noted that HEC-RAS requires Manning’s n values as input parameters but we decided to present our results using Strickler’s values K_s ($K_s = \frac{1}{n}$). Here, the floodplain coefficient applies only to the floodplain located between the river and the levees (see Figure 2); beyond no roughness coefficients are required as storage areas are used to model the flow characteristics. For each parameter, a single value was “calibrated” over the whole study area as both river and floodplain properties were deemed homogenous. The resulting values for the main channel and floodplain Strickler’s coefficient are 40 and 20, respectively. Compared to MASCARET results, the bias is equal to 2 cm and the mean absolute value to 6 cm.

Once the “calibration” performed, a comparison between models on another flood event has been performed as part of the benchmark.

3.4 Comparison With MASCARET And TELEMAT

Maximum water surface elevations calculated for the 1981 flood event in the main channel have been compared to elevations obtained with MASCARET 1D and TELEMAT 2D in a previous study [12]. Although levee breaches occurred during this flood, they are not considered for the sake of model comparison. The results shown in Figure 3 are similar, however, the elevations obtained with HEC-RAS

appear to be generally lower than MASCARET and TELEMAC with a mean bias error calculated over the whole reach of -0.32m and -0.17m, respectively.

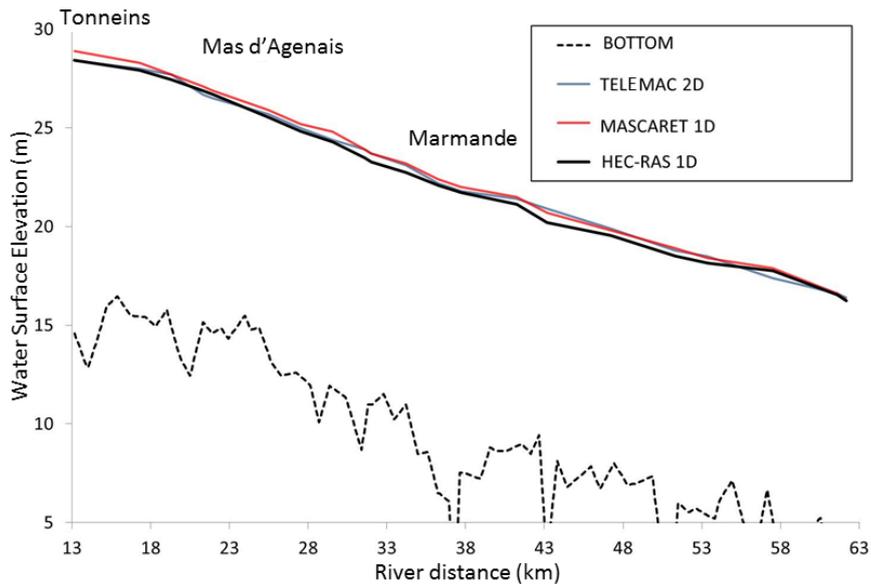


Figure 3: Maximum water surface elevation during 1981 flood with HEC-RAS along the study area compared with EDF results.

As a result of this difference in maximum water surface elevations obtained in the models, maximum water levels calculated with HEC-RAS in the protected floodplain are expected to be lower too. In the 15 storage areas, it is found that the bias between HEC-RAS and MASCARET is around -0.50m (Figure 4). Note that no inflow to the storage area 6 was simulated with HEC-RAS as it is particularly sensitive to the water levels calculated in the river.

