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Assessment of Discharge through a Dike Breach and Simulation of Flood Wave Propagation

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Abstract. This paper presents a simple and fast method to calculate flow through a dike breach. The approach was based on two-dimensional numerical simulations of idealized dike breakages at straight river-sections. As a result, computation of discharge through a breach can be achieved by use of the new developed formula (denoted as *dike break formula*). Furthermore, a methodology that combines one-dimensional hydrodynamic modelling, the dike break formula and a simple GIS-based method to estimate inundation areas is described. This fast and easy-to-handle tool can be utilized for near real-time forecasting or evacuation decisions. Detailed predictions were made for a number of flood and dike break scenarios at the River Rhine to prove the accuracy of the new method compared with two-dimensional numerical models.

Key words: dike break, inundation, Poleni formula, flood waves, numerical simulation

1. Introduction

Any forecasting of the impacts of floods has to balance between accuracy and efficiency. The complexity ranges from simply intersecting a plane representing the water surface (Priestnall *et al.*, 2000) with a Digital Elevation Model for estimating the flooded area to full solutions of the Navier–Stokes equations. Techniques which use two-dimensional, depth-averaged solutions of the Navier–Stokes equations, incline high computational costs and are generally not applicable for near real-time purposes (Bates *et al.*, 2003). However, the sophistication of flood inundation modelling has increased in line with model developments and increased computational resources. Nevertheless, it is still an open question, if simpler models may provide similar levels of predictive ability (Bates and De Roo, 2000). This is the focus of the paper, where we present the development and testing of

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a combined one-dimensional hydrodynamic model and a simplified two-dimensional flood inundation model. The fluxes from one model to the other are calculated with a new empirical approach.

The coupling of one-dimensional hydrodynamic models with two-dimensional methods (e.g. applications based on geographic information systems) is not new and has been well published over the last years. Different researchers reported successful developments of combined models using a variety of flow approximations (e.g. Bechteler *et al.*, 1994; Estrela and Quintas, 1994). Combined one- and two-dimensional hydrodynamic models are recent developments, aimed especially at modelling flood waves and inundation areas (e.g. Gianmarco *et al.*, 1996; Dhondia and Stelling, 2002). Flow within the main river channel is approximated as one-dimensional flow, while the out-of-bank-flow is treated as a two-dimensional situation.

The simplest way to achieve distributed routing of water over the floodplain is to treat each cell as storage volume for which the change of water is equal to the fluxes into and out of it during the time step and to neglect the conservation of momentum. These simple storage models are called raster- or compartment-based methods. However, flow in the main channel should be described by the one-dimensional Saint–Venant equations for conservation of mass **and** momentum.

Large-scale hydrodynamic computations usually assume that the location of the dike-failure and the breach width are given as boundary conditions. The dike is represented as a one-dimensional break-line in the model. Predictions of scouring to obtain the breach growth can be applied i.e. according to Broich (1997) or Visser (1998, 1999) for sand-dikes and according to Verheij (2002) for sand and loamy soil.

In this paper, the main aim is to predict the flooded areas and local water depth for a number of possible scenarios with given location **and** breach width. In these cases the weak point for accurate quantification of inundation areas is the calculation of flow through the breach in the dike.

A widespread approach to calculate the discharge at weirs is the application of Poleni's formula (Equation (1)):

$$Q_{\mathrm{Br}} = 2/3 \cdot \mu \cdot \sqrt{2g} \cdot b \cdot h^{3/2} \tag{1}$$

where g [m/s^2] is the gravitational force, b [m] the length of the weir, h [m] the overfall depth and μ [−] a flow factor which includes all energy losses. This standard formula is very precise as long as μ as known and constant. Unfortunately, for a dike break induced flow both assumptions are not evident. However, Muslu (2002) showed that Poleni's equation is well applicable to calculate flow across side weirs with a non-constant flow factor μ.

Thus, the intention is the calculation of discharge through a given breach and the coupling of a one-dimensional hydrodynamic model with a raster-based approach to quantify inundation areas. For coupling purposes, a modified Poleni formula to calculate the breach discharge (in the following denoted as *dike break formula*) were derived from a number of two-dimensional simulations of synthetic dike breakages.

2. Numerical Models

2.1. NUMERICAL MODEL TO SIMULATE SYNTHETIC DIKE BREAK INDUCED FLOW

To analyze discharge through the dike breach as well as to collect data for the required parameters the river simulation model RISMO was used (Rouvé and Schröder, 1993; Baur *et al.*, 1997).

RISMO solves the two-dimensional, depth-averaged shallow water equations in the Reynolds-averaged form with the finite element method (Kuipers and Vreugdenhil, 1973). The shallow water equations are obtained by vertical integration of the Navier–Stokes equations (Equation (2)) and the continuity equation (Equation (3)):

$$h\left(\frac{\partial U_i}{\partial t}+\frac{\partial U_i U_j}{\partial x_j}\right)=h\left(-g\frac{\partial S}{\partial x_i}+\frac{1\partial}{\rho\,\partial x_j}\left(\rho\nu\frac{\partial U_i}{\partial x_j}-\rho\overline{u_i u_j}\right)+F_i\right) \tag{2}$$

$$\frac{\partial h}{\partial t}+\frac{\partial (hU_i)}{\partial x_i}=0 \tag{3}$$

where h is the flow depth, U_i are time-averaged horizontal velocities, x_i are the Cartesian coordinates, S is the water surface elevation, F are outside influences such as shear stress, ρ the density of water and ν the kinematic viscosity. As turbulence model, the depth-integrated k-ε model is implemented. Bottom roughness is considered by Manning's n [s/m$^{1/3}$]. A wet–dry algorithm considers bank overflow and partly submerged regions.

