NUMERICAL INVESTIGATION OF FLOODING OF REAL-TOPOGRAPHY DEVELOPED AREAS FOLLOWING RIVER EMBANKMENT FAILURE

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Abstract: The paper concerns numerical simulations of flood wave propagation in an urban area resulting from a river embankment failure. Simulations have been performed to predict and analyze the parameters of flash and catastrophic flow in a developed area with the aim of presenting numerical calculations useful for identification of inundation zones. The shallow water equations were assumed as the mathematical model of free-surface unsteady water flow. A numerical scheme of the finite volume method was applied to solve the model equations and the Roe method used to approximate the mass and momentum fluxes. Two test cases of embankment failure are investigated in the paper. The aim of one experiment is to simulate a flood in a model city area, where a group of buildings representing a simplified urban configuration was introduced. In order to verify the calculations, numerical results were examined against experimental data available from laboratory measurements. An experiment of the model city's flooding event was carried out at the hydraulic laboratory of the Gdansk University of Technology. The other test case concerns flash flood simulation on an embanked developed area of real topography.

Keywords: numerical simulation, river embankment failure, inundation, developed area

1. Introduction

Numerical simulation of inundations seems to be the main tool for assessment and reduction of the risks of flooding. As flood prediction is indispensable in management of flood risks in natural and urban areas, estimation of flood inundation parameters is an important task for water engineers. It allows creating inundation risk maps, *i.e.* identification of hazard zones, in accordance with article 82 of the Polish Water Law. Simulations of flood events are necessary to better protect the floodplains, cities and embanked areas thus enhancing public safety. They can be used to develop emergency plans, control development within potential flood zones or to determine property insurance premiums. Adequate information about potential flood must include [1, 2]:

- the time of flood wave's arrival at characteristic points of the inundated area,
- the maximum depth, velocity and energy of water in the flooded area, and
- the duration and range of flooding.

When estimating the extent of flood zones, digital terrain models (DTM) of the flood area and mathematical models are needed to describe flood wave propagation. Models of flood dynamics range in complexity from simple one-dimensional Bernoulli equations for steady flow to full three-dimensional solutions of the Navier-Stokes equations with turbulence models [3] for detailed description of the flood wave's transformation. Unfortunately, the latter have been so far too complex to be applied in the majority of practical cases [4, 5].

Recently, the most widely used models of flood wave propagation have been the Saint Venant (SV) equations for one-dimensional open channel flow solved with numerous finite difference method schemes (FDM) or the finite element method (FEM) [6–8]. Such schemes describe the river and floodplain as the series of cross sections. Numerical solution of the governing equations enables a cross section-averaged velocity and water depth (or water table elevation) to be calculated at each location. In order to identify the inundation zone, the water surface level values can be interpolated between cross-sections and joint with a DTM. Finally, visualization of the results can be performed, in the form of flood zones maps, using Geographical Information Systems (GIS) [9, 10].

Although one-dimensional hydrodynamic models are usually efficient for modeling water wave propagation in open channels, they are often inconvenient for inundation prediction due to their rough representation of the flooded area's topography. The flow area of a hydrodynamic model being defined with a limited number of cross-sections of the river channel and the floodplain may lead to significant errors in the predicted extent of inundation, identified by the intersection line between the interpolated water surface and the DTM. Two-dimensional flood propagation models can be applied in order to overcome this problem, as they ensure almost continuous, DTM-based representation of topography in hydrodynamic computations [1, 2, 4, 11, 12]. Moreover, two-dimensional models allow the water's level and averaged velocity to be computed at each computational node, so that no interpolation process is required to determine the flood range. Additionally, solving two-dimensional hydrodynamic equations is necessary when two-dimensional water flow effects are expected and cannot be omitted, *e.g.* with flooding following river embankment failure. After embankment failure a body of water is released and the flood wave propagates downstream in many directions of the horizontal plane. Two-dimensional flow analysis is also indispensable for flow simulation in developed areas. The flow in a city has some specific features due to its interaction with buildings and other structures, with characteristic effects [13] such as wave reflections, diffractions, intersections, sudden water surface swallows and depressions with steep gradients, local velocity variations due to contraction among buildings, flow splitting, wake zones, hydraulic jumps, *etc.* Such complex, rapidly varied flow must be modelled with two-dimensional models solved with dedicated numerical methods. In this paper, flood wave propagation due to river embankment failure in a developed area is investigated and the shallow water equations are assumed as the mathematical model of free-surface unsteady water flow. A numerical scheme based on the finite volume method (FVM) is applied in order to solve the equations.

2. The mathematical model and the solution method

Generally speaking, flood wave propagation is a three-dimensional, time-dependent and incompressible problem of fluid dynamics with a free surface. It can be perfectly described with the Navier-Stokes equations (NSE), the dynamic model of any fluid [3]. However, due to the complexity of data needed to perform computations and the insufficient computer power available at the moment, this general flow model cannot be applied to simulate practical cases of flooding. The model most widely used to simulate flood propagation in natural river valleys is a system of shallow water equations (SWE) [4]. It can be obtained from the NSE model through a depth-averaging procedure, assuming hydrostatic pressure and uniform velocity distribution along the water's depth [8].

The SWE model used here for a numerical simulation of flooding following river embankment failure can be presented in a conservative form as [6]:

$$\frac{\partial \boldsymbol{U}}{\partial t}+\frac{\partial \boldsymbol{E}}{\partial x}+\frac{\partial \boldsymbol{G}}{\partial y}+\boldsymbol{S}=0, \tag{1}$$

where

$$\boldsymbol{U}=\begin{pmatrix} h \\ uh \\ vh \end{pmatrix},\quad \boldsymbol{S}=\begin{pmatrix} 0 \\ -gh(S_{ox}-S_{fx}) \\ -gh(S_{oy}-S_{fy}) \end{pmatrix},\quad \boldsymbol{E}=\begin{pmatrix} uh \\ u^2h+\frac{1}{2}gh^2 \\ uvh \end{pmatrix},\quad \boldsymbol{G}=\begin{pmatrix} vh \\ uvh \\ v^2h+\frac{1}{2}gh^2 \end{pmatrix}. \tag{2}$$

In this system of equations, h represents water depth, u and v are the depth-averaged components of velocity in the x and y direction, respectively, S_{ox} and S_{oy} denote the bed slope terms, S_{fx} and S_{fy} are the bottom friction terms defined by the Manning formula, and g is the acceleration due to gravity.

The mathematical model of free-surface water flow (Equations (1)–(2)) was solved using the Roe scheme [14, 15] of FVM [16]. The numerical algorithm was prepared and tested for simulation of flood wave propagation resulting from dam-breaking events in natural river valleys [1], adapted for flow in urban areas [13]. Herein, the model is used to investigate the flooding of a developed area due to river embankment failure. First, the mathematical model was validated with depth variation measurements performed during a city model flooding event at the Gdansk University of Technology's hydraulic laboratory. Then, the model was used to predict the flash flood hydraulic effects in a real-topography embanked urban area. In order to carry out computations for the second test case, the flow domain was established using a DTM of the floodplain according to the real topography of the inundated area [2, 11, 12]. Then, the area was meshed assuming that all buildings could be excluded from the computational domain and that their walls fitted the cells' edges exactly. Triangular meshes were used to cover the flow area.

