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Zonification of areas with inundation risk by means of mathematical modelling in the Rosario region, Argentina

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Abstract River flood valleys are being subjected to urbanization processes in big cities of the world. Therefore, urban planning strategies based on water resources management must be established to avoid environmental deterioration. The mapping of risk zones by means of mathematical modelling is a tool to carry out control policies of land use and occupation. This paper describes a two-dimensional mathematical model and the determination of inundation risk maps for two rivers in the Rosario region of Argentina. The mapping was made over both Saladillo and Ludueña rivers, for floods of return periods of 50, 100 and 500 years. The studied zones embraced an area of 7000 ha, with a population of 500 000 inhabitants. Based on the results, state and local governments are planning non-structural rules with the associated legislation. The paper concludes that it is important to use flow simulation models for urban planning strategies and water resources management.

INTRODUCTION

The Rosario region in the south of the Santa Fe state, Argentina, is crossed with rivers and channels. The upper reaches of these water courses are located in a rural zone with non-defined beds. Downstream, they flow in more defined beds and with permanent water draining. The flood plain includes big inhabited areas like Rosario and Villa Gobernador Gálvez which are highly urbanized. The overflows produce both serious material effects and the loss of human life. To solve this flooding problem, public organizations have built several structural works. Now, it is necessary to define non-structural rules to determine the legislation that permits both flood plain land occupation and use. The laws will contribute to the regional planning of water resources. In this context, it is proposed that configuration of inundation risk maps associated with return periods by means of flow simulation mathematical modelling be created. In this paper, a cell model is used, its results capable for subcritical flow simulation (or slightly supercritical) with superficial propagation, over river and flood plain. Concerning urban zones routing, the model was developed to simulate flood propagation both on streets (major system of urban drainage), and inside close conduit networks (minor system).

MODEL FORMULATION AND GOVERNING EQUATIONS

The model hypothesis agrees with the flow characteristics in the regional water courses, where rivers and streams have a main permanent bed and a temporarily occupied flood plain. This configuration allows division into a certain number of interconnected cells. Moreover, the model uses a numeric resolution algorithm in which no restrictions are related to the links between the cells. Hence, the model provides alternatives of both topological and hydraulic discretization. Figures 1 and 2 present a representation of a reach of a water course and an urban reach shown for major and minor systems. The equations that govern the model are those of continuity and discharge between linked cells.

Continuity equations

It is supposed that the whole ***i*** cell corresponds to a characteristic water level z_i which is assumed in the cell centre (Fig. 3). Also, it is assumed that the water surface is horizontal between the borders of the cell and it is z_i. The two fundamental hypothesis are:

- the volume V_i of water stored in cell ***i*** is directly related to its water level z_i: $V_i = V(z_i)$;

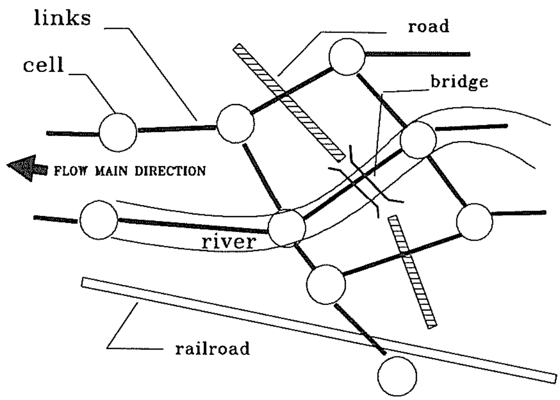

Fig. 1 Discretization of cells.

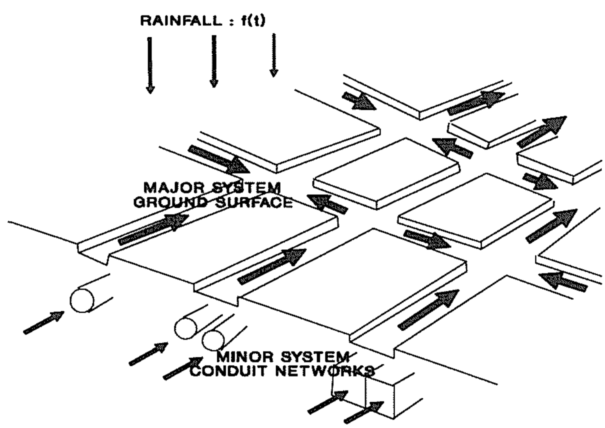

Fig. 2 Urban runoff.