Manifold dike break scenarios were calculated with the model RISMO. Boundary conditions were adapted from typical flood scenarios at the River Rhine (Rhine km 768.5). Discharge differed between 10,000 and 14,500 m^3/s. The upper range has a return period of approximately 100 years. The mean discharge of the Rhine is 2000 m^3/s. The width of the breach varied from 50 to 400 m.

2.2. COMBINED MODEL WITH APPLIED DIKE BREAK FORMULA

To calculate inundation areas and water-levels with the help of a dike break formula the raster-based tool Floodarea (developed by Geomer, Germany), embedded into a geographic information system, was combined with one-dimensional Saint–Venant modelling using the commercial tool Sobek (developed by WL | Delft Hydraulics, The Netherlands). Floodarea calculates the water-level in each compartment by means of the continuity equation, while the flow across the compartment boundaries is calculated based on water-levels in the adjacent compartments. However, raster-based methods are not as accurate as a depth-integrated Navier–Stokes-based approach, but they are less time-consuming (Bates and De Roo, 2000). Only the variation of the volume $A_j \cdot h_j$ [m^3] in time is solved for each compartment or cell (Equation (4)).

$$\frac{\mathrm{d}h_j}{\mathrm{d}t} A_j = \sum_k Q_{j,k}\big(h_j(\tau), h_k(\tau)\big) \qquad Q_{j,k} = \frac{1}{n} A \cdot R^{2/3} \cdot S^{1/2} \tag{4}$$

In Equation (4) h_j [m] is the water-level of a compartment j, A_j [m^2] is the area of the compartment, $Q_{j,k}$ [m^3/s] are the discharges between the jth and kth compartments, h_k [m] are water-levels in the adjacent compartments at the time $t < \tau < t+1$. The flow $Q_{j,k}$ [m^3/s] is treated e.g. with the Manning formula, where n [s/m$^{1/3}$] is the Manning coefficient, A [m^2] is the wetted section, R [m] is the hydraulic radius and S [−] the slope of free water surface (Estrela and Quintas, 1994).

3. Development of a Dike Break Formula

3.1. SIMPLIFICATIONS

Analytical studies of dike break flow and flood wave propagation must include all relevant forces, i.e. gravity and inertia. Nevertheless the development of the empirical approach requires two essential simplifications:

(1) The empirical dike break formula should be applicable to one-dimensional models and only parameters resulting from those models (e.g. velocity, flow depth and geometric characteristics) were investigated.
(2) Only straight river reaches were analyzed which can be divided into one main channel and two floodplains.

The breach itself was supposed to be a rectangular gap in the dike. Besides, it was assumed that the dike is completely broken down to the level of the floodplain. The width of the breach was defined as constant. As mentioned before, the development of breach growth is not the issue of this paper. Figure 1 shows a typical cross-section of a river with geometric and hydrodynamic variables and the chosen descriptions.

3.2. INFLUENCING CHANNEL WIDTH

Figure 2 shows the contours of the mean velocity $U_r = \sqrt{u^2 + v^2}$ [m/s] and computed streamlines for an idealized two-dimensional Reynolds-averaged simulation of a dike break-induced flow from the riverbed into the polder.

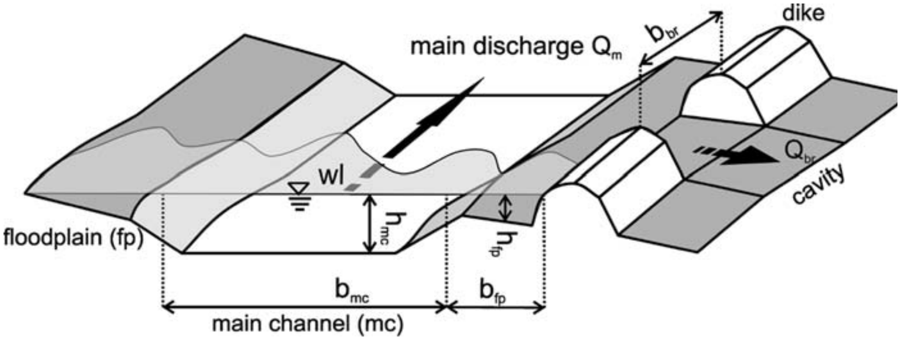

Figure 1. Typical trapezoidal cross-section with geometric parameters.