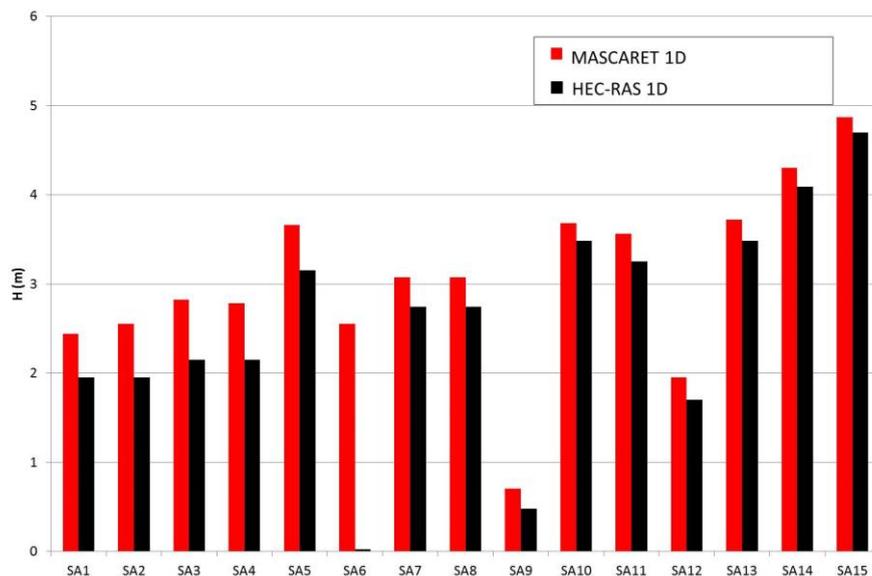


Figure 4: Comparison of the maximum water depth modelled in each storage area between HEC-RAS (in black) and MASCARET (in red) for the 1981 flood.

When a river bursts its banks, differences in flow velocity between the main channel and the floodplain result in complex 3D flow at the interface, which should be taken into account to appropriately calculate discharge conveyance. HEC-RAS uses the Divided Channel Method (DCM) in which cross-sections are divided into vertical sub-sections that do not exchange any lateral momentum. MASCARET uses the Debord

Method (DM) which is an empirical correction of DCM that represents turbulent transfers. [18] compared these methods in a flume experiment and showed that the DCM and the DM both overestimate discharge capacities, prediction errors being greater for the former. Therefore, the DCM tends to generally underestimate simulated water depths, which can explain the differences observed between HEC-RAS and MASCARET for the 1981 flood event. Even though HEC-RAS and MASCARET are two common hydraulic models, the model inter-comparison highlights that differences in model structure, i.e. compound channel flow modelling, lead to significant uncertainty in model results.

4. SENSITIVITY ANALYSIS

Two local sensitivity analyses have been carried out separately by varying the parameters one-at-a-time to identify the influence of, first, a set of 4 hydraulic parameters and, second, a set of 5 levee breach parameters.

4.1. Hydraulic Parameters

Regarding the hydraulic parameters, the influence of roughness coefficients in the main channel and the floodplain, and of weir coefficients have been evaluated on maximum water depths within the main channel and the storage areas by varying them in the whole study area. For each parameter, a large range of values determined by expert judgement and consistent with the physical values usually observed for this kind of river reach is explored to quantify their influences. Strickler's coefficients have been changed by $\pm 5 \text{ m}^{1/3} \cdot \text{s}^{-1}$ and $\pm 10 \%$ compared to their calibrated values and weir coefficients by $\pm 5 \%$ and $\pm 10 \%$ compared to their nominal values. In addition, the influence of inflow hydrograph volume has been assessed by varying it by $\pm 10 \%$ and $\pm 28 \%$, the latter being derived from the 90 % confidence interval of flood volume distribution at Marmande (located between Tonneins and La Réole) for a 20-year flood. Note that levee breaches are not considered in this sensitivity analysis.

As all the parameters have been varied by $\pm 10 \%$, it is possible to compare them directly. The root mean square error (RMSE) values for the reach and storage areas reveal that the most influencing parameter is the main channel roughness coefficient (0.245m at +10 % value in the main reach), followed by the floodplain roughness coefficient (0.027m), the hydrograph volume (0.016m) and the weir coefficient (0.009m). The influence of the main channel roughness coefficient is emphasized by the mean bias error values calculated in the storage areas for the range $\pm 14 \%$, i.e. for Strickler's coefficient values of 35 and 45 $\text{m}^{1/3} \cdot \text{s}^{-1}$, of 0.532m and -0.902m, respectively.

