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Modeling Results : Case Study of Belo Horizonte
Coupled 1D-2D flood modelling
.

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SWITCH Deliverable Briefing Note Template

SWITCH Document **Deliverable D1.2.11** **Modeling Results : Case Study of Belo Horizonte**
Audience The document was prepared for an audience both inside and outside the SWITCH consortium. For consortium members it gives an overview on how the optimization techniques by using multi-objective genetic algorithm can be used together with an urban drainage computational engine to solve urban flooding. It also demonstrates the potential to use this optimization technique in complex urban drainage networks that can take a considerable amount of computation resources and time, by using parallel computation and clusters. External audience consists of mainly academic fellows that are currently working on similar subjects.
Purpose The purpose of the document is to review the progress in the implementation of coupled 1D-2D flood modelling in Belo Horizonte case study/
Background This document contributes to the SWITCH approach proposed during the project. The document is an application of tools developed for urban flood modelling under SWITCH project.
Potential Impact The document presents the results of the on-going research work that aim to contribute to the development of a set of tools for coupled one dimensional sewer network model and two dimensional urban flood model which can help municipalities to developed flood hazard mapping, identify flood vulnerable areas and evaluate effect of appropriate measures to reduce urban flood problems.
Issues Not applicable
Recommendations The document consists of a description of the application of developed tools for 1D-2D coupled urban flood modelling for the city of Belo Horizonte.

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Case Study of Belo Horizonte
A Coupled 1D-2D Flood Inundation Model for Urban Flood Simulation

SUMMARY

Pluvial flooding in urban areas drained by storm sewer networks is characterized by surcharge-induced inundation. Urban inundation models need to reproduce the complex interaction between the sewer flow and the surcharge-induced inundation to make reasonable predictions of the likely flood damage in urban areas. A 1D-2D model coupling the storm sewer model SWMM5 and a newly developed non-inertia 2D overland-flow model has been developed to simulate the interaction between the sewer and overland flows. The 2D model uses an Alternative Direction Implicit numerical procedure in combination with iteration procedure. The employed solution algorithm is unconditionally stable, however, the time step is limited for accuracy and it is controlled by the use of iteration to home in on an accurate solution at each sweep. The interaction between the two models is bidirectional, and the interacting discharges are calculated according to the water level differences between the flows in the sewer network and aboveground. The paper describes the newly developed non-inertia 2D overland-flow model, the coupling of the 2D model with a sewer network (SWMM5) model and the application of both models on a sub-catchment in Belo Horizonte.

1. INTRODUCTION

Urban flood modeling is undertaken using a hydrological model to determine the runoff for a rainfall event (or time series) over the urban surface, and a hydraulic model to simulate the flow through the urban drainage network and aboveground.

The function of a sewer network is to convey household and industrial wastewater, and surface runoff from impermeable surfaces due to localized rainfall, to appropriate locations for treatment and disposal. Sewer systems are not always capable of carrying all the runoff caused by heavy rainfall due to the limited capacity of the inlets and conduits. The inadequate capacity of the drains and the heavy precipitation can therefore be one of the main causes of urban flooding. As a result inundation takes place as excess water, including the runoff that is prevented from entering the drainage network due to the surcharged flow coming out of the manholes, becomes overland flow draining towards low lying areas and possibly drainage inlets that are under-capacity. Such flooding can cause large damage to residential and commercial buildings and public and private infrastructure.

Many models have been developed for assessing the hydraulic performance of drainage networks. These include both advanced commercial software such as MOUSE (DHI Group 2008a) and InfoWorks-CS and open source software like the Storm Water Management Model (SWMM) developed by the U.S. Environmental Protection Agency (Rossman et al. 2005); see Hsu et al. (2000). Most of these models commonly use the water stage-volume curve to determine the flood depth caused by manhole surcharge. Although some models can replicate the interaction with surface channels it is generally the case that the models cannot properly simulate the complex phenomenon of the interaction between the flows in the sewers and aboveground.

Numerical models of overland flow have been applied to a number of practical problems of interest in Engineering, including overland hydrology, open channel management and surface irrigation (Garcia-Navarro and Brufau 2006). These types of numerical models are particularly interesting for the simulation of flood waves and their interaction with existing structures in urban flood modeling.