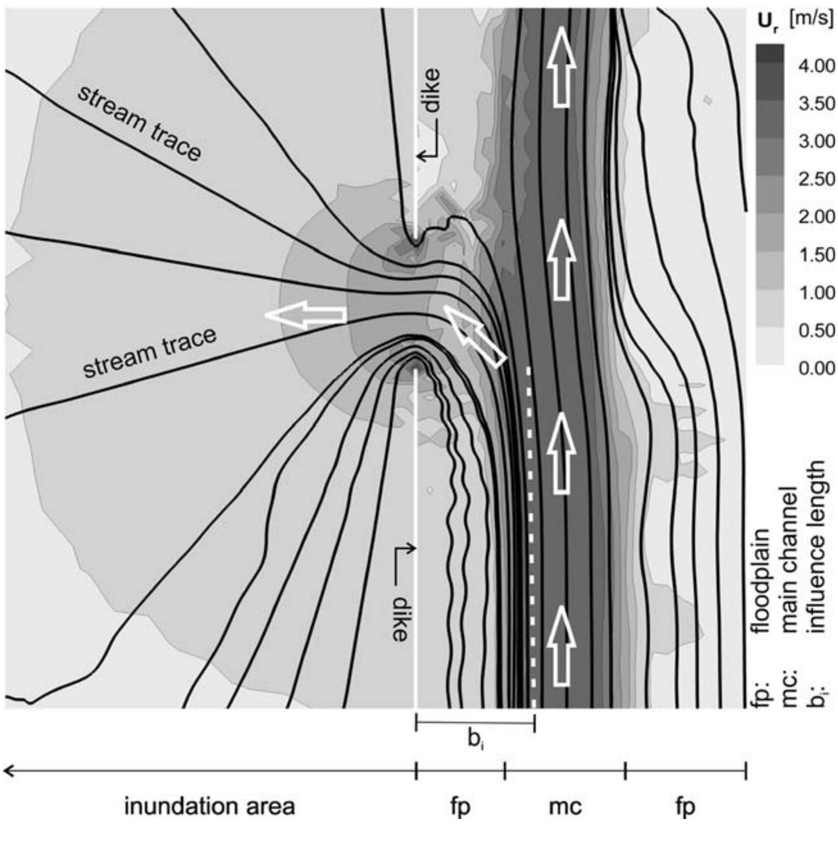

Figure 2. The influencing channel width (influence length).

There is one part of the flow that is not or only little influenced by the breach and another part of the flow that is strongly affected. In the latter case streamlines do not remain confined to the riverbed but pass through the breach. It is obvious that the discharge is highly dependent on the influenced river width, which is called ‘influence length’ b_i [m] here.

The influence length itself is dependent on the length of the breach b_{br} [m]. Thus, we may hypothesize and prove below that flow Q_{br} [m^3/s] through a broken dike could be described by a function of b_i, b_{br}, the main velocity U_r and the flow depth h [m], which denotes the weir head (Figure 1).

3.3. DIMENSIONAL ANALYSIS

Non-dimensional descriptions of hydraulic and geometric characteristics are crucial to compare the results of diverse solutions of a given problem. The Buckingham π theorem is a key theorem in dimensional analysis. The theorem states that the functional dependence between a certain number (e.g. n) of variables can be reduced by the number (e.g. k) of independent dimensions occurring in those variables to give a set of $p=n-k$ independent, dimensionless numbers. Then, different systems which share the same description by dimensionless numbers become equivalent (Curtis *et al.*, 1982).

In consideration of this background, discharge Q_{br} can be written as a Poleni-like formula (Equation (5)). Thus, the discharge through the breach becomes a function of water depth on the floodplain h_{fp}, width of the gap b_{br}, influence length b_i, the gravitational force g and velocity U_r.

$$Q_{br} = \underbrace{\frac{1}{\sqrt{2}} f\left(\frac{b_i}{b_{br}};\ \mathrm{Fr}\right)}_{\equiv 2/3\mu} \cdot \sqrt{2g} \cdot b_{br} \cdot h_{fp}^{3/2} \tag{5}$$

The π theorem states that the flow parameter μ is dependent on two dimensionless numbers: ß = b_i/b_{br} and Fr (Froude’s number).

3.4. RELATIONSHIP BETWEEN μ AND DIMENSIONLESS NUMBERS ß AND Fr

The maximum applicable flow factor μ for wide-crested weirs with flow approaching parallel to the gap can be derived with the Bernoulli energy conservation equation and is (Bollrich, 1996):

$$\mu_0 = \frac{1}{\sqrt{3}} = 0.577 \tag{6}$$

The maximum value is called μ_0 henceforth. A normalized dike breakage parameter $\mu^* = f$ (ß, Fr), which can vary between zero and one, may describe the reducing influence of the dike breakage. Thus, μ is the product of μ_0 and μ*:

$$\mu = \mu_0 \cdot f(\beta, \mathrm{Fr}) = \mu_0 \cdot \mu^* \tag{7}$$

First attempts to determine μ* using the automatic polynomial neuronal network approach GMDH (Group Method of Data Handling, e.g. Aksyonovaa *et al.*, 2003) failed to achieve the intended result. Thus, the unknown function was bisected in an internal polynomial distribution and an outer logarithmic or exponential relation. The polynomial distribution should fit the following relationship:

$$\xi(\beta, \mathrm{Fr}) = \alpha \cdot \beta^i \cdot \mathrm{Fr}^k \tag{8}$$

First appropriate exponents i and k as well as the parameter α had to be found by reasonable trials. Afterwards the external function $f(\xi)$ could be achieved by regression analysis. By Monte-Carlo-simulations for varying α, i and k a suitable relation was found for:

$$\xi = 0,4 \cdot \sqrt{\mathrm{Fr}} \cdot \left(\frac{b_i}{b_{br}}\right)^2 \tag{9}$$

Figure 3 shows the correlation between ξ and the dike parameter μ* as computed by numerical simulations. The best fitting function for the data points of Figure 3 is a natural logarithm. The correlation coefficient R^2 is 0.87. In contrast, when μ is assumed to be constant ($\mu = 0.577$) for all test cases, the correlation between empirical calculation and numerical simulation is small.