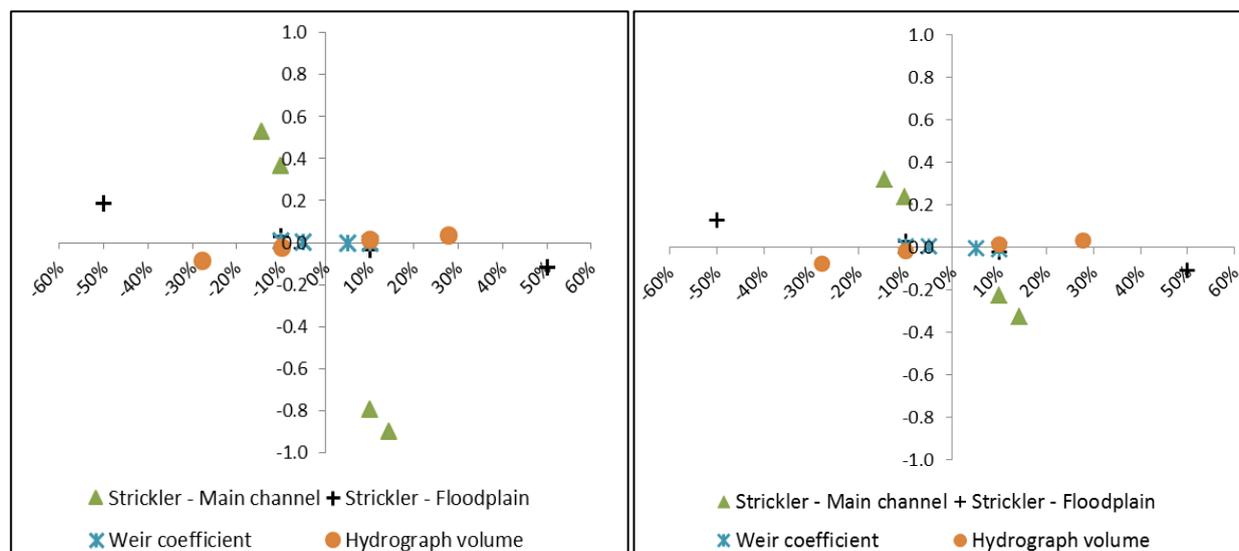


Figure 5: Mean bias error of the maximum water depths (in meter) calculated in all the storage areas (left) and in all the cross-sections of the reach (right), with respect to the values of the sensitive parameters expressed relatively to their nominal value.

Figure 5, in which parameters are shown in relative values and model sensitivity is expressed as the mean bias error, illustrates the way selected hydraulic parameters influence the maximum water depths in the storage areas (left panel) and in the reach (right panel). Either increasing Strickler's coefficient values, i.e. decreasing roughness, or increasing weir coefficient values lead to lower water depths calculated, while the opposite holds for hydrograph volume values. In the storage areas, the mean bias errors calculated for the main channel roughness coefficient exhibit a non-linear behaviour of the model response. Indeed, flooding of the protected floodplain is more likely to occur when the main channel roughness is high (i.e. for low Strickler's coefficient), highlighting the presence of a threshold process controlled by levee crest levels in the study area. Moreover, when levee overflowing occurs, storage area inflow is determined by a weir formula that is a non-linear equation giving the discharge with respect to the hydraulic head.

4.2. Levee Breach Parameters

A second local sensitivity analysis has been conducted to assess the influence of 5 breach parameters introduced in Table 1. In this study two main assumptions have been made. First, only levee overflowing triggered by a given water level (dynamically during the simulation) has been selected as a possible failure mode. Second, breach parameters vary equally in the whole domain. For example, if 5 levees are triggered at different locations and different times they will have the same breaching characteristics. The triggering water depth corresponds to the water depth above the crest at which a breach would be initiated during the simulation. Then, the breach develops gradually according to the formation time, both horizontally to the final width and vertically to the final depth, described here with a crest lowering parameter whose 100%-value corresponds to a breach deepened to the local ground level. The value and ranges of the parameters have been chosen from available literature or based on expert judgement, however, uncertainty remains in the selected values.