- the $Q^{n}_{i,k}$ discharge between two adjacent cell i and k at one time given as $n\ \Delta t$ is a function of the energy levels: $z^{n}_{i} + \alpha_i V^2_i/2g$ and $z^{n}_{k} + \alpha_k V^2_k/2g$.

The forces originated in the local acceleration are neglected. It is possible to arrive at the continuity equation written under the differential form used by the model (Cunge, 1975):

$$A_{S_i} \frac{\mathrm{d} z_i}{\mathrm{d} t} = P_{i(t)} + \Sigma_i^k Q^{n}_{i,k}\ (z_i, z_k) \tag{1}$$

There are many equations (1) as well as cells i in the model, as well as unknown water levels $z_i(t)$. The solution to this system exists and is unique if the set of initial conditions z_i $(t = 0)$ is prescribed (Cunge, 1975). Once this set is known, the functions $z_i(t)$ and $Q_{i,k}$ may be computed numerically. Moreover the boundary conditions varying in time must be prescribed.

Laws of discharge between the cells

Simple river type links The $Q_{i,k}$ expression is deduced by discretization of the momentum equation for flow with inertial forces negligible and considering the Strickler–Manning resistance formula:

$$Q_{i,k} = \mathrm{sign}(z_k - z_i)\ \frac{K}{\sqrt{\Delta x}}\ \sqrt{|z_k - z_i|} \tag{2}$$

where z_i and z_k are the water level in the cells; K is the conveyance coefficient, defined by $K = k\ A\ R^{2/3}$ with k the roughness coefficient of Strickler $(1/\eta)$, A cross-section, and R hydraulic radius (Fig. 4); and Δx is the fixed distance between the cell centres.

Weir type links This link type is used to represent links between cells where there is a physical limit between them: cells separated by highway embankments, roads, bridges, links between main course and flood plain, etc. For the discharge calculation the expression of the wide crest weir (Fig. 5) is utilized:

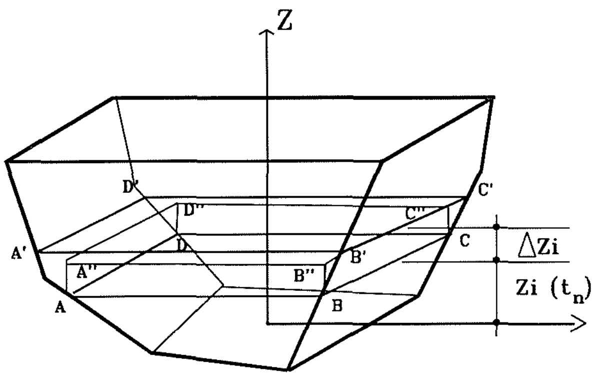

Fig. 3 Continuity equation of a cell.

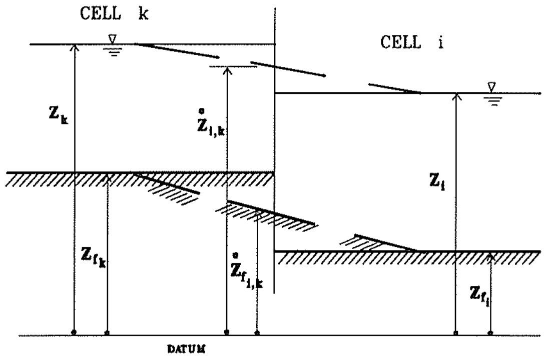

Fig. 4 River type link.

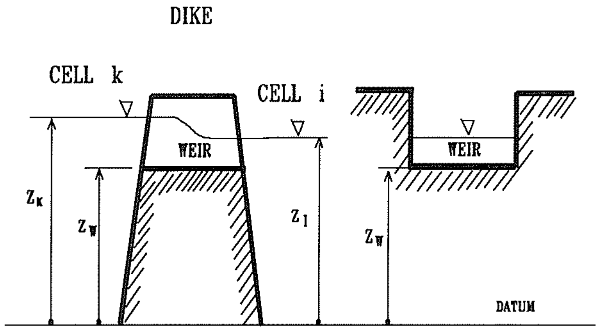

Fig. 5 Weir type link.

$$Q_{i,k} = \mu_1 \, b \, \sqrt{2g} \, (z_k - z_w)^{3/2} \qquad \text{Free flow weir} \tag{3}$$

$$Q_{i,k} = \mu_2 \, b \, \sqrt{2g} \, (z_i - z_w) \, \sqrt{z_k - z_i} \qquad \text{Flooded flow weir} \tag{4}$$

where: μ_1 and μ_2 are the coefficients discharge and b the effective weir width.