Finally, Equations (10) and (11) present the new empirical formula to calculate μ* and Q_{br} respectively:

$$\mu^* = 0.1146 \cdot \ln(\xi) + 0.6895 \tag{10}$$

$$Q_{\mathrm{br}} = 2/3 \cdot 0.577 \cdot \mu^* \cdot \sqrt{2g} \cdot b_{\mathrm{br}} \cdot h_{\mathrm{fp}}^{3/2} \tag{11}$$

The result of Equations (9) and (10) is the dike breakage parameter μ*. Table I lists discharges computed by Equation (11) versus discharges extracted from numerical simulations. Besides, discharges with constant μ are listed as well.

While the average error of the results with constant $\mu^* = 1.0$ is about 125% (see Table I, column Q_{br}^c) compared to the depth-integrated numerical simulations, the results of the dike-break approach show only an average difference of 24% (see Table I, column Q_{br}^a), which is quite an improvement. The maximum error is 150% for the

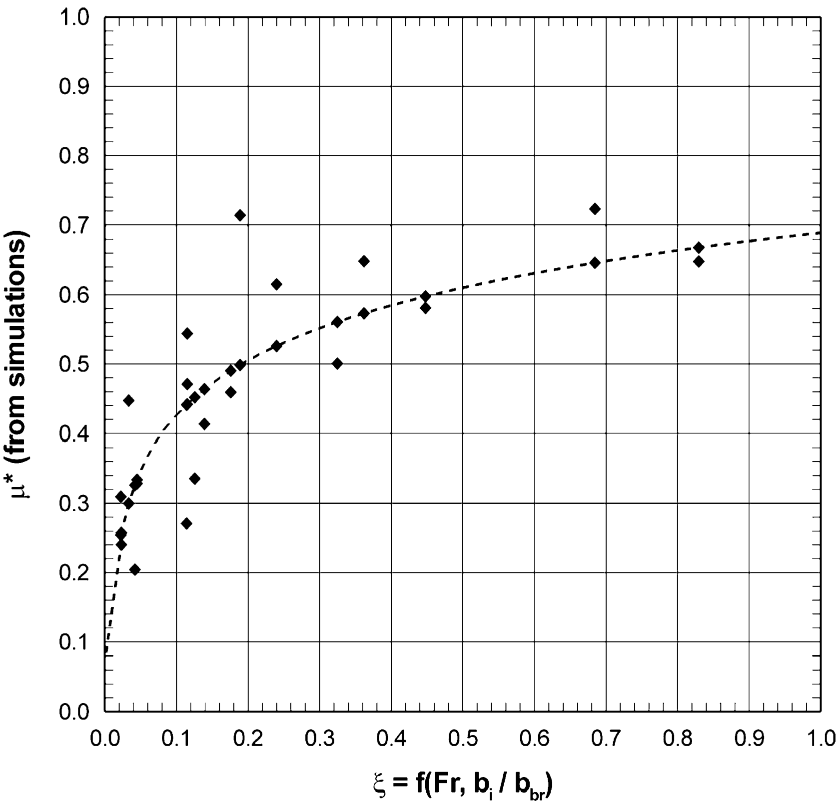

Figure 3. Correlation between ξ and the dike break parameter μ *.

applied dike-break algorithm and 262% for a constant values of $\mu^* = 1.0$.

3.5. APPROXIMATION OF INFLUENCE LENGTH

For the development of the dike break formula the influence length b_i was extracted from the numerical data. For the application of the one-dimensional model, b_i has to be estimated. It could be proved, that b_i is a function of the width of the floodplain and the floodplains Froude number (Equation (12)):

$$\beta = \frac{b_i}{b_{br}} \approx (1.18 - \mathrm{Fr}^{0.5}) \cdot \frac{b_{fp}}{b_{br}} \tag{12}$$

When calculating the influence length by Equation (12), the overall correlation is smaller ($R^2 = 0.72$) but remains acceptable. The complete algorithm

Table I. List of parameters and calculated results for dike break formula, constant flow factor $\mu = 0.577$ and numerical simulations (steady state).

Breach [m]	Width$_{fp}$ [m]	h_{fp} [m]	b_i/b_{br} [–]	Fr$^{0.5}$ [–]	μ^{*a} [–]	μ^{*b} [–]	Q_{br}[a] [m^3/s]	Q_{br}[a] [%]*	Q_{br}[b] [m^3/s]	Q_{br}[c] [m^3/s]	Q_{br}[c] [%]*
50	200	2.10	3.060	0.318	0.710	0.345	184	106	89	259	191
50	400	3.09	3.045	0.295	0.693	0.276	321	150	128	463	262
100	200	4.25	2.273	0.370	0.668	0.648	997	3	968	1494	54
150	400	4.99	2.215	0.349	0.646	0.723	1843	−11	2063	2854	39
200	400	2.96	1.995	0.298	0.597	0.581	1035	3	1007	1735	72
200	400	2.30	1.394	0.466	0.572	0.648	680	−12	771	1189	54
200	400	7.62	0.850	0.481	0.462	0.413	3311	12	2966	7174	142
250	200	4.06	1.043	0.417	0.497	0.714	1734	−30	2491	3491	40
250	400	4.68	1.659	0.295	0.559	0.500	2414	12	2159	4315	100
300	400	4.15	0.568	0.348	0.331	0.328	1430	1	1420	4324	205
350	200	4.48	0.793	0.410	0.440	0.471	2491	−7	2670	5668	112
350	400	4.07	1.323	0.295	0.524	0.615	2574	−15	3019	4909	63
400	400	2.06	0.996	0.317	0.450	0.335	910	34	678	2023	198
400	400	2.25	0.352	0.452	0.250	0.309	576	−19	713	2305	223

*Percentage error compared to 2D simulation.
[a]Data derived with dike break formula.
[b]Data extracted from 2D simulation.
[c]Constant parameter, $\mu = 0.577$ ($\mu^* = 1.0$).

for the dike break formula including all relevant parameters is given in Figure 4.