The one dimensional hydrodynamic modeling approach has been widely used in modeling flood flows due to its computational efficiency, ease of parameterization and easy representation of hydraulic structures in dealing with flows in large and complex networks both of conduits underground and of channels on the surface. However, one-dimensional (1D) models neglect some important aspects and suffer from a number of drawbacks when applied to floodplain flows including the inability to simulate lateral diffusion of the flood wave, the discretization of topography as cross sections rather than as a surface and the subjectivity of cross-section location and orientation (Hunter et al. 2007, Kuiry 2010). The 1D assumption for modeling two-dimensional (2D) surface flow may be insufficient in many urban areas where the flow paths on the surface are often complicated to define because of crowded buildings, houses and roads (Mark et al. 2004).

The recently emerged 1D-1D and 1D-2D coupled models for simulation of urban flooding can be classified in two categories; research based application such as SIPSON (Djordević 2005), SIPSON/UIM (Chen et al. 2007) and commercial packages such as MOUSE-MIKE 21 and Sobek Urban (Leandro et al 2009). The potential and limitations of both kinds of coupled models have been studies (Mark et al. 2004 and Leandro et al 2009). These studies suggest that 1D surface models are economical, robust and preferred alternatives as long as the flood water remain in the street profile and as long as the overland flow paths can be identified. However, during heavy flooding the 1D approximation may be insufficient and the use of 2D models to describe the surface flow is preferred.

The development and use of coupled 1D-2D models for urban flood modeling are driven by at least two factors. The first is the need to assess adequately the performance of storm or combined sewer systems and any associated surface flooding in order to predict the likely flood damages and to provide decision makers with information to take appropriate measures. The other factor is the development of 2D flood models in providing better predictions of the extent of flooding with respect to the depth-averaged Navier-Stokes equations. For these reasons, current modeling of urban flood flows utilises a combination of one and two dimensional hydraulic models. This allows those elements that are essentially one-dimensional, such as drains, culverts and channels to be modelled explicitly, whilst modeling the wider overland flows with a two-dimensional schematization. Results from the hydrological rainfall-runoff model are extracted and input to the hydraulic model at specified manholes (junction nodes) in the drainage network. When the capacity of the pipe network is exceeded, excess flow spills into the two-dimensional model domain from the manholes (Swan *et al.* 2007).

In general terms, 2D models are computationally more demanding when compared to 1D models and the application of 1D-2D coupled models are rather limited. 2D models without the convective momentum terms tend to be stable and use larger time steps than the full 2D models. Hence, researchers and practitioners tend to use 2D models with reduced complexity.

The aim of this paper is to describe the scheme used in developing the 2D flood model which employs an Alternative Direction Implicit numerical procedure in combination with iteration procedure, the way it is coupled with a sewer network model and to demonstrate its performance on two case studies.

2. METHODOLOGY

A new non-inertia 2D model is developed and coupled with SWMM5 to simulate the complex nature of the interaction between surcharged sewer and flows associated with urban flooding.

The non-inertia 2D model represents the urban topography by the ground elevations at the centers and boundaries of cells on a rectangular Cartesian grid and determines the water levels at the cell centers and the discharges (velocities) at the cell boundaries. The alternating direction implicit finite difference procedure is used to solve the governing equations. The schematization of the topography coupled with the way the governing equations are solved allows a good representation of small-scale topographical elements in urban environment including defined flow paths such as road networks and channels.

2.1 Sewer Network Model

The Storm Water Management Model (SWMM5) developed by the United States Environmental Protection Agency (USEPA) (Rossman et al. 2005) is used to simulate flows in storm sewer system.

SWMM5 solves the conservation of mass and momentum equations (the Saint Venant equations) that govern the unsteady flow of water through a drainage network of channels and pipes by converting the equations into an explicit set of finite difference equations and using a method of successive approximations with under relaxation to solve them. In implementing this procedure, SWMM5 uses a constant relaxation factor of 0.5, a convergence tolerance of 0.0015m on nodal heads, and limits the number of trials to four (Rossman 2006).

In SWMM surcharge occurs when all pipes entering a node are full or the water surface at the node lies between the crown of the highest entering pipe and the ground surface. Flooding is a special case of surcharge which takes place when the hydraulic grade line breaks the ground surface and water is lost from the sewer node to the aboveground system.

2.2 The Overland flow model

The 3D Navier Stokes equations describing the flow of water in flood plains are developed based on the classical concept of conservation of mass and momentum. It is generally accepted that the unsteady flow of water in two-dimensional space may be described by the shallow water equations (Garcia-Navarro and Brufau, 2006). The system of 2D shallow water equations are obtained by integrating the Navier Stokes equations over depth and replacing the bed stress by a velocity squared resistance term in the two orthogonal directions.