3. Test case 1: flood wave in a model developed area

A hydraulic test stand was prepared at the hydraulic laboratory of the Gdansk University of Technology in order to carry out the experiment of city flooding due to river embankment failure [17]. A measuring station, 6.75m long and 3.5m wide, was built in concrete, steel and wood and to reproduce the riverbed and the floodplain (see Figure 1).

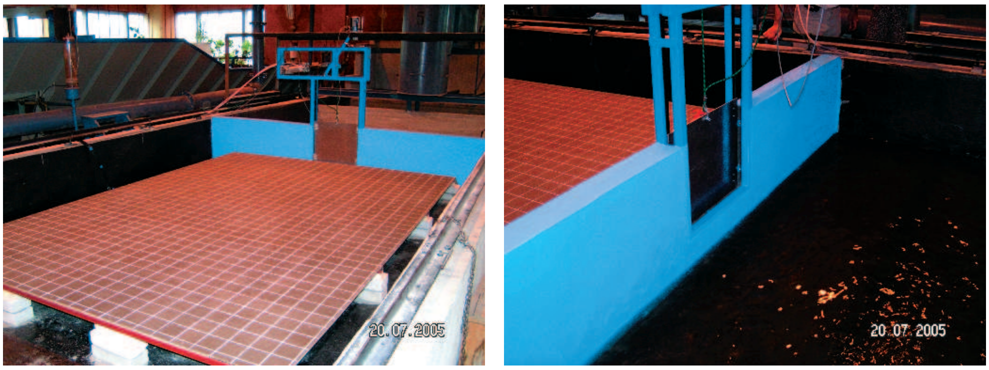

Figure 1. Test case 1: general view downstream and upstream of the hydraulic test stand

The hydraulic test stand was composed of a river, represented with a reservoir (3.0m long, 3.5m wide), and a floodplain along the river, simulated by a horizontal flat plate (3.75m long, 3.0m wide). The flow area geometry is presented schematically in Figure 2.

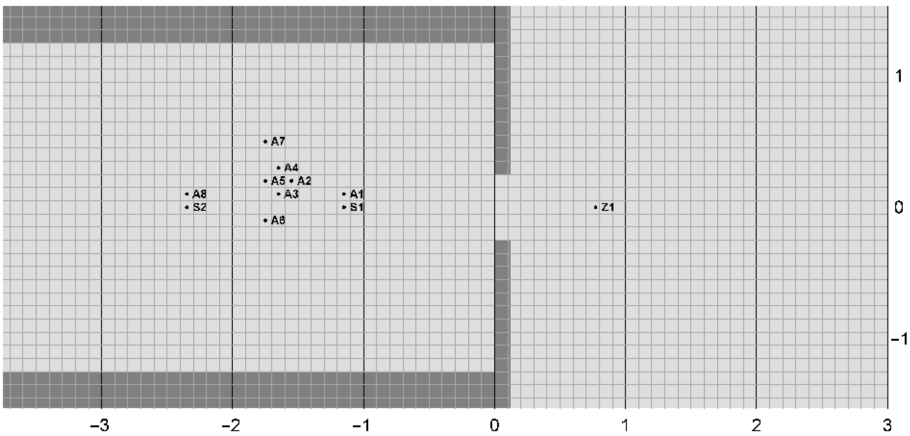

Figure 2. Test case 1: geometry of the flow area (light grey) and sensors' location (dimensions in metres)

The floodplain was separated from the river (reservoir) with an embankment built as a masonry wall 0.12m in width. Three other boundaries of the plate were open, ensuring free water falling into the outflow basin. The embankment wall contained a rectangular breach 0.5m in width, initially closed by a gate. The gate could be open fully and suddenly (in 0.1s) with a remote control system. The sudden opening of the gate simulated river embankment failure with flood wave propagation across the floodplain. The failure process itself was not considered in the simulation. The Manning friction coefficient of the plate and reservoir bottom was estimated as $n = 0.018\,\mathrm{m}^{-1/3}\mathrm{s}$ [17].

Depth variation was measured during the experiment at control points shown in Figure 2 with ten pressure sensors installed under the plate (S1, S2, A1–A8). Additionally, a depth-control gauge was installed in the reservoir (Z1). The sensors were used to measure water depth at various locations as a function of time. The measurements were recorded and stored on a personal computer (PC) and could be used to verify the computation.

Models of buildings could be installed on the plate in order to simulate a flood in a developed area. The models were built as cubes with 0.1 m sides: high enough not to be toppled by the flood wave. A few configurations of the buildings' layout were investigated during laboratory research, but only one test case is presented in this paper – the flow in aligned buildings' configuration. In this test, the buildings were placed in rows parallel to the main axis of the plate forming a grid configuration (see Figure 3).

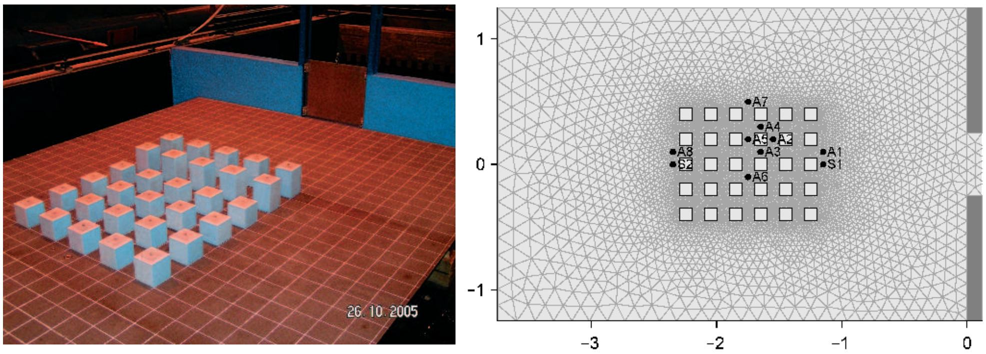

Figure 3. Test case 1: the aligned buildings' configuration and a part of the numerical mesh

Initially, there was no water on the floodplain and the reservoir was filled up to 0.21 m above the plate's level. After filling the reservoir with water, the hydrostatic state was achieved using a pipe weir installed in the test stand's bottom. After opening the gate, the body of water was released and the floodplain with the developed area was flooded. The experiment was repeated thrice and the time series of water-depth variation were averaged. In general, good measurement repeatability was observed [17].

In order to simulate the city flooding experiment, the calculation domain was covered by an unstructured triangular mesh composed of 11 860 computational cells (see Figure 3). The mesh used in the numerical simulation was locally refined between buildings to better represent the complex structure of flow in the city area. The sides of computational cells around each building were 0.02 m in length, increasing to 0.15 m near the flow area's boundaries.