3.6. VALIDATION

Preliminary to application of the dike break formula for the combined model, flooding of the polder Friemersheim at the River Rhine earlier simulated with RISMO was chosen as a validation case (Köngeter *et al.*, 2001). In this scenario any effects of backwater were negligible because of the enormous size of the polder area. The water depth and velocities in the river resulted from the performed numerical simulations and were used as boundary conditions for the dike-break algorithm. The discharge through the breach was then recalculated with the use of the dike break formula. Figure 5 plots the original outflow and actual polder volume for a period of 125 hr against the calculated values. The results show, that the overall error is less than 20%. This is a quite good result especially when comparing the ratio of computational costs which is 1:20,000 [dike-break algorithm: numerical simulation].

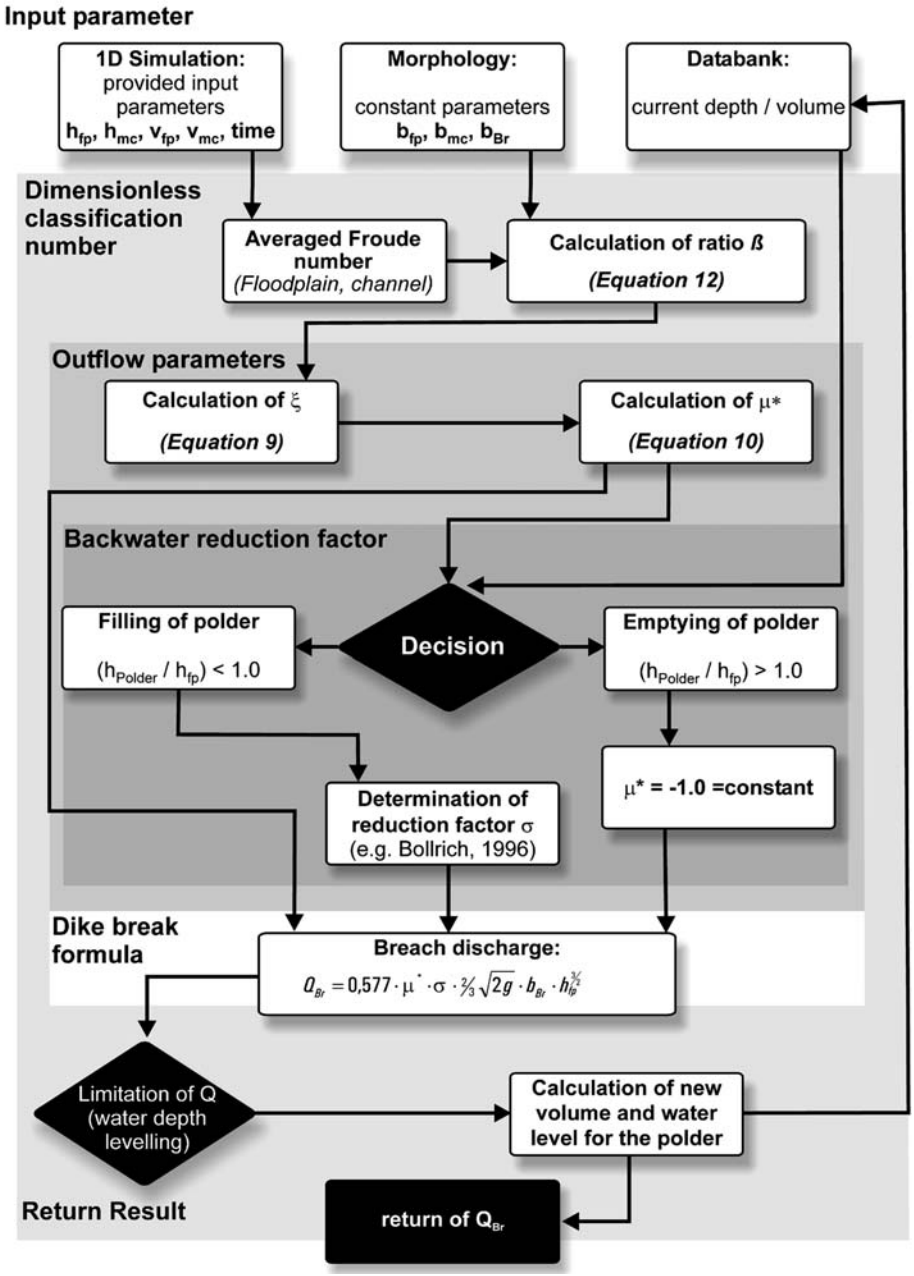

Figure 4. Algorithm for flow computation due to a dike breakage. Note: reduction of flow by multiplier σ when water depth behind the breach reaches 70% of the depth in front of the breach (measured from the elevation of the floodplain) (Bollrich, 1996).