2.3 Continuity and momentum equation

The continuity equation for the 2D flood plain flows is formulated as in Eq. (1).

$$\frac{\partial h}{\partial t}+\frac{\partial (hu)}{\partial x}+\frac{\partial (hv)}{\partial y}=0 \qquad (1)$$

Neglecting eddy losses, Coriolis force, atmospheric pressure, wind shear effect and lateral inflow, the momentum equation can be written as in Eq. (2)and (3).
In the x-direction

$$\frac{\partial (hu)}{\partial t}+\frac{\partial \left(hu^2\right)}{\partial x}+\frac{\partial (huv)}{\partial y}+gh\frac{\partial H}{\partial x}+gC_f u\sqrt{u^2+v^2}=0 \qquad (2)$$

In the y-direction

$$\frac{\partial (hv)}{\partial t}+\frac{\partial (huv)}{\partial x}+\frac{\partial \left(hv^2\right)}{\partial y}+gh\frac{\partial H}{\partial y}+gC_f v\sqrt{u^2+v^2}=0 \qquad (3)$$

where h is the water the water depth; H is the water level, u and v are the velocities in the directions of the two orthogonal axes (the x and y directions) ; g is the acceleration due to gravity, and the coefficient C_f appearing in the friction terms is normally expressed in terms of the Manning n or Chézy roughness factor (Garcia-Navarro and Brufau, 2006).

The first term in Eq. (2) and (3) represent the local accelerations, the second and third terms represent the convective acceleration, the fourth terms represent the pressure forces and the last terms represent the frictional force. Various flood flow models can be constructed, depending on which terms in the governing equations are assumed negligible in comparison with the remaining terms. Hunter et al. (2007) reviewed the application of simplified spatially-distributed models for predicting flood inundation. It was noted that whilst neglecting inertia terms lead to local inaccuracies, reduced complexity 2D schemes have been tested successfully against analytical solutions, results from physical or alternative numerical models and measurements from actual flood events. Two-dimensional flow over inundated urban flood plain is assumed to be a slow, shallow phenomenon and the convective acceleration terms can be assumed to be small compared with the other terms and therefore can be ignored.

Expressing the velocities in terms of the discharges and using Chézy roughness factor, the simplified momentum equations can be written as:
In the x-direction

$$\frac{\partial}{\partial t}\left(\frac{Q}{Z_Q}\right)+\Delta Y g\frac{\partial h}{\partial x}+g\frac{Q}{C^2 Z_Q^2}\left(\left(\frac{1}{\Delta Y}\frac{Q}{Z_Q}\right)^2+\left(\frac{1}{\Delta X}\frac{R}{Z_R}\right)^2\right)^{0.5}=0 \tag{4}$$

In the y-direction

$$\frac{\partial}{\partial t}\left(\frac{R}{Z_R}\right)+\Delta X\, g\frac{\partial h}{\partial y}+g\frac{R}{C^2 Z_R^2}\left(\left(\frac{1}{\Delta Y}\frac{Q}{Z_Q}\right)^2+\left(\frac{1}{\Delta X}\frac{R}{Z_R}\right)^2\right)^{0.5}=0 \tag{5}$$

where h is the water level; Q and R are the discharges in the directions of the two orthogonal axes (the x and y directions) ; ΔX and ΔY are the grid spacing in the X and Y directions; Z_Q and Z_R are the water depths at the cell boundaries, g is the acceleration due to gravity, and C is the Chezy friction factor.

2.4 Numerical Model

A finite difference method is implemented for the numerical solution of the governing equations. The PDEs of the governing equations are transformed to difference equations on a regular Cartesian grid. A two point forward spatial and temporal difference scheme is adopted based on a uniform time step $\Delta t = t^{n+1}-t^n$, where n is time step counter.

2.5 Solving the governing equation

The alternating direction implicit finite difference (ADI) procedure is used to solve the governing equations for flood flows. In the ADI algorithm the solution procedure is split in such a way that in one direction the conservation of mass and conservation of the direction-corresponding momentum are solved, and, after that, in the other direction, the conservation of mass is again solved but now with the conservation of momentum introduced in that direction. For this purpose, the time step is divided into two parts such that flow in one direction is defined at the half time step and in the orthogonal direction at the whole time step. Therefore the equations are solved sequentially in the x and y directions in two half time steps, $t^{n+1/2}$ and t^{n+1}, respectively. Schematic representation of ADI algorithm is shown in Figure 1.