The boundary conditions were imposed in accordance with the experiment. The reservoir walls were treated as closed boundaries. The free outflow condition was imposed at the open boundaries of the floodplain. The areas occupied by the buildings were excluded from the flow domain: closed boundaries were located at the buildings' edges. The calculation was carried out with a time step of $\Delta t = 0.005$s, ensuring the calculation's stability. The total simulation time was 20s.

The flow in the developed area can be generally described with the results of numerical simulations presented in Figure 4, where the shape of computed water surface after 3 seconds of numerical simulation juxtaposed with the photo of the observed flow at approximately the same time.

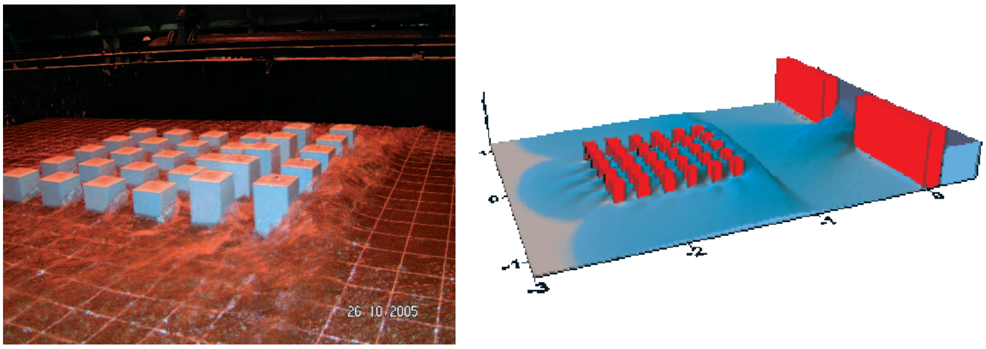

Figure 4. Test case 1: observed and computed water surface after $t = 3$s of simulation

The flood wave's front reached the first row of buildings after about 0.4s of simulation. Having reached the urban area, swelled water level and decreased velocity could be observed upstream of the first row of buildings, where the flood wave was reflected against the buildings and formed an upstream-moving hydraulic jump. Then, the flow suddenly accelerated along longitudinal streets. A re-circulation zone with a steep gradient of water surface could be observed at the downstream side of each building (see Figure 5).

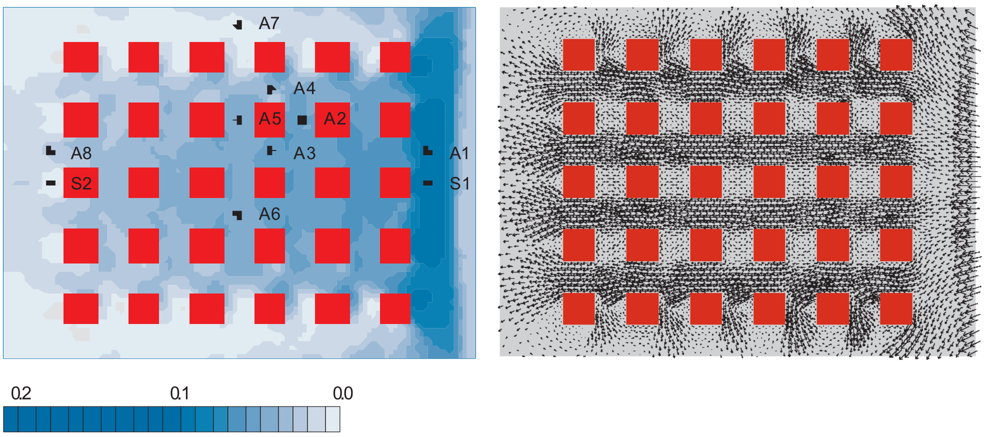

Figure 5. Test case 1: contours of the water depth [m] and velocity vectors after $t = 3$s

A comparison of computed and measured depth variations at the control points is shown in Figure 6. Agreement of the results is quite good, though the calculated

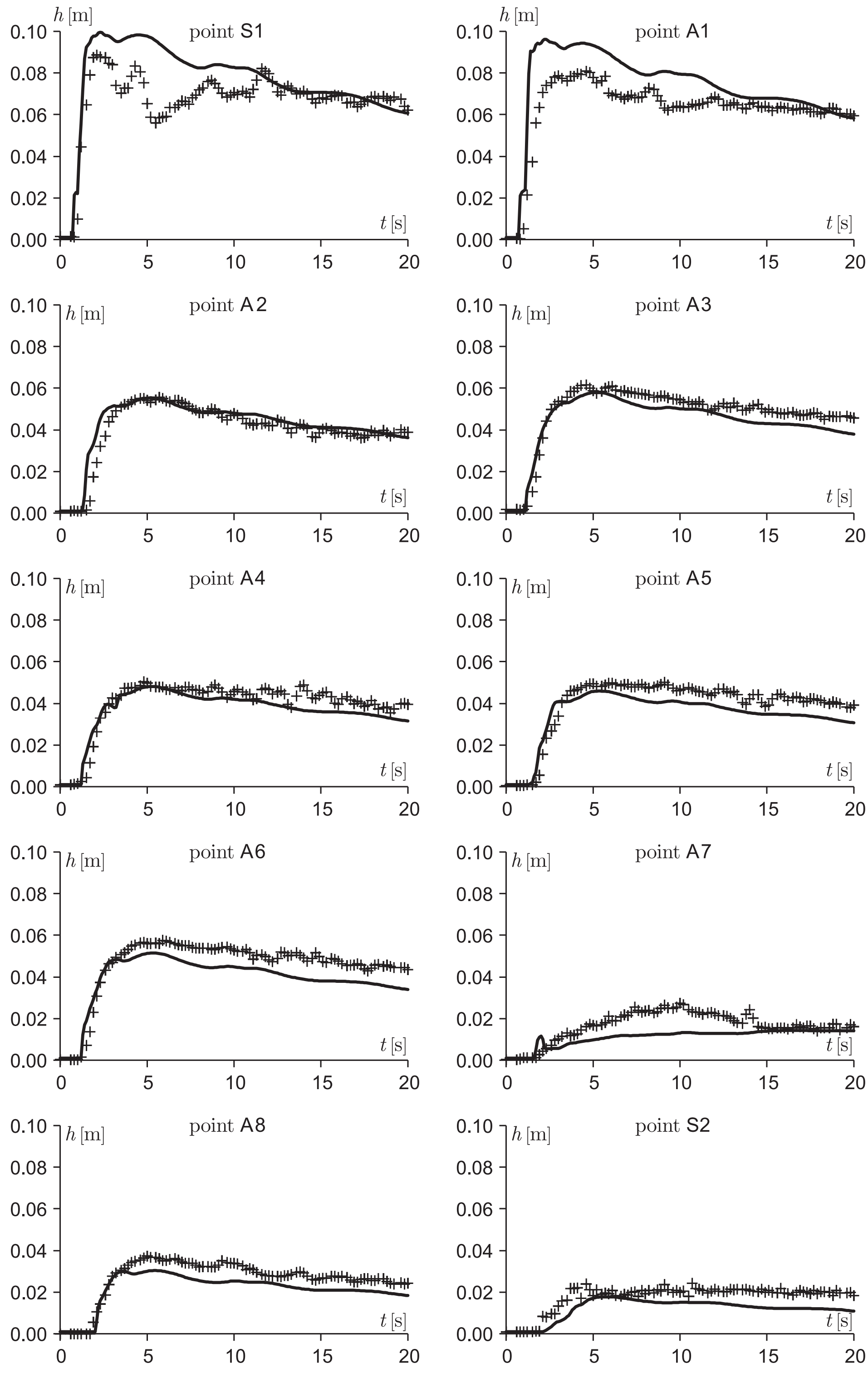

Figure 6. Test case 1: depth variation calculated (−) and measured (+) at control points