Breach parameters	Values
Triggering water depth above levee crest (m)	[0, 0.1, 0.2 , 0.3, 0.5, 0.7, 1, 1.5]
Final width (m)	[10, 25, 50, 100 , 150, 225, 300]
Formation time (h)	[0, 1 , 2, 3, 6, 12, 24]
Crest lowering (%)	[20, 40, 60, 80, 100]
Breach weir coeff. (%)	[-20, -10, Levee weir coeff. , +10, +20]

Table 1: Levee breach parameters considered in the sensitivity analysis (values in bold are the nominal values within the model failure).

Figure 6 shows for each storage areas the minimal (green triangle) and maximal (red square) maximal water depths obtained from all the simulations, the maximum water depths obtained from simulations set with parameter nominal values with (blue bar) and without levee breach (black dot). For the first two storage areas SA01 and SA02, located upstream of the study area, significant extent values are obtained between the minimal and maximal values simulated, reaching 1.45m and 1.91m, respectively, whilst the mean extent value is 0.49m for all the storage areas. For downstream storage areas (SA07 to SA15) the results highlight that water depths in the storage areas are less sensitive to variations in breach parameters. In the Garonne floodplain, many local flood defence systems consist of levees which integrate small weirs whose crest levels are lower than the adjacent levees. These weirs are typically located downstream of the main storage areas to be protected. Therefore when a flood occurs, the storage areas are first filled by means of these weirs. In doing so, it is aimed at ensuring load relief for downstream communities. In addition, once the storage areas are inundated adverse consequences of potential levee failures, i.e. high flow velocities, are less likely to occur. In the study area, the downstream storage areas are already inundated before the water level reaches the levee crest levels, therefore, as shown in Figure 6, levee breaches have less influence there.

As can be seen on Figure 6, comparison of the sensitivity analysis results with the no-breaches results shows that in the upstream part of the study area maximum water depths may be underestimated if breaches are not considered, while overestimation is likely in the downstream part. For example, the no-breaches

simulation compared to the simulation with the nominal breaching parameters values leads to maximum water depths 1m lower in the storage area SA01 and 0.27m higher in the storage area SA15.

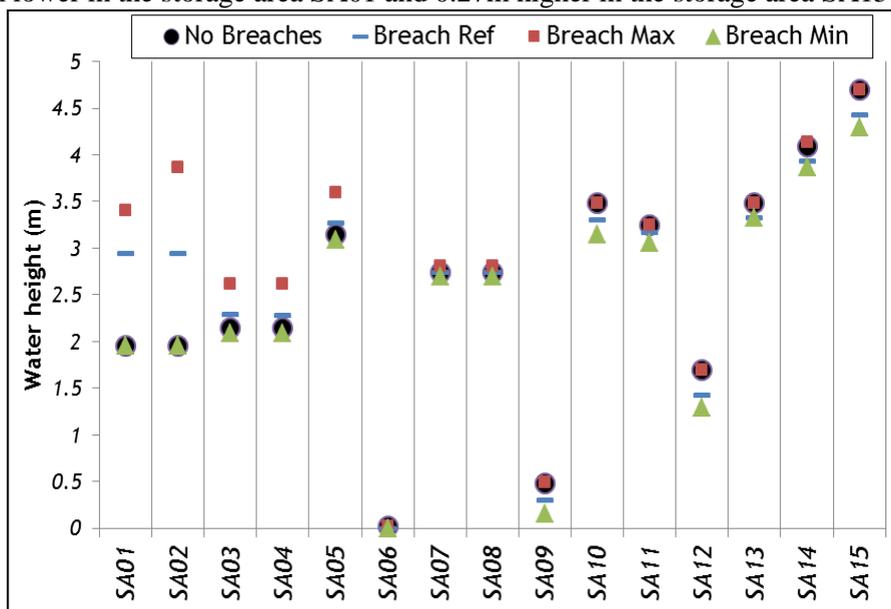


Figure 6: Maximum water depths calculated in the 15 storage areas of the model.