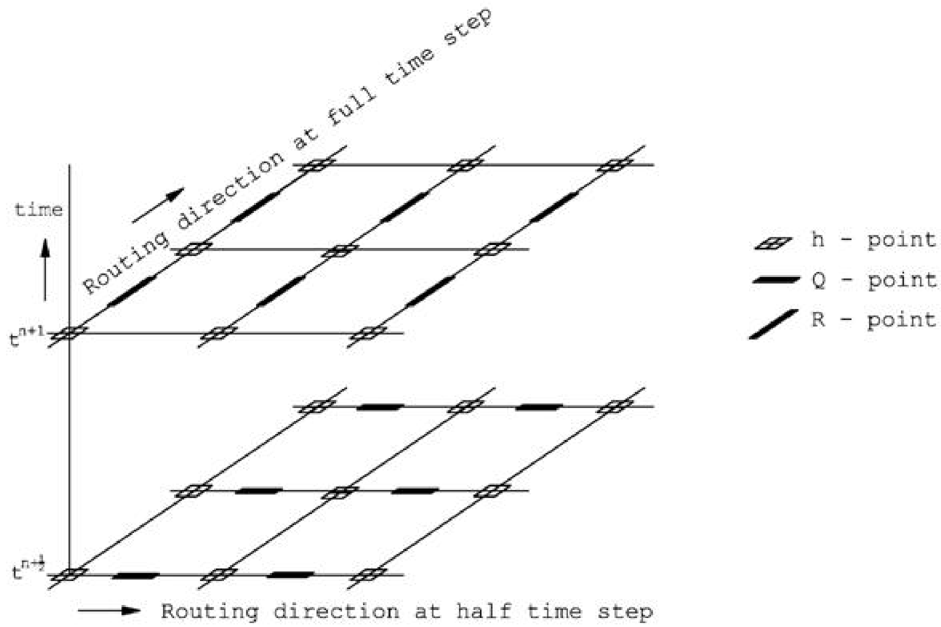

Figure 1: Schematic representation of alternating direction flood routing during each half time step.

2.6 Coupled 1D-2D model

2.6.1 Model linkage

In this study, two distinct models are combined for simulating the flow dynamics in sewer networks and on aboveground surface.

The hydrological rainfall-runoff process and routing of flows in drainage pipes are carried using the 1D sewer network model EPA SWMM. When the capacity of the pipe network is exceeded, excess flow spills into the two-dimensional model domain from the the sewer network nodes and routed using the non-inertia 2D overland flow model. Both models use different numerical schemes and time steps with the discharge through manholes adopted as the linkage of the models. The source code of SWMM is modified such that the surcharge in the sewer network is represented in terms of hydraulic head rather than overflow volumes. The interacting discharges are determined by weir or orifice equations by taking account of the hydraulic head at the sewer network nodes and the aboveground surface for every time step of the sewer network model and treated as point sinks or sources, correspondingly, in the 2D model within the same time interval.

The SWMM model only consists of the sewer network with connecting nodes like manholes, basins and outlet. The 2D model consists of urban surface with specified grids for computation, where the flood water comes from and goes to the sewer system. The models are executed individually and linked by exchanging information at connecting nodes at each time step of the sewer network model. In this study, the grids containing the manholes are considered as the locations where the interactions occur. The time step used in SWMM is also regarded as the timing for model linkage.

2.6.2 Interacting discharge

The bidirectional interacting discharge is calculated according to the water level difference between the sewer network nodes and the aboveground surface. In this method, during flooding, water is assumed to pond in the manhole and therefore the water level in the manhole can be greater than the water elevation in the overland surface. The upstream and downstream levels for determining the discharge are defined as $h_U = max\{h_{mh}, h_{2D}\}$ and $h_D = min\{h_{mh}, h_{2D}\}$ respectively, where h_{mh} is the hydraulic head [m] at manhole and h_{2D} is the water surface elevation [m] on the overland grid.