water depth within the urban area (*e.g.* at points A3, A5, A6) tends to be slightly underestimated. It is probably due to additional energy losses (*viz.* local, building wall friction) ignored in the mathematical model of the water flow, in which only the bottom friction was considered.

The water depth at points S1 and A1, placed upstream of the first row of buildings (*cf.* Figure 3), has been overestimated. The discrepancy of results at these locations can be attributed to the water splash effect observed just after wave reflection against the first row of buildings. A splashing body of water cannot be measured with hydrostatic sensors or be simulated with the SWE model.

Small oscillations of the water level can also be observed in measurements at each control point. They are attributable to measurement error and reflections against walls. They are generally a result of the complex structure of the flow area where rapidly varying water flow is present. At the same time, the graphs of computed water depth history are smooth. This is so because 3D hydraulic effects cannot be represented in 2D numerical simulations at all. Moreover, the numerical scheme used to solve the SWE model is locally dispersive and adds some numerical diffusion to the solution ensuring stability of computations and resulting in a more regular shape of the solution.

The good result of comparison between laboratory investigation and numerical simulation of an extreme city flooding event due to river embankment failure has confirmed that SWE can be used as a model of rapidly varying surface flow in urban areas.

4. Test case 2: inundation of real-topography embanked urban area

In order to present the possibility of using the SWE model to simulate and predict a flood event due to river embankment failure in a real-topography urban area, a flash flood simulation was performed on the fictitious embanked developed area described by Twaróg [11] and Twaróg and Kostecki [12]. The geometry of the flow area is presented in Figure 7. It is consists of a floodplain (515 m long and 340 m wide) located along a river and a 50 m long breach in the river's embankment. The process of embankment failure was not analyzed in the simulation. The length of the breach was assumed to remain constant. Its natural crest level was assumed at ground level (*i.e.* a sudden and total embankment collapse was assumed). The flow in the river was ignored. A constant water level was assumed in the river, 3 m over the breach crest. The calculations reported in this paper used a constant value of the Manning friction coefficient equal to $0.033\,\mathrm{m}^{-1/3}\mathrm{s}$ in the whole flow area.

Terrain information is indispensable to create a two-dimensional numerical simulation of a flood flow through a real-topography area. It can be obtained from a digital elevation model (DEM), part of the DTM. The relief of the floodplain and the shape of the embankment breach analyzed in the simulations were defined by Twaróg [11] and Twaróg and Kostecki [12] in the form of series of point coordinates x, y, z. The DEM was calculated on the basis of this set with a resolution of 0.5 m using the Surfer 8.0 software (Golden Software). The inverse distances procedure was applied with an exponent of 2. The morphology of the river channel was excluded

Figure 7. Test case 2: geometry, topography and the numerical mesh for test cases 2a (a) and 2b (b) (dimensions and elevation in metres)

from the DEM because the flow in the river was not investigated in the numerical simulation of the flash flood due to embankment failure. The DEM of the floodplain is presented in Figure 7. During the flow simulation, the ground level was linearly interpolated inside each computational cell using the DEM data.

Two simulations were carried out in order to assess the influence of buildings on the flood parameters: with (test case 2b) and without (test case 2a) buildings. The geometries and meshing of flow areas for test cases 2a and 2b are presented in

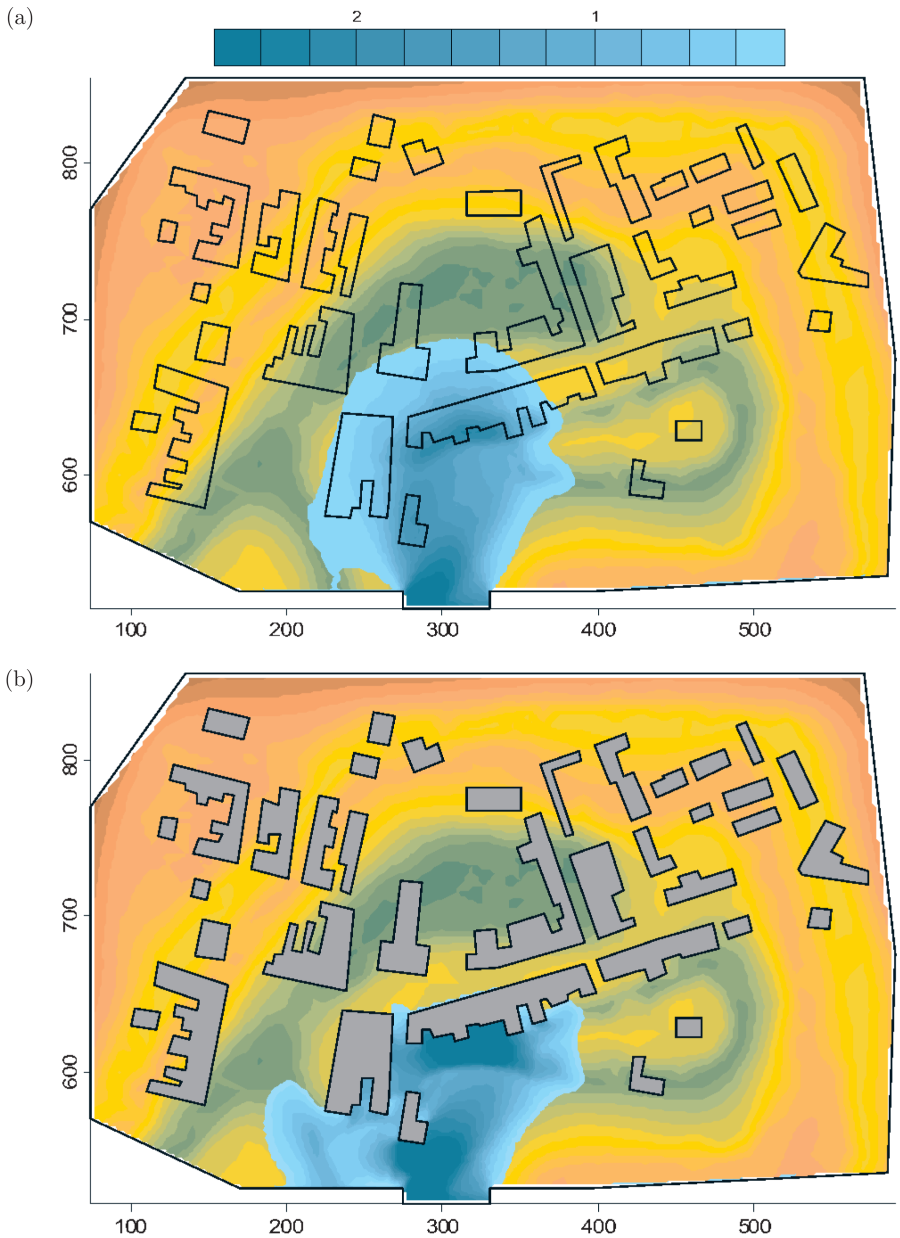

Figure 8. Test case 2: computed contours of water depth after $t = 30$s of simulation for test case 2a (a) and 2b (b) (dimensions and depth in metres)