Regarding the influence of each parameter on the maximum water depths calculated in all the storage areas, the results show that the model is greatly sensitive to the final width, followed by the breach depth (i.e. crest lowering). To illustrate this, Figure 7 shows the maximum water depths calculated in the storage area SA01, ranging from 2.07m to 3.40m for final widths of 10m and 300m, respectively, and from 2.07m to 2.94m for 20%- and 100%-crest lowering, respectively. A large amount of water is retained when a significant breach occurs, which is the case in SA01 for large breach widths and depths, thus leading to a capped hydrograph propagating downstream of the reach. As a consequence, these two parameters have a spatially differentiated effect in the model, flood hazard is increased in the upstream part while decreased downstream. Figure 7 also shows that the breach weir coefficient and the formation time have a marginal influence in SA01, hence in the whole model area.

As for the breach triggering water depth, the model appears to be also sensitive to this parameter, although in a different way. Indeed, the results show that the number of breaches by simulation varies accordingly to the triggering water level, from 3 to 16 breaches for a triggering water depth of 1.5m and 0m, respectively. For the other parameters, the number of breaches is relatively constant as it lies between 12 and 15.

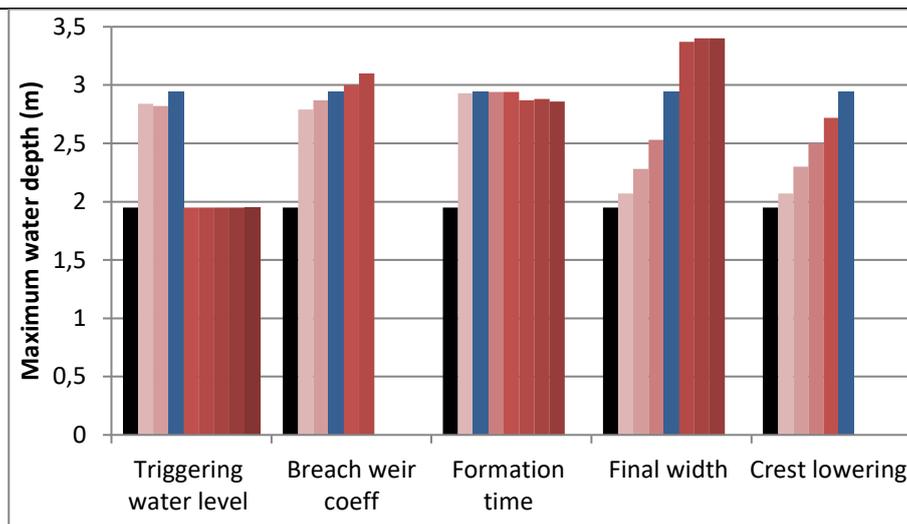


Figure 7: Maximum water depths in the storage area SA01 obtained during the breach parameters sensitivity analysis (the black bar represents the Garonne level reference without breach, the dark blue bar represents the results obtained with the nominal parameter values and the other coloured bars represent the maximum water depths obtained for each tested parameters values as they appear ordered in Table 1).

4.3. Limitations Of The Levee Breach Model

In the second sensitivity analysis, simple assumptions have been made to model levee breaches. First, only overflowing has been considered as a potential failure mode. Although it is generally reported as the most frequent failure mode, levees can also breach as a consequence of internal erosion or macro-instability [19]. Second, breach initiation is triggered by the exceedance of a single threshold, i.e. the water depth above levee crest. Hence, if overflowing occurs with a low water depth, no breach is triggered. Levee resistance with respect to overflowing could be more realistically evaluated by considering the overflowing duration as an additional threshold [6]. Third, the levee breach model in HEC-RAS is physically simplified as erosion processes are not calculated by the code itself, instead, they have to be defined by the user in advance. In fact, no geotechnical properties of the levees are taken account by the model. The last HEC-RAS version (5.0) allows the user to define widening and deepening rate curves associated with breach outflow, hence to describe erosion processes more explicitly. However, these curves are not yet given in the support manual or available in the literature. Fourth, the breaching parameters were considered as equal for all the triggered breaches. In reality this is very unlikely to occur as each breach development will be controlled by local flow characteristics and levee properties. It is difficult to model such local phenomena especially as high resolution levee data is not often available. In practice, it is possible to take into account this epistemic uncertainty in embedding it into a probabilistic framework (see e.g. [6, 20, 21]) where relevant levee properties are represented with probability density functions.