The ground levels of the sewer network nodes reflect the point values on the top of the nodes where as the grid cell elevations in the 2D model are usually computed as the mean elevation of the topography included within each grid cell. Inconsistency between the ground levels of the sewer network nodes and the 2D grid cell elevations is often present when the connecting nodes are located at local peaks or depressions inside the grids. This inconsistency causes inaccuracies in calculating the interacting discharges and makes the model result sensitive to the DEM. To minimize such inaccuracies the crest elevation Z_{crest} of a manholes is assumed to be equal to the elevation of the grid cell where the manhole is located.

The interacting discharges are calculated using the equations for a free weir, submerged weir and orifice depending on the upstream and downstream water levels (Chen et al. 2007). The free weir equation is adopted when the crest elevation Z_{crest} is between the values of the upstream water level h_U and the downstream water level h_D, as shown in Figure 6. The discharge is calculated using:

$$Q = (h_{2d} - h_{mh}) C_w W \sqrt{2g}\,(h_{2d} - Z_{crest})^{3/2} \tag{6}$$

If both the upstream and downstream water levels are above the crest elevation of the manhole then either the submerged weir or the orifice equation is used to calculate the interacting discharge. If the upstream water depth above the manhole crest is greater than the area of manhole divided by the weir crest width $(h_U - Z_{crest}) < A_{mh} / W$ then the submerged weir equation (Eq. (7)) is used; otherwise the manhole is considered fully submerged and the orifice equation is used (Eq. (8)):

$$Q = sign[h_{mh} - h_{2D}] C_w W \sqrt{2g}\,(h_U - h_{crest})(h_U - h_D)^{1/2} \tag{7}$$

$$Q = sign[h_{mh} - h_{2D}]C_o A_{mh}\sqrt{2g}\,(h_U - h_D)^{1/2} \tag{8}$$

where C_o is the orifice discharge coefficient, C_w is the weir discharge coefficient and W is the weir crest width. The negative value meant drainage flow from surface into sewer.

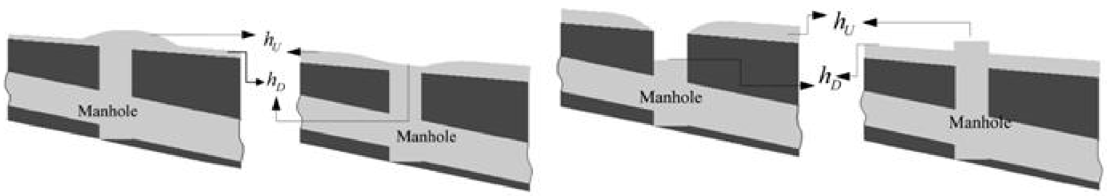

Figure 2: (a) Free weir linkage and (b) Submerged weir or orifice linkage.

3. APPLICATION TO THE CASE STUDY

3.1 Belo Horizonte Drainage Network

Belo Horizonte (BH) is the capital of the State of Minas Gerais, which in economic terms (gross product) is the third among the 26 Brazilian states. The city lies at 20° South latitude and 44° West longitude (Figure 6) and has an altitude of 750 to 1,300 meters. It is located in a mountainous region of tropical soils that originated from the decomposition of metamorphic rock. Tropical highlands weather predominates in this area, with average yearly rainfall of 1,500 mm and average yearly temperature of 26°C. The rainy season lasts from October to March, when 90% of the total yearly rainfall occurs. The highest monthly average rainfall (315mm) takes place in December. Typical rainfall intensities are also relatively high (e.g.: 200 mm/h in the case of a 10-year return period event with 5 minutes duration; 70 mm/h for the 1h and 50-year return period event). Mean relative humidity reaches 50% during winter and 75% in summer.

Figure 3: Location of the municipality of Belo Horizonte

BH has 2,227,400 inhabitants with a population density of 6,900 inhabitants/km^2. It is a planned city, built in 1898 to become the capital of the state. The total area of the municipality is 330 km^2. BH is inserted into a metropolitan area; the RMBH (Belo Horizonte Metropolitan Area), gathering 33 distinct municipalities, with an area of 9, 179 km^2 and 3,900,000 inhabitants.

The Belo Horizonte territory locates at two main catchments (Arrudas creek and Onca creek catchments), each representing at about 50% of the total municipal area. Part of those catchments locates at neighbourhood municipalities: Contagem, upstream of Belo Horizonte, and Sabará and Santa Luzia, downstream of Belo Horizonte. There are no rivers in the municipal territory, although Arrudas and Onça are direct tributaries of the Velhas River, with a total drainage area of about 40,000 km2, which itself is the tributary of the Sao Francisco River, the longest one entirely within Brazilian territory (approximately 600,000 km^2 of drainage area).