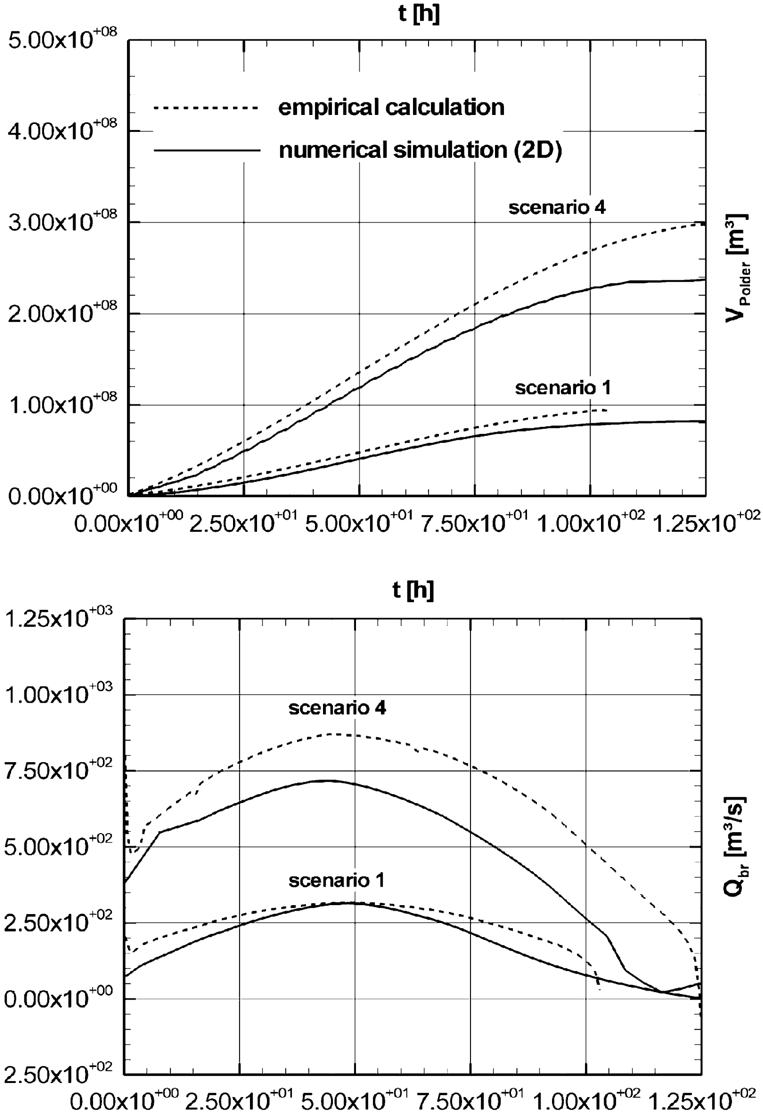

Figure 5. Comparison between a two-dimensional simulation of a dike break induced flow and a one-dimensional model with applied dike break formula.

4. Application

4.1. COMBINED MODEL OF THE RIVER RHINE AND ITS MAIN TRIBUTARIES

Temporal and spatial development of water-levels and discharge during chosen flood scenarios in the river Rhine and its main tributaries were calculated by the *Federal Institute of Hydrology* (BfG) within in the

Table II. One-dimensional hydrodynamic model of the River Rhine and its main tributaries.

River	Rhine	Neckar	Main	Moselle
Length [km]	500	61	251	52

framework of the research project *German Research Network Natural Disasters* (DFNK). One aim of these calculations was the consideration of dike break-scenarios. The simulations might give considerable advice for evacuation decisions during an actual event and predict water-level reductions downstream the breach. The unsteady simulations were performed by coupling the one-dimensional hydrodynamic model Sobek and the raster-based two-dimensional approach Floodarea with the dike break formula.

The model Sobek covers a total length of 864 km (Table II). Dikes accompany the River Rhine over a total length of 650 km (left and right bank). The model was calibrated by means of past flood events in 1988 and 1995 and validated with measured data of flood events in the year 1993 and 1983. Besides the simulation of the flood-events in 1993 and 1995, an artificially generated possible maximum flood-event 1995^{+} was also considered. The hydrograph of the 1995^{+} flood results by increasing discharges of the flood from the year 1995 by 50%. Thus, it was chosen as a worst-case scenario. Inundation areas, discharge through a defined breach and water-levels were calculated for every hour. The effect of a dike breakage on flood wave propagation was examined exemplary for two polders at the River Rhine. For the first scenario a dike breakage at Rhine km 768.5 close to Dusseldorf was chosen and secondly a scenario at Rhine km 802.0 at the small village Mehrum was selected.

Computed results differed due to the influence of topography. The area of case one (768.5 km) covers a very large surface. Therefore, backwater effects are negligible. In contrast, the area around Mehrum (802.0 km) has only a limited retention volume of 65.9×10^6 m^3 with an inundation surface of 14.4 km^2. So, flow back into the Rhine is an important aspect to consider.

For both scenarios, the dike broke at the moment when 80% of the maximum water-level compared to the mean water-level was reached. The duration of the breaking process was less than 15 min with a resulting breach width of 110 m.

4.2. COMPARISON BETWEEN TWO-DIMENSIONAL SIMULATIONS AND COMBINED MODEL AT POLDER MEHRUM

Figure 6 compares the water volumes stored in the polder due to a dike breakage simulated with a two-dimensional simulation and the combined