The four assumptions mentioned above highlight the need to acquire a better knowledge of historical breaches or geotechnical levee properties to improve the levee breach model proposed in this study.

5. IMPROVING LEVEE BREACH MODELLING

5.1. Historical And Geotechnical Data

A literature review and a field survey have been carried out to have a better view of the historical breach scenarios that occurred within the study area, in particular during the 1981 flood. During the field survey, four representative soil samples from different levees within the study area were studied to appreciate their erodibility and their critical shear stress, both factors being traditionally used to estimate the erosion rate of a soil subject to hydraulic stress.

Concerning historical breaches, the literature review has shown that many breaches occurred in the study area during the last two centuries [11]. In the study area the number of breaches reported in the

different available documents including grey literature is 122, 4, 10, 1, 27, 2 and 1 for the floods which occurred in 1875, 1930, 1952, 1955, 1981, 2003 and 2009, respectively. These numbers should be considered as estimates, most probably underestimating the real values, particularly those for the oldest floods. In addition, breaches are reported in 1936, 1960 and 1979, however no numbers could be found for these floods. For the breaches which could be geo-localised, it has been observed that they are mainly located close to stretches where the river conveyance decreases abruptly, as a result of floodplain width reduction, therefore leading to increased hydraulic stress on the adjacent levees [10].

In [11], 18 of the breaches which occurred during the 1981 flood were analysed in detail. The reported breach widths had a mean value of 40m and ranged from 10m to 100m, values we confirmed with aerial pictures of the 1981 flood. The authors reported a fast breach development (about a few hours) common to all the breaches, whatever the failure mode. This is in line with the constitutive materials of the levees that we have identified as silty sands according to the Unified Soil Classification System (USCS), therefore probably highly erodible. Given the data available, we identified overflowing and internal erosion as the two primary failure modes. It should be noted that these breaches developed quickly, some of them overnight. In addition, many defects were reported such as crest level settlement, trees located in or very close to levees, and the presence of burrows. All these defects resulted in decreased levee resistance and contributed to fast breach development. Therefore, the failure modes are only indicative and cannot be further specified. Incidentally, the authors noted for a few breaches the presence of erosion pits with depths up to 10m.

Although relevant data about the levees in the Garonne floodplain and their historical breaches have been collected during the literature review and the field survey, our study also highlights that little is known about the nature of backfill materials as well as the compaction standard applied to these levees, which may significantly vary for levees built decades ago such as the ones in the Garonne floodplain. These two properties may affect both breach initiation and breach development.

5.2. Global Sensitivity Analysis

As many parameters remain uncertain when it comes to model levee breaches, the assessment of uncertainty to understand the role of each input parameters is required in order to reduce and enhance model outputs analysis.

Over the last few years, IRSN has conducted several studies to evaluate the impact of uncertainties related to hydraulic model input parameters [22]. The hydrodynamic simulations are performed in a probabilistic framework which allows to describe the variance associated with the inputs, and assess the variance of the results. Such an approach is time consuming as it requires performing hundreds or thousands of calculations. Thus, IRSN has coupled several hydrodynamic codes (1D model Crue9 [23], 2D model FullSWOF_2D [24], and tsunami model Calypso) with an efficient parametric modelling software to perform uncertainty analyses. This software, Promethee, is freely distributed by IRSN (<http://promethee.irsn.org/doku.php>). Promethee allows the parameterization with any numerical code and is optimized for intensive computing. Statistical post-treatment can be directly achieved through Promethee as it integrates R statistical computing environment [25].