Quasi-inertial river type links The deduction of the discharge between two adjacent cells (Fig. 6) is based on the complete momentum equation. Considering slow variations of z_i, z_k and $Q_{i,k}$ through the time, it is possible to arrive at an explicit expression of the flow $Q_{i,k}$:

$$Q_{i,k} = \Phi_{i,k} \sqrt{ABS\left[\frac{z_k - z_i}{1 + \frac{\Phi_{i,k}^2}{2g}\left(\frac{1}{A_i^2} - \frac{1}{A_k^2}\right)}\right]} \tag{5}$$

Loss energy type links This type of link is capable for whose flow singularities have energy loss due to abrupt changes in the cross-section. These are present usually in sewers, collectors mouths, junctions box, etc. (Fig. 7). Considering k as the portion of velocity energy loss in a link, the expression $Q_{i,k}$ as a contraction (equation (6)) and expansion (equation (7)) are defined as:

$$Q_{i,k} = \sqrt{2g}\sqrt{\frac{z_k - z_i}{\left(\frac{1+k_i}{A_i^2} - \frac{1}{A_k^2}\right)}} \quad (6)$$

$$Q_{i,k} = \sqrt{2g}\sqrt{\frac{z_k - z_i}{\left(\frac{1}{A_i^2} - \frac{1-k_k}{A_k^2}\right)}} \quad (7)$$

Frictional conduit type links This link is used for connections between cells of close conduits (Fig. 8). The discharge is similar to the links described above. The difference from those is given for the cases where the conduit works under pressure. In these cases, the conveyance factor remains constant for values of hydraulic gradients more than for the top conduit elevation: $K = k\,A_{i,k}R^{2/3}i,k$ = constant. Also, in continuity equation (1) the value of A_{Si} is minimum, corresponding to the Preissmann slot surface (Cunge *et al.*, 1980).

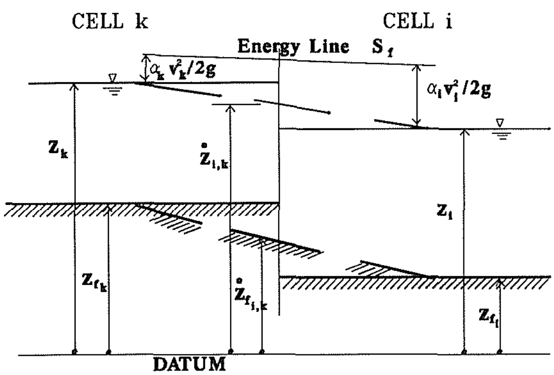

Fig. 6 Composite river type link.

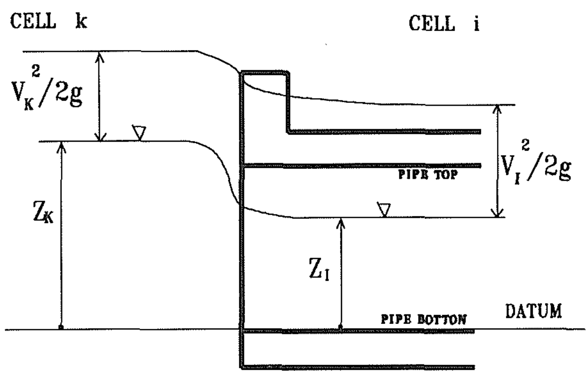

Fig. 7 Energy loss type link.

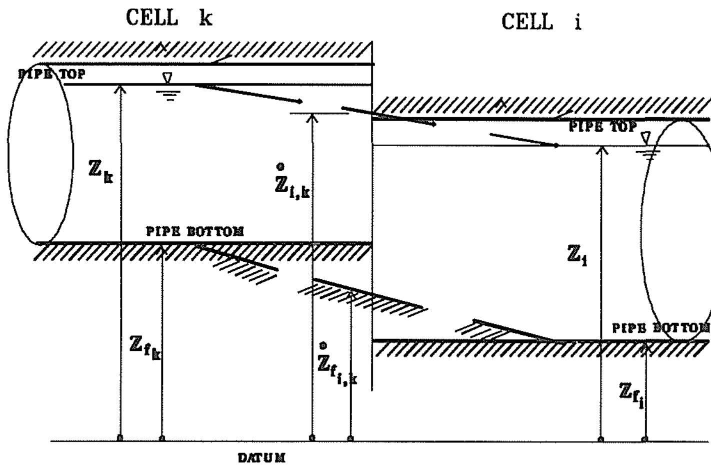

Fig. 8 Frictional link in conduit.