Figures 7a and 7b, respectively. The same DEM was used in both simulations. In order to simulate flooding due to river embankment collapse, the calculation domain was covered with an unstructured triangular mesh composed of 15 278 computational cells of 5m side length in the case of flow through an undeveloped floodplain and 17 327 cells for the urban flooding event (Figures 7a and 7b). The mesh used in the numerical simulation of an extreme flow in an urban area was locally refined between buildings to better represent the complex structure of the flow in the city. The length of sides

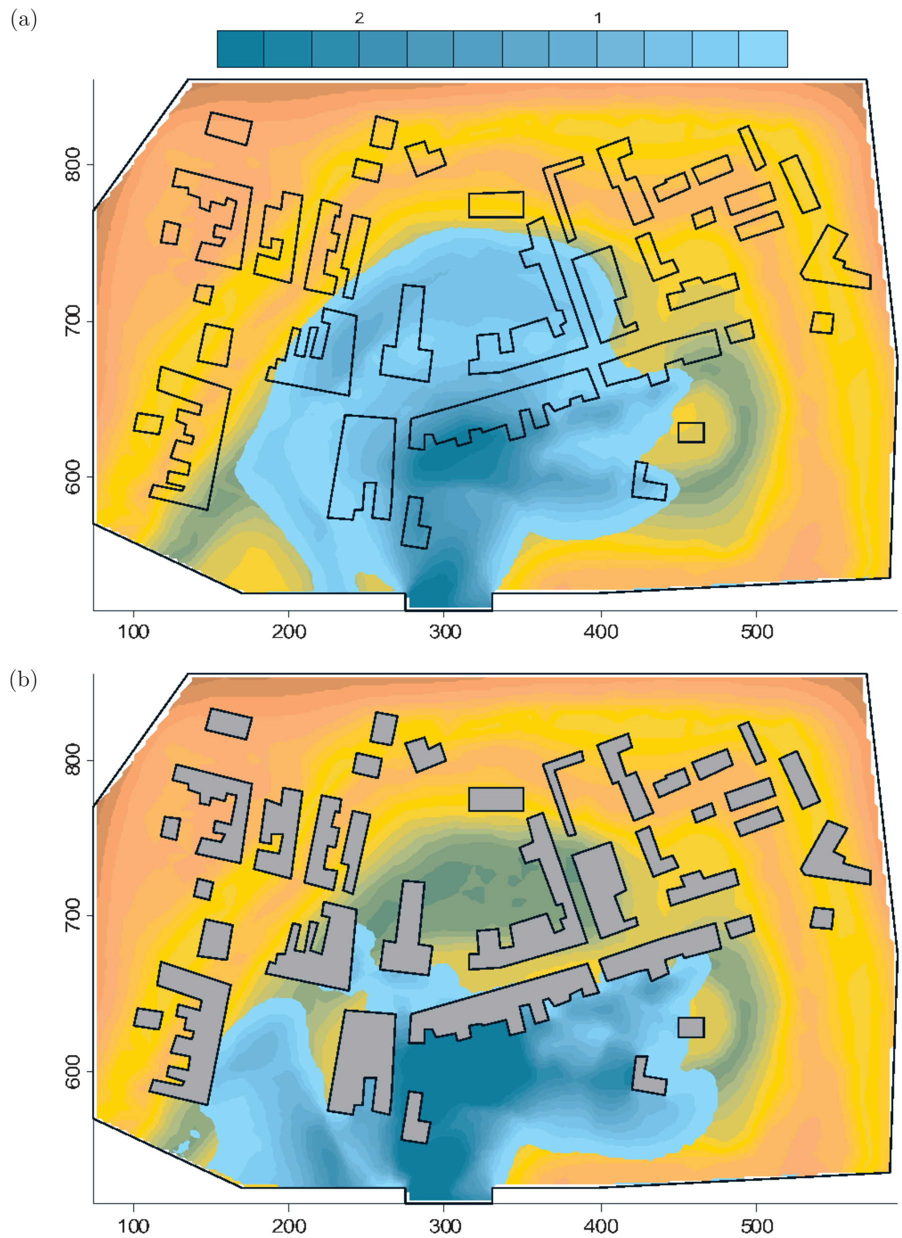

Figure 9. Test case 2: computed contours of water depth after $t = 60$s of simulation for test case 2a (a) and 2b (b) (dimensions and depth in metres)

of computational cells around each building was 3m, increasing to 6m near the flow area's boundaries.

A constant water level of 200m – corresponding to 3m above the ground level defined in the DEM at this location – was imposed at the open boundary of the embankment breach. The remaining floodplain boundaries were treated as closed. The areas occupied by buildings were excluded from the flow domain: closed boundaries were introduced at the buildings' edges. The information about build-