Promethee has recently been coupled with HEC-RAS and allows the user to vary a large number of model parameters. With respect to levee breach modelling, this coupled tool could be used to carry out global sensitivity analyses aiming at precisely determining the influence of each uncertain breach parameter on the model results. It could also be used to compare the effect of hydraulic parameters (e.g. main channel roughness coefficient) with the effect of breach parameters (e.g. final breach width). Finally, uncertainty propagation considering varying hydraulic and breach parameters could provide valuable information on the overall uncertainty associated with the results of hydraulic models when severe floods are simulated.

6. CONCLUSIONS

In this study a 1D hydraulic model has been developed with HEC-RAS. As part of the benchmark Garonne, its performance has been compared to other models against the simulation of the 1981 flood in a stretch of the Garonne River between Tonneins and La Réole. The results highlight that HEC-RAS underestimates the

maximum water levels compared to MASCARET 1D and TELEMAC 2D. Particularly, the differences between HEC-RAS and MASCARET could be explained by their distinct representation of compound channel flows, with respectively the Divided Channel Method and the Debord Method, the former being known for overestimating river conveyance.

A first local sensitivity analysis with respect to four hydraulic parameters has been carried out. The most influencing parameter was found to be the main channel roughness coefficient, followed but with significant less influence by the floodplain roughness coefficient, the hydrograph volume and the weir coefficient.

The model has been set-up to dynamically trigger levee breaches due to overflowing and a second local sensitivity analysis carried out with respect to five breach parameters. The analysis identified final breach width, followed by final breach depth, as the most influencing parameters on the maximum water levels calculated in the whole study area. Their influence was most notably observable in the upstream part of the study area, while flooding in the downstream part was controlled by what occurred upstream. This specific behaviour due to levee breaches and the resulting hydrograph capping effect is difficult to predict without any simulation. The threshold used to trigger levee breaches, i.e. water level above levee crest, influenced greatly the number of breaches initiated. Both the breach weir coefficients and the breach development time had a low effect on the results.

Since our survey on the Garonne levees showed that many breaches occurred during historical flood events, modelling levee breaches is relevant in this case. Although the levee breach approach applied in this study is rather simplistic, differences in maximum water depths with the no-breaches case are similar or greater than those obtained by comparing HEC-RAS with MASCARET or by assessing the model sensitivity to hydraulic parameters. Therefore, levee breaches appear to be a non-negligible source of uncertainty in hydraulic modelling comparable to uncertainties arising from model structure or model calibration.

Future work should benefit from the survey to improve the levee breach modelling approach by considering internal erosion as a possible failure mode and by defining realistic breach parameter ranges derived from historical data. Future work should also benefit from the coupled Promethee/HEC-RAS tool to develop a modelling structure in which the breaches triggered would not have the same parameter values, and to carry out uncertainty propagation and global sensitivity analysis.

ACKNOWLEDGEMENTS

This work could not have been carried out without the organisation of the “Benchmark Garonne” project by EDF. The authors would like to thank especially Nicole Goutal and all researchers implied in the project for data provided and rewarding technical exchanges during this project.

REFERENCES

- [1] Allsop, W. (2007). *Failure mechanisms for flood defence assets*. HR Wallingford.
- [2] Morris, M. (2009). *Breaching processes: A state of the art review*. HR Wallingford.
- [3] Wahl, T.L. (2004). Uncertainty of predictions of embankment dam breach parameters. *Journal of hydraulic engineering*, **130**(5), p. 389-397.
- [4] Alliau, D. & al. (2015). Flood study of industrial site by extreme flood risk: uncertainties with hydrologic and hydraulic extreme values. *La Houille Blanche*, (2), p. 67-74.
- [5] Mazzoleni, M. & al. (2013). Flooding hazard mapping in floodplain areas affected by piping breaches in the Po River, Italy. *Journal of Hydrologic Engineering*, **19**(4), p. 717-731.
- [6] Vorogushyn, S. & al. (2010). A new methodology for flood hazard assessment considering dike breaches. *Water Resources Research*, **46**. DOI: 10.1029/2009WR008475.
- [7] Brunner, G. (2016). *HEC-RAS, River Analysis System Hydraulic Reference Manual*. US Army Corps of Engineers - Hydrologic Engineering Center, Davis, CA, USA.
- [8] Brunner, G. (2016). *HEC-RAS, River Analysis System Applications Guide*. US Army Corps of Engineers - Hydrologic Engineering Center, Davis, CA, USA.