NUMERICAL FORMULATION AND BOUNDARY AND INITIAL CONDITIONS

For numeric formulation, the function of discharge between cells is explicit, then it is introduced in the continuity equation (1). It is assumed that the discharge $Q_{i,k}(zi(\tau),zk(\tau))$ is an intermediate discharge between $Q^{n}_{i,k}$ and $Q^{n+1}_{i,k}$ with varying between $n\,\Delta t \leq \tau \leq (n+1)\Delta t$. It has been shown that the use of an implicit method of finite difference for the numerical resolution is appropriate (Cunge, 1975):

$$A_{S_i}\,\frac{\Delta z_i}{\Delta t} = P_i + \sum_1^k Q^n_{i,k} + \sum_1^k \frac{\partial Q^n_{i,k}}{\partial z_i}\Delta z_i + \sum_l^k \frac{\partial Q^n_{i,k}}{\partial z_k}\Delta z_k \tag{8}$$

A_{Si}, P_i and $Q_{i,k}$ are known in time $t = n\,\Delta t$ and the increments Δz_i and Δz_k are unknown.

The model uses a matrix resolution algorithm based on the Gauss–Seidel method. A system of $m \times m$ equations is defined, where m is the amount of internal cells. In each step of time must be solved the matrixes in order to determine the corresponding Δz. Then, discharges $Q^{n+1}i,j$ is computed in explicit form.

There are three types of boundary conditions that the model may simulate:

(a) level given as a function of time $z(t)$;
(b) discharge given as function of $Q(t)$;
(c) relationship is given between level and discharge: $Q = f(z)$.

The model requires the specification of water levels in all cells at the initial time.

COMPUTATIONAL IMPLEMENTATION

The last version of the computational system was called CELDAS5. It is written for a series of subroutines and a main program. First, a series of auxiliary routines carry out a treatment of topographical and hydraulic information for all cells and links that

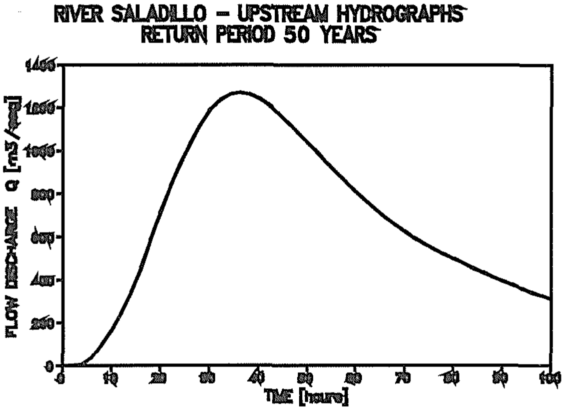

Fig. 9 River Ludueña upstream hydrograph.

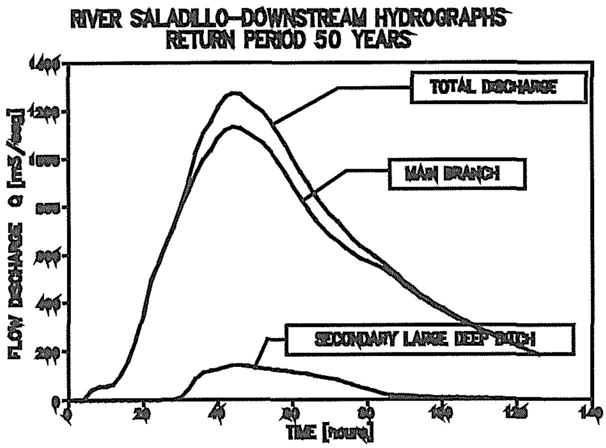

Fig. 10 River Ludueña downstream hydrograph.