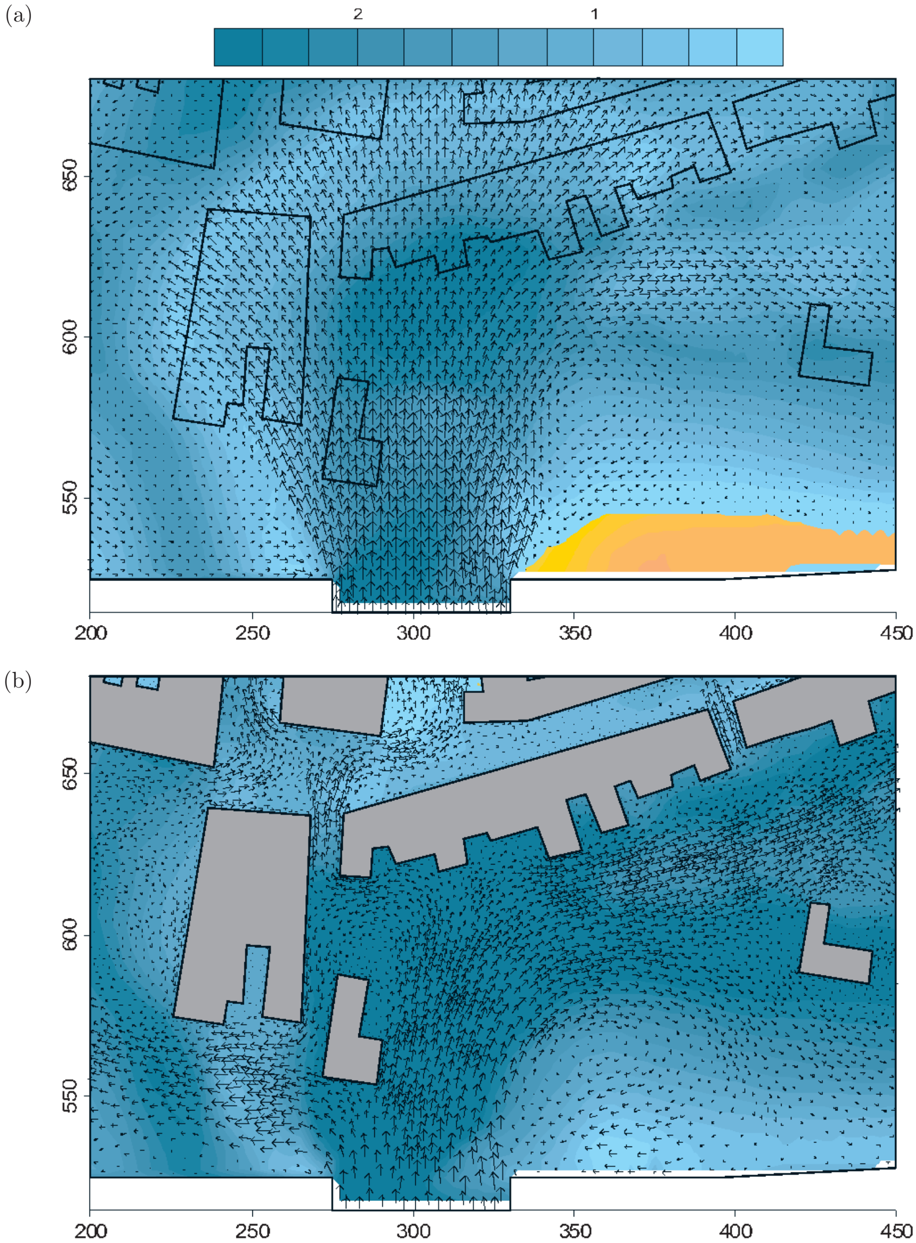

Figure 10. Test case 2: detail of computed contours of water depth and flow directions after $t = 180$s of simulation for test case 2a (a) and 2b (b) (dimensions and depth in metres)

ings location was obtained from the DTM prepared by Twaróg [11]. The calculations were carried out with a time step $\Delta t = 0.1$s and the total simulation time was 300s.

The calculation results of flood simulations in developed and undeveloped floodplain are presented in Figures 8 till 14. They were prepared in a form that may be useful in assessing the influence of urban development on the main flood parameters and creating maps of the inundation risk (identification of hazard zones).

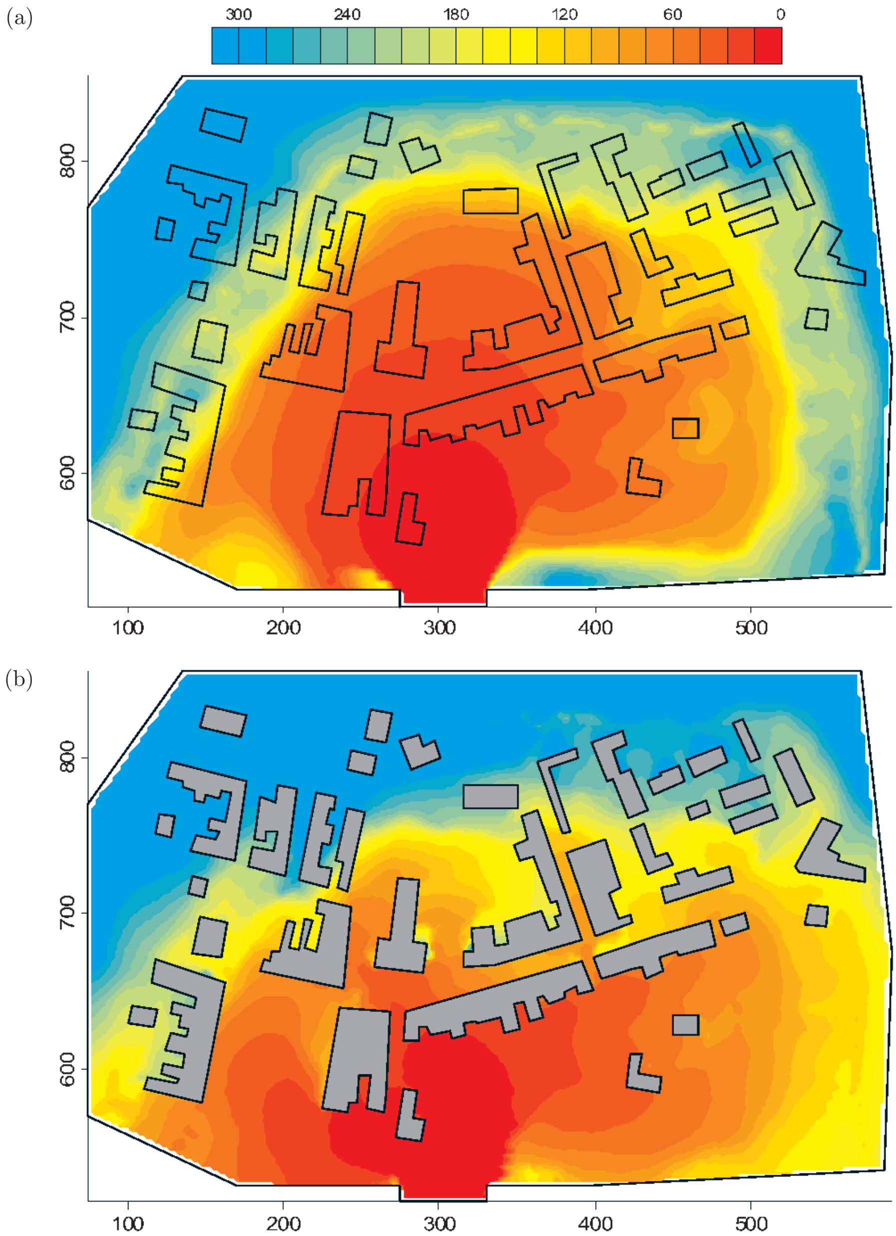

Figure 11. Test case 2: computed contours of water wave front arrival time for test case 2a (a) and 2b (b) (dimensions in metres, time in seconds)

First two pictures (see Figures 8 and 9) show two time steps of the flood's evolution. The contours of water depth on the area's topography background are presented for two moments in time ($t = 30$s, $t = 60$s) after the river embankment failure. The extreme flood wave propagates adequately to the location of the embankment breach and the relief of the floodplain. The buildings considerably reduce the speed of the flood wave's front, increasing the water depth upstream of the buildings and modifying the extent of the inundation area. Such information about