Stormwater management has been entirely under the responsibility of the BH municipality since the city foundation. Traditional storm water systems prevail in the city, although experiences with detention ponds exist since the 50s. There are at about 4,300 km of roads all of them equipped with gutters, inlets etc. The municipal database on drainage infrastructure keeps details about 64,000 inlets (gullies), 11,500 manholes, 1,100 outflow structures (outfalls), and almost 770 km of stormwater sewers. There are some 700 km of perennial creeks in the municipal area. Part of those creeks have been lined, most of them as culvert concrete channels. The length of lined channels reaches near to 200 km.

The creek lining policy, which prevailed up to the 90s, was mainly justified by the following rationale:

- Lining is required for increasing the flow velocity and the channel conveyance, reducing the flood risk;
- Lining makes easy the implantation of interceptor pipelines and the so called sanitary roads;
- Lining makes easy the creek maintenance;
- Health risk due to directed contact with polluted waters may be reduced by creek lining;
- Inhabitants of riparian zones usually ask for creeks to be lined

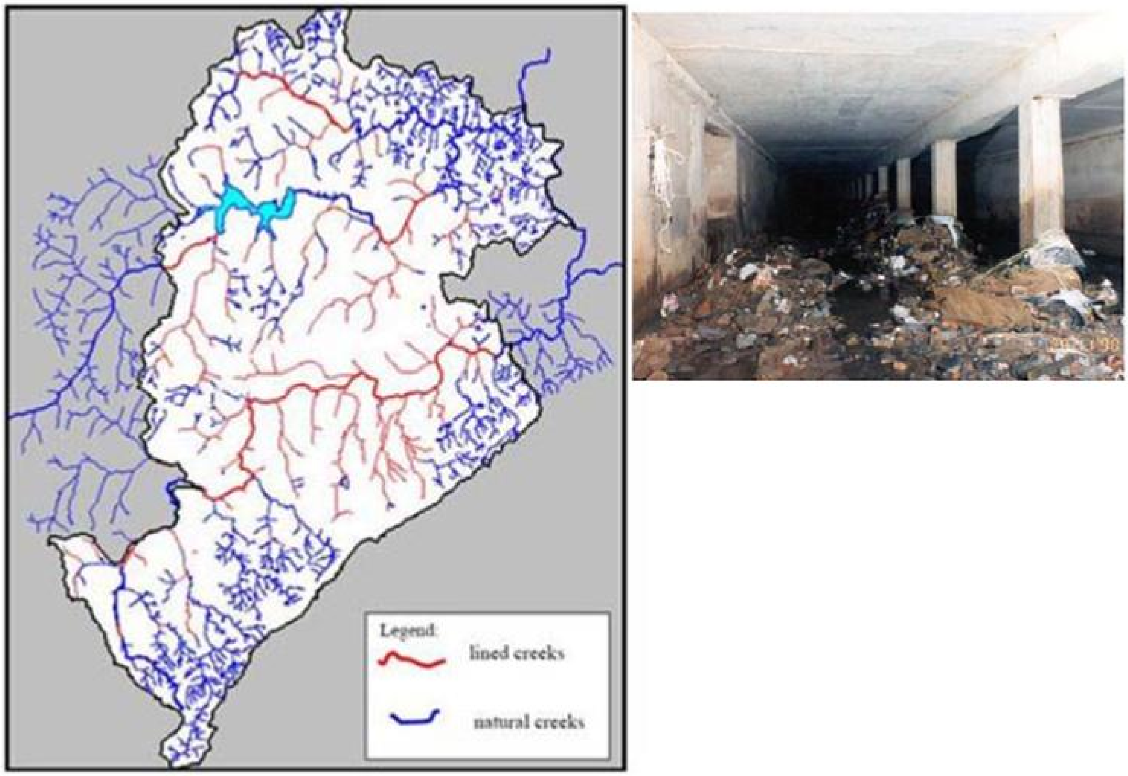

Figure 4: Belo Horizonte drainage network and lined channel photo

The apparent simplicity of stormwater management, as perceived almost during all the last century, led to the use of very simple design methods for storm water systems. Synthetic models were used which do not require observed data to calibrate parameters (e.g.: rational method and synthetic unit hydrograph). Since observed data were considered as not necessary for storm water management, during all the last century the BH municipality did not invest in monitoring stream discharges or water quality parameters. One of the consequences of those approaches is high uncertainty in hydrologic design. A similar oversimplification also prevailed in hydraulic design. Complex flow conditions, including the effects of stream confluence, flow transitions or unsteady flow, were infrequently regarded and model simulations of these conditions were rarely done.