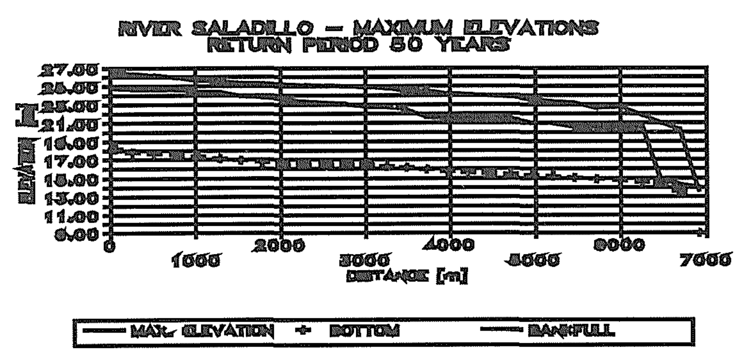

Fig. 11 River Ludueña downstream hydrograph.

compose the system. Later, the main process begins in which the program computes and stores the results in disk by mean of tabular files: water level–time in each cell; discharge, velocity, Froude's number, time in each link; hydrograph and limnigraph in incoming and salient links.

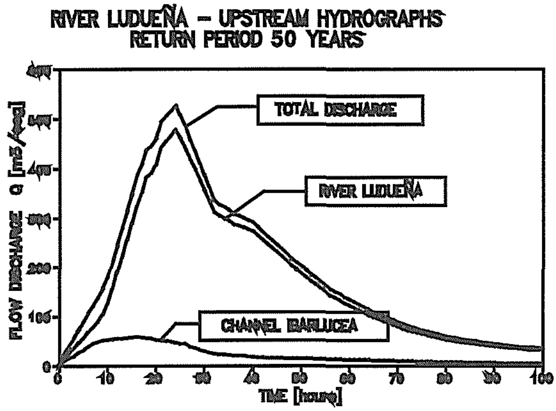

Fig. 12 River Saladillo upstream hydrograph.

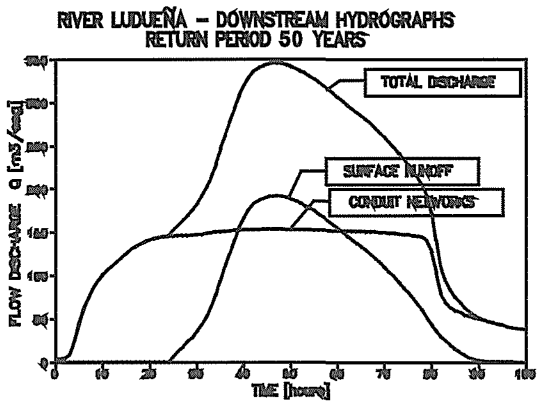

Fig. 13 River Saladillo downstream hydrograph.

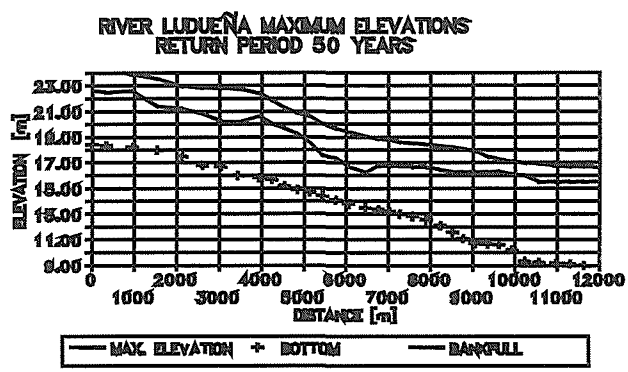

Fig. 14 River Saladillo maximum elevations.

DESCRIPTION OF THE MODELLED PHYSICAL ENVIRONMENT

The region of Rosario city is called Pampa Ondulada Argentina. Inside it there are two basins which contain that city: Saladillo and Ludueña. Both flow toward the River Parana.

River Saladillo

This river drains a 3000 km^2 basin area. The reach studied divides two cities (Fig. 15): Rosario and Villa Gobernador Gálvez. The reach length is 7 km, beginning in a rural zone and ending in a cascade of 15 m in height. The bottom medium slope is 0.7‰. The flood plain includes a total surface of 20 km^2, 70% is in a rural zone, 15% is semiurbanized, and the 15% is fully inhabited. The area studied has a population of 200 000 inhabitants. There are seven bridges with lengths from 43 m to 145 m. Near the cascade, there is a second outflow constituted by a paleostream that flows downstream of the cascade. Embankments normal to the stream are barricades for the flow due to its height, establishing "storage cells".