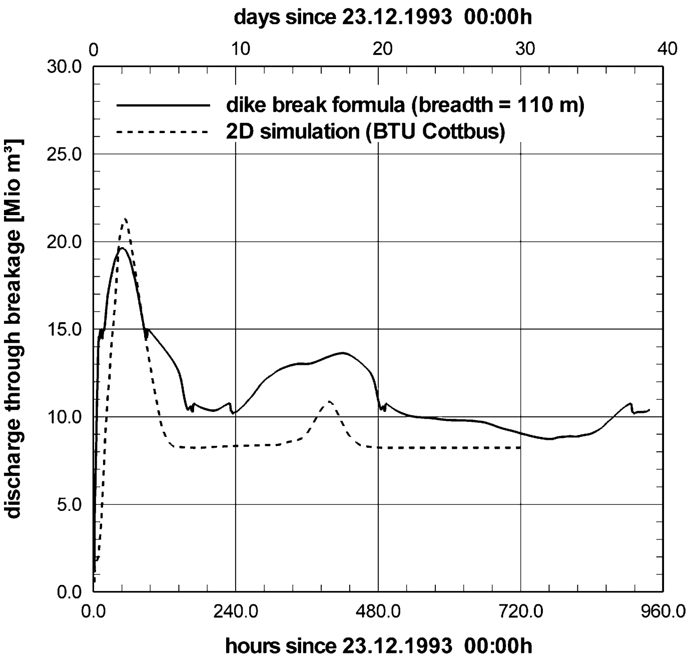

Figure 6. Comparison between two-dimensional simulation and combined model at Polder Mehrum.

model with the applied dike break formula. In general, the combined model overestimates the volume in the polder, but it calculates the maximum at the same time after 48 hr. The main reason for the differences lies in an incorrect waterlevel-volume relation, which determines the water height differences between the polder and the main channel.

4.3. INFLUENCE OF BREAK TIME AND BREACH WIDTH

Due to the short computation time of the combined model it was possible to analyse by manifold simulations the influences of break time and breach width on the flood routing process. The chosen breach widths are 50, 75, 100, 125 and 150 m respectively. Breaking times were defined, when 80, 90 and 100% of the height between the mean water-level and the flood crest are reached.

When the dike fails before the discharge reaches its maximum peak, an intentional breakage is a realistic possibility to flood a defined area in order to protect the downstream region. As an example, Figure 7 plots water-level reductions of a 250 km section upstream and downstream from the breach ($b_{br} = 150$ m). For this case a downstream water-level reduction of 0.6 m could be achieved. Furthermore, 100 km downstream of the breach still remains 50% of the maximum reduction.

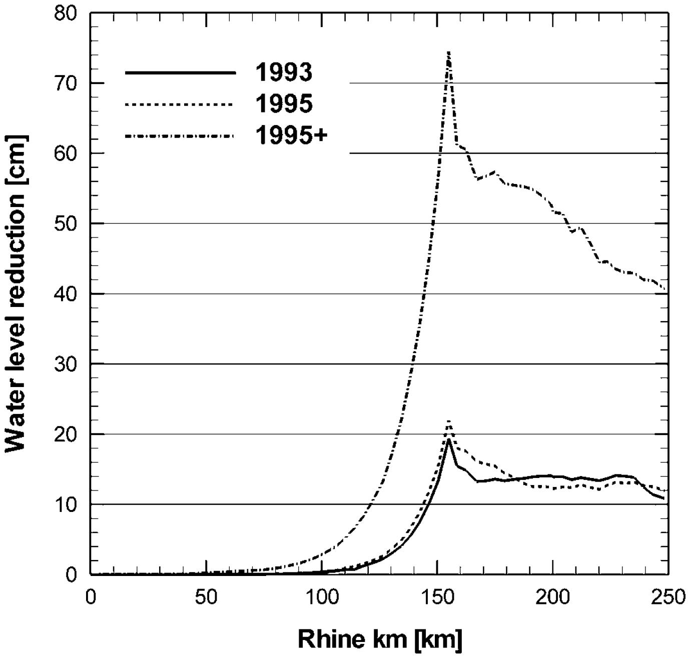

Figure 7. Water-level reduction in the Rhine due to a dike breakage ($b_{br} = 150$ m). Breaking time: 80% of the height between the mean water-level and the flood crest.

5. Conclusions and Perspectives

For flood forecasting it is necessary to find a balance between efficiency and accuracy. Multi-dimensional hydrodynamic models might have a high level of accuracy, but they are still not fast enough for real-time forecasting purposes. Combining the presented dike break formula with a one-dimensional hydrodynamic river model and a simplified two-dimensional polder model leads to an accuracy comparable with that achieved by two-dimensional simulation techniques.

The new formula for discharge calculations in the event of sudden dike failure offers the following advantages: (1) Increasing the level of accuracy compared to commonly used weir equations (e.g. Poleni). (2) Minimizing effort of calculation compared to multi-dimensional methods. (3) The algorithm can be quickly and easily integrated in existing flood forecasting systems.

Additionally, reduced water levels downstream of the breach can easily be calculated and real-time predictions of water depths in the inundated polder become feasible. Together with a well-organized information system, existing forecasting systems can be improved in the case of dike failure (see Figure 8). Thus, the presented coupling of models by the dike

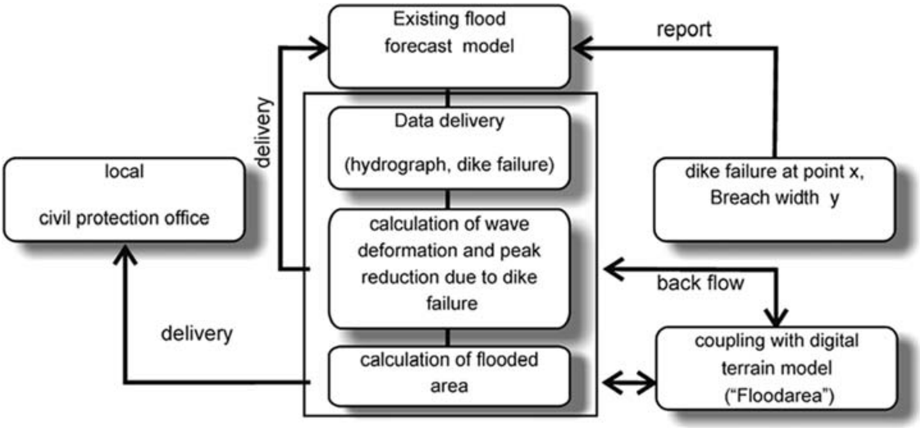

Figure 8. Principle model scheme of a forecasting module connected with the combined simulation module for dike break scenarios.

break formula is a great improvement compared with today's methods to calculate flooded areas with extrapolated water-levels from one-dimensional models.