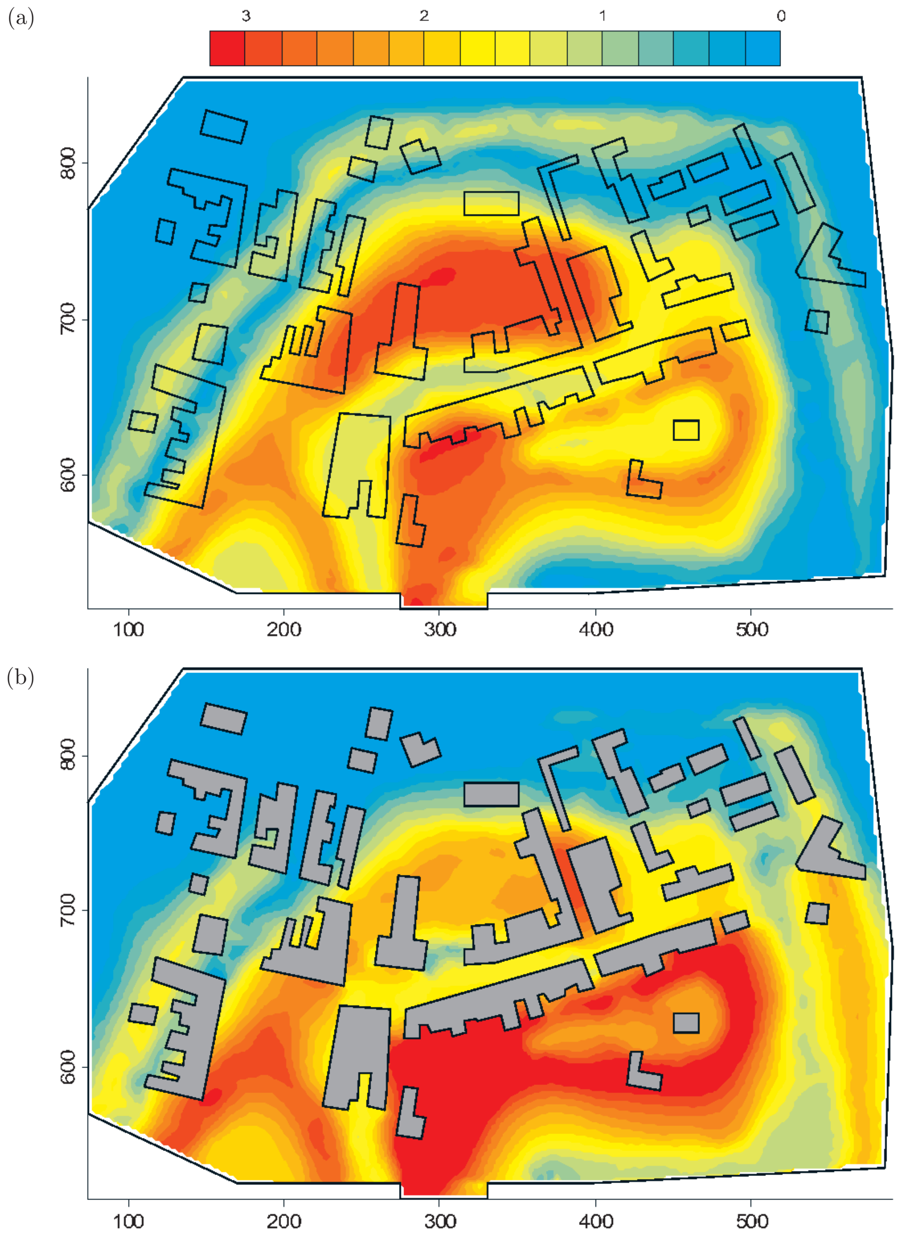

Figure 12. Test case 2: computed contours of maximal water depth for test case 2a (a) and 2b (b) (dimensions and depth in metres)

time evolution of the flood zone and water depth can be used to develop emergency plans, *etc.*

Details of the depth contour and the flow's direction near the embankment breach for time $t = 180$s are shown in Figure 10. A substantial increase in water depth is visible in front of the first row of buildings. Moreover, development has altered the flow's direction with local flow acceleration along the streets. Flow circulations and velocity reductions can be also observed in some regions near the buildings. The