3.2 Building and Running the Coupled Model

A portion of Bello Horizonte drainage network was setup in SWMM5 to be coupled with overland flow model. The topography and the information to build the model were provided by the local partners in the learning alliance. The information was Pre-processed in ArcGis to define the sub-catchments and main streams. This part of the network corresponds to the Vendanova Catchment located in the North side of Belo. See Figure 5.

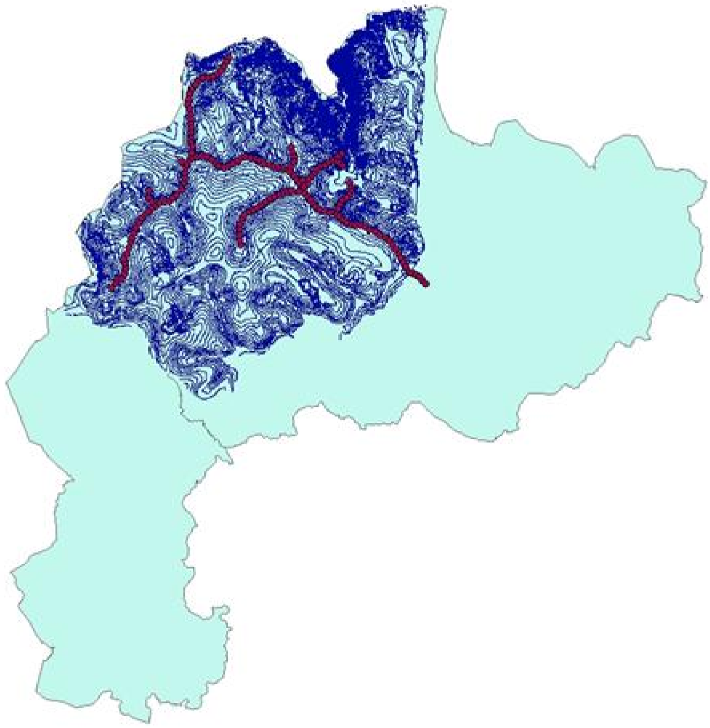

Figure 5: Layout of the Drainage network build for the case study (Vendanova Catchment).

The network is composed by 168 pipes and 169 nodes (see Figure 6). A rainfall event corresponding to a precipitation of 60 mm over a period of half an hour was used for the simulation. Figure 6 shows the network and digital elevation model used in the study.