-
- [9] Di Baldassarre, G., Castellarin, A. & Brath, A. (2009). Analysis of the effects of levee heightening on flood propagation: example of the River Po, Italy. *Hydrological sciences journal*, **54**(6), p. 1007-1017.
- [10] SMEPAG (1989). *Monographie des crues de la Garonne (du Pont du Roy au Bec d'Ambès) - 2 tomes [Monograph of the Garonne floods (from Pont du Roy to Bec d'Ambès) - 2 volumes]*.
- [11] LPCB (1983). *Vallée de la Garonne - Enquête sur les ruptures de digues de Décembre 1981 entre Meilhan et Port Sainte Marie [Garonne Valley - Investigation on the dike breaches of December 1981 between Meilhan and Port Sainte Marie]*.
- [12] Besnard, A. & Goutal, N. (2011). Comparison between 1D and 2D models for hydraulic modeling of a floodplain: case of Garonne River. *La Houille Blanche*, (3), p. 42-47.
- [13] Bozzi, S. & al. (2015). Roughness and Discharge Uncertainty in 1D Water Level Calculations. *Environmental Modeling & Assessment*, **20**(4), p. 343-353. DOI: 10.1007/s10666-014-9430-6.
- [14] Pappenberger, F. & al. (2005). Uncertainty in the calibration of effective roughness parameters in HEC-RAS using inundation and downstream level observations. *Journal of Hydrology*, **302**(1), p. 46-69.
- [15] Brunner, G. (2016). *HEC-RAS, River Analysis System User's Manual, version 5.0*. US Army Corps of Engineers, Institute For Water Resources, Hydrologic Engineering Center, Davis, CA, USA.
- [16] Goodell, C. (2014). *Breaking the HEC-RAS Code: A User's Guide to Automating HEC-RAS*. h2ls.
- [17] CETMEF (2005) - Notice sur les déversoirs - Synthèse des lois d'écoulement au droit des seuils et déversoirs [Note on weirs - Synthesis of sill and weir flow laws]
- [18] Fernandes, J., Leal, J. & Cardoso, A. (2012). Flow structure in a compound channel with smooth and rough floodplains. *European Water*, **38**, p. 3-12.
- [19] Wu, W. (2011). Earthen Embankment Breaching. *Journal of hydraulic engineering*, **137**(12). DOI: doi:10.1061/(ASCE)HY.1943-7900.0000498.
- [20] Hall, J.W. & al. (2003). A methodology for national-scale flood risk assessment. In *Proceedings of the Institution of Civil Engineers-Water Maritime and Engineering*, p. 235-248.
- [21] De Bruijn, K., Diermanse, F. & Beckers, J. (2014). An advanced method for flood risk analysis in river deltas, applied to societal flood fatality risk in the Netherlands. *Natural Hazards and Earth System Sciences*, **14**(10), p. 2767.
- [22] Bacchi, V. & al. (2017). Feedback from uncertainties propagation research projects conducted in different hydraulic fields; outcomes for engineering projects and nuclear safety assessment. In *SimHydro 2017, 14-16 June 2017, Sophia Antipolis - Nice (France)*.
- [23] Nguyen, T. & al. (2015). Uncertainties propagations in 1D hydraulic modeling. *La Houille Blanche*, (5), p. 55-62.
- [24] Abily, M. & al. (2016). Spatial Global Sensitivity Analysis of High Resolution classified topographic data use in 2D urban flood modelling. *Environmental Modelling & Software*, **77**, p. 183-195.
- [25] R Core Team, 2016 - "R : A langage and environment for statistical computing", R Foundation for Statistical Computing. Available from <http://www.R-project.org>.