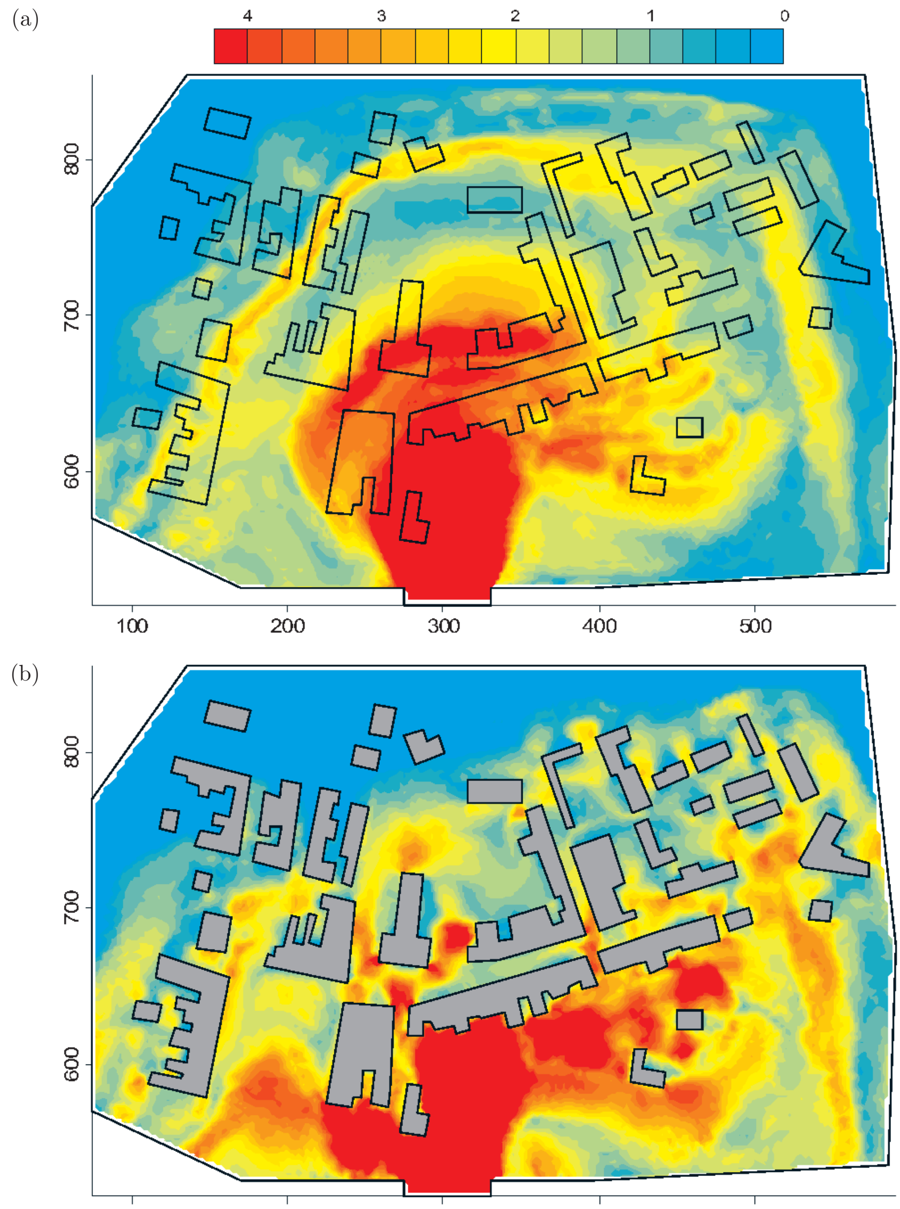

Figure 13. Test case 2: computed contours of maximal flow velocity for test case 2a (a) and 2b (b) (dimensions in metres, velocity in metres per second)

information about potential local changes in depth and flow velocity in an urban area during extreme flooding can be used to assess a flood's impact on buildings' structure, *e.g.* hydrostatic, hydrodynamic, erosion, debris and buoyancy actions (*cf.* Kelman and Spence [18]).

Information about the arrival time of a flood wave's front moving across an inundated area is very important in flood risk management. A comparison of the arrival time for developed and undeveloped floodplain is presented in Figure 11. The

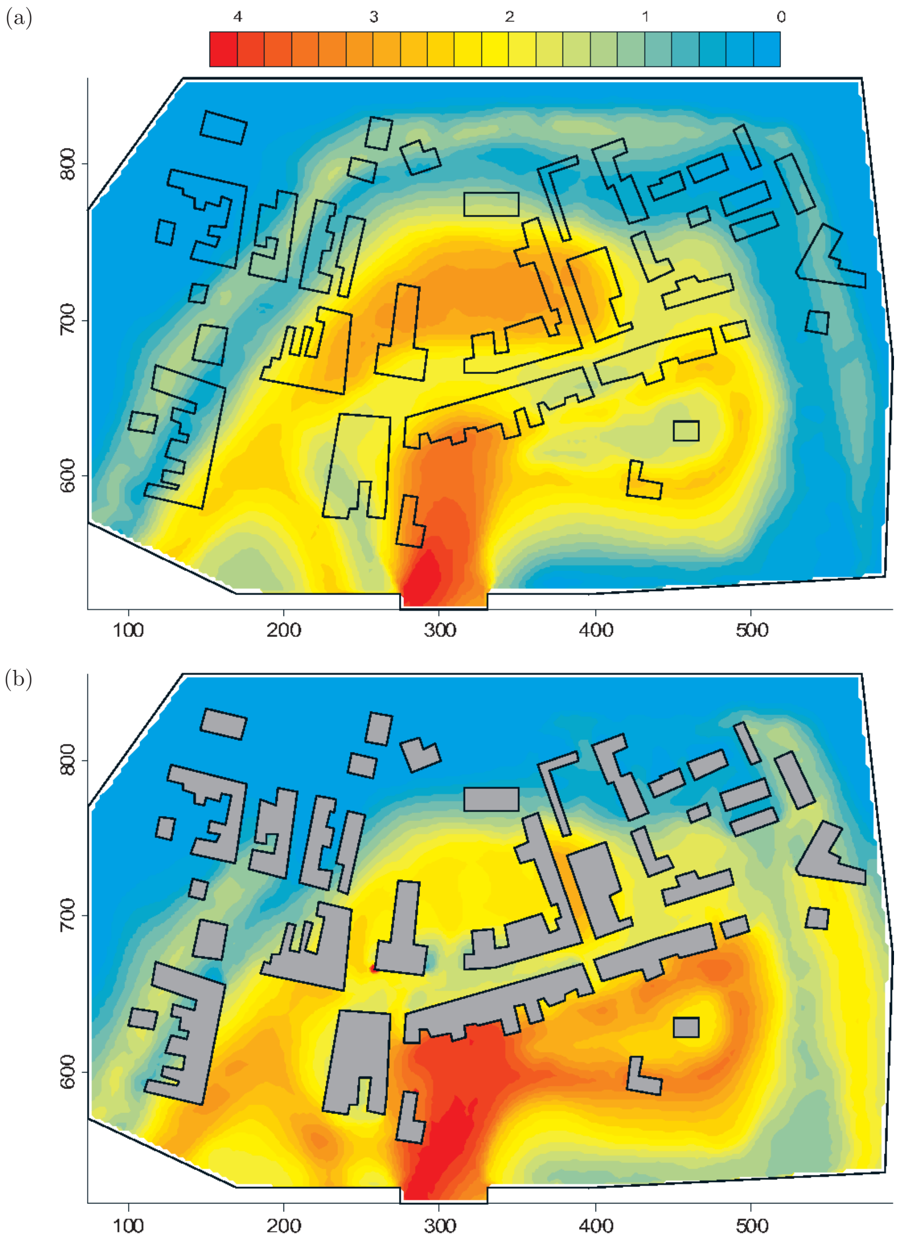

Figure 14. Test case 2: computed contours of maximal flow specific energy for test case 2a (a) and 2b (b) (dimensions and energy in metres)

time was calculated assuming that the wave front's minimal depth is greater than 0.05m. In general, buildings increase the water front arrival time at some floodplain points located in the urban area. It is a result of the flow direction change due to the buildings' walls and increased flow resistance. Therefore, the differences in arrival time between the two flooding scenarios depend strongly on city configuration.

The basic hydrodynamic parameters of the flood are presented in Figures 12, 13 and 14: the distribution of maximal water depth, the absolute velocity value and the specific energy in the whole inundation area. The predicted data set allows creating inundation maps for the floodplain and controlling development in potential flood zones. Information about potential flood parameters may be useful not only for water and civil engineers, flood managers and local authorities, but also for insurers to determine premiums for property and economic losses [19].

5. Conclusions

Two numerical simulations of an extreme flood inundation of an urban area resulting from river embankment failure have been presented. The results of the first simulation have been compared with laboratory measurements, with quite good agreement observed. The rapidly changing surface water flow in the city model has been adequately represented with the numerical solution, proving that a mathematical model based on SWE is capable of simulating extreme flooding of developed as well as undeveloped areas.

The second simulation has been a case study simulation of flash flood in the embanked developed area of real topography after embankment failure. A rich data set of flood parameters has been presented and analyzed for both developed and undeveloped floodplains. The predicted information about the inundation's spatial and temporal evolution is useful in creating maps of flood hazard zones. The knowledge of potential flood parameters is helpful in analysis and mitigation of the flood risk, protecting the cities and enhancing public safety.

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