In the end, two-dimensional simulations of dike break scenarios remain necessary to calibrate simplified models. However, the developed dike break formula is only the first step. Further research is needed to determine the dynamic filling of the polder. Thus, a reliable waterlevel–volume-relationship has to be established in order to consider backwater effects more realistically.

References

Aksyonovaa, T. I., Volkovich, V. V., and Tetko, I. V.: 2003, Robust polynomial neural networks in quantitative-structure activity relationship studies, *Syst. Anal. Model. Simul.* **43**(10), 1331–1339.

Bates, P. D. and De Roo, A. P. J.: 2000, A simple raster based model for flood inundation simulation, *J. Hydrol.* **236**, 54–77.

Bates, P. D., Marks, K. J., and Horrit, M. S.: 2003, Optimal use of high resolution topographic datain flood inundation models, *Hydrol. Process.* **17**, 537–557.

Baur, T., von Lukowicz, J., and Köngeter, J. (1997). Numerical optimization of a Riverbed-geometry with special regard to erosion, Proc. 3rd Int. Conf. on River Flood Hydraulics. In J. Watts (ed.), *Proc. 3rd Int. Conf. on River Flood Hydraulics*, South Africa: Stellenbosch.

Bechteler, W., Hartman, S., and Otto, A. J. (1994). Coupling of 2D and 1D models and integration into Geographic Information Systems (GIS), Proc. 2nd Int. Conf. on River

Flood Hydraulics. In W. R. White & J. Watts (Eds.), *Proc. 2nd Int. Conf. on River Flood Hydraulics* (pp. 155–165). Chicester, UK: John Wiley and Sons.

Broich, K.: 1997, Computergestützte Analyse des Dammerosionsbruchs, Institut für Wasserwesen, UniBw München, Dissertation am Lehrstuhl Hydromechanik und Hydrologie, Mitteilungen 61, München.

Bollrich, G.: 1996, *Technische Hydromechanik I, Grundlagen. 4th edn*, Verlag für Bauwesen, Berlin, Germany.

Curtis, W. D., Logan, J. D., and Parker, W. A.: 1982, Dimensional analysis and the π theorem, *Lin. Alg. Appl.* **47**, 117–126.

Dhondia, J. F. and Stelling, G. S.: 2002, Application of one-dimensional two-dimensional integrated hydraulic model for flood simulation and damage assessment. In: R. A. Falconer, B. Lin, E. L. Harris and C. A. M. E. Wilson (eds.), *Proc. 5th International Conference in Hydroinformatics*, Vol. 1, Cardiff, UK, pp. 265–276.

Estrela, T. and Quintas, L. (1994). Use of GIS in the modelling of flows on floodplains, Proc. 2nd Int. Conf. on River Flood Hydraulics. In W. R. White & J. Watts (Eds.), *Proc. 2nd Int. Conf. on River Flood Hydraulics* (pp. 177–189). UK: John Wiley and Sons.

Gianmarco, P. Di., Todini, E., and Lamberti, P.: 1996, A conservative finite elements approach to overland flow: the control volume finite element formulation, *J. Hydrol.* **175**, 267–291.

Köngeter, J., Briechle, S., and Liem, R.: 2001, Bestimmung der Überflutungsflächen zur Festlegung der Verbands- und Poldergrenzen der Deichverbände Friemersheim, Orsoy und Poll – Hydrodynamische Simulation ausgewählter Deichbruchszenarien entlang des Rheins, Technical Report, Institute of Hydraulic Engineering and Water Resources Management Aachen, Germany.

Kuipers, J. and Vreugdenhil, C. B.: 1973, Calculations of two-dimensional horizontal flow, Delft Hydraulics Laboratories, Report No. S163–1.

Muslu, Y.: 2002, Lateral weir flow model using a curve fitting analysis, *J. Hydraulic Eng.* **128**(7), 712–715.

Priestnall, G., Jaafar, J., and Duncan, A.: 2000, Extracting urban features from LiDAR-derived digital surface models, *Comput. Environ. Urban Syst.* **24**, 65–78.

Rouvé, G. and Schröder, M.: 1993, One-dimensional vs. two-dimensional prediction of compound open channel flow. In: *Advances in Hydro-Science and Engineering. Proc. of the ICHE-'93*, Vol. 1, June 7–11, Washington DC, USA.

Verheij, H. J.: 2002, Breaching in Cohesive Soils. WL|Delft Hydraulics, Research Report Q2959, Delft, The Netherlands.

Visser, P. J.: 1998, Breach growth in sand-dikes, Doctoral dissertation (also published in the series "*Comm. Hydraulic Geotechn. Eng.*", *Fac. Civil Eng. Geosci.*), Delft Univ. Techn., Rep. 98–1.

Visser, P. J.: 1999, Breach erosion in sand-dikes. In: *Proc. 26th Int. Conference Coastal Eng.*, Copenhagen, 1998, pp. 3516–3528.