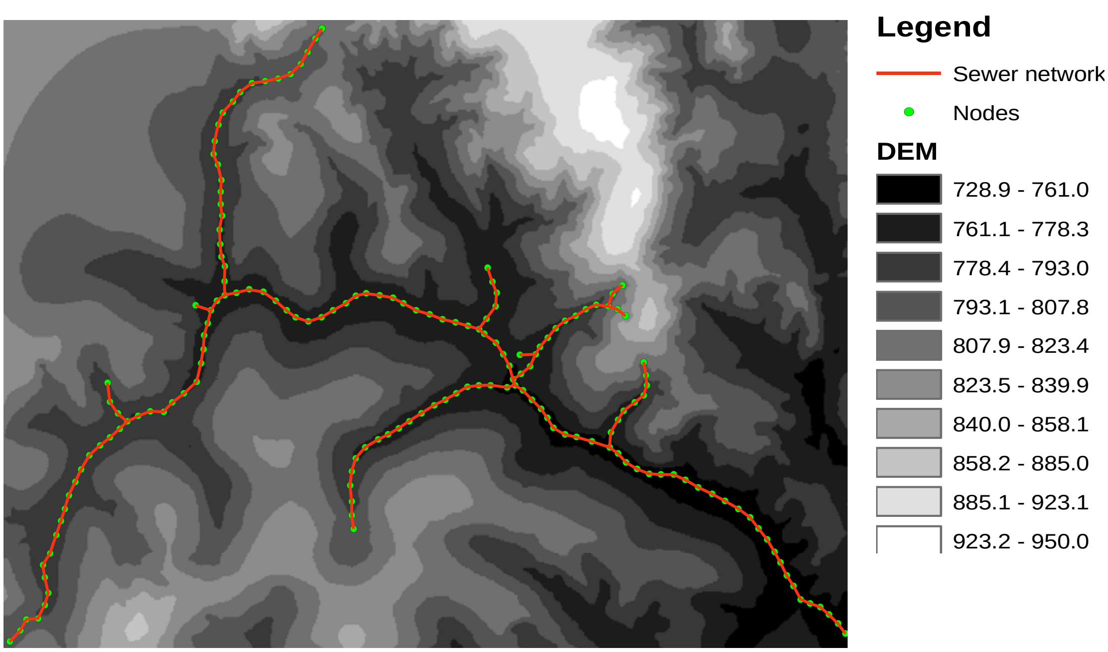

Figure 6: Network schematization in Mouse

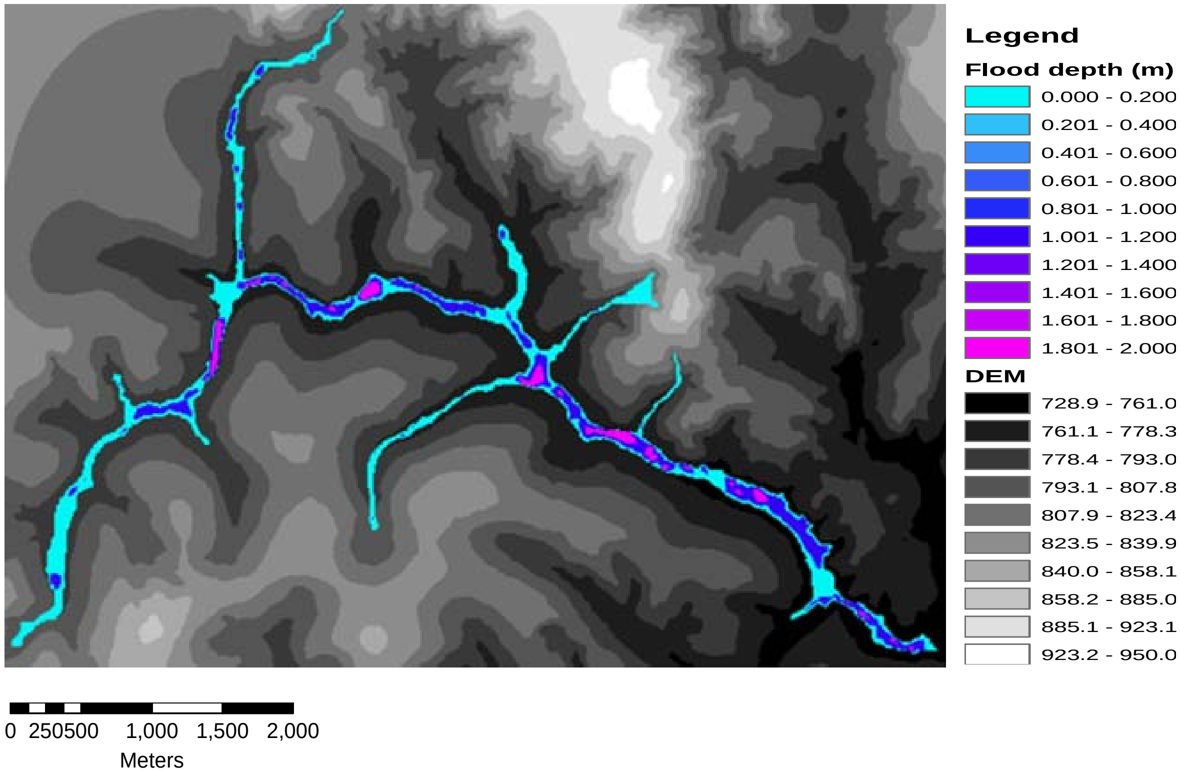

Figure 7: Maximum Flood depth for the 1 hour storm

Figure 7 shows the maximum flood depth that can be expected for a rainfall event of 60mm in half an hour. Here a synthetic rainfall event is used just to show the capability of the coupled model to produce flood maps. To use the model for simulation of real flood events, the model needs to e calibrated and validated against an observed rainfall event with field measurements of flood depths. The coupled model can also be used to evaluate the effect of proposed measures to reduce flood risk.

4 ACKNOWLEDGEMENT

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