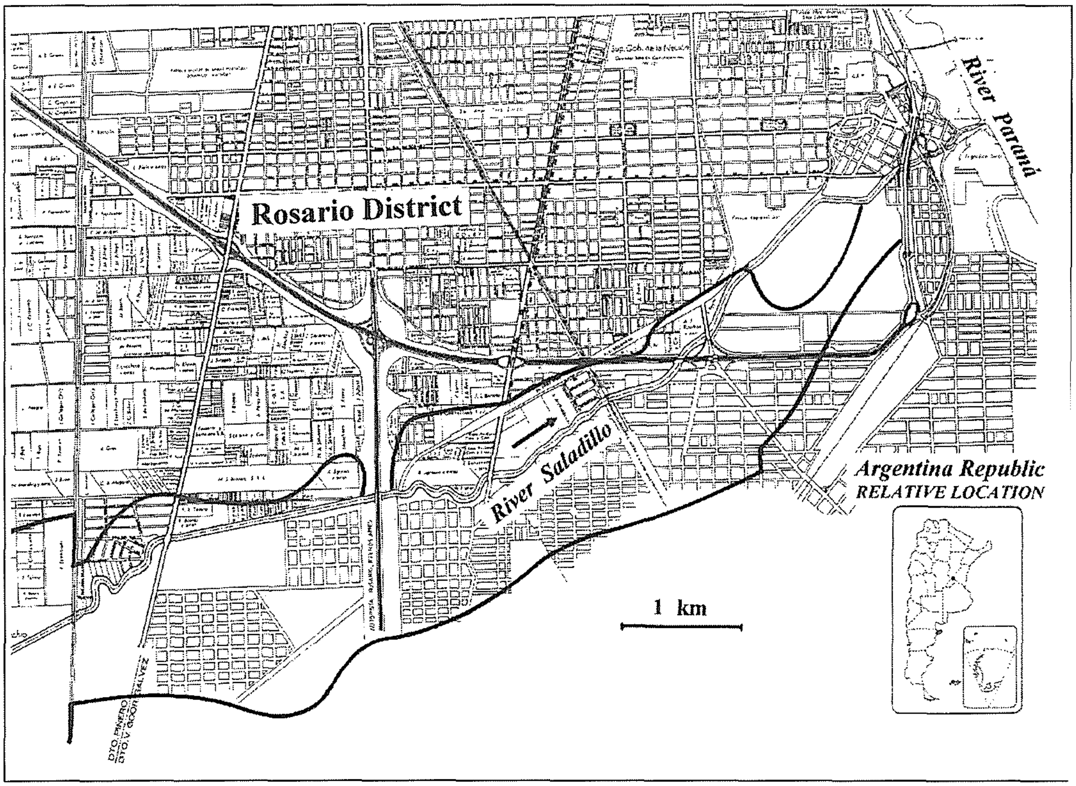

Fig. 15 River Saladillo study section.

River Ludueña

This area embraces Rosario city and Funes village (Fig. 16). The river drains a 800 km^2 basin area. The length of river and its tributaries is 19 km, with 1.2‰ medium slope. In the main course it overflows at about 80 m^3 s^{-1}. The flood plain in this study includes a total surface of around 50 km^2: 75% is rural, 15% semiurbanized and 10% is fully inhabited. There are around 300 000 inhabitants. In a reach 1.5 km long the water course is piped in five conduits. These conduits have a cross-section of 73.3 m^2, and drain in the River Parana flowing free by a channel of 0.8 km of length. The discharge maximum capacity of the conduit network is 350 m^3 s^{-1}. A retention dam has been built recently at the upstream boundary.

APPLICATIONS

The goal of the study was to analyse the hydraulic behaviour to determine maps with inundation risk both for the natural stage (without works) and projected stage with structural works. Those maps were determined for 50, 100 and 500-year return periods.

For both case the results for a 50-year return period are presented in Figs 9–16. These figures show the inflow from the high catchment, the hydrographs computed at the downstream boundary, the profiles of maximum water elevations and the inundation maps of the studied zones. For the River Ludueña the downstream hydrograph in the urban zone, routing over the major system and in the conduit network is presented.

River Saladillo

In this application, the structural works analysed were: cross-section increment, embankment construction and works to avoid the movement of the cascade upstream. The model configuration resulted in 88 cells, 30 river types and 58 valley types. The width of the river cells was 140 m. Conveyance coefficients of river links were calculated with a preliminary Manning coefficient of 0.030. Flow coefficients of bridges were calculated according to Chow methodology (Chow, 1959). Boundary conditions were water level upstream, free fall law at the two outlet points downstream. Near the zone of the cascade there is a critical regime, routing its influence upstream. River and valley cells were 250 m in length. The adjustments was made by means of previous overflows, firstly the April 1986 one (1200 m^3 s^{-1} peak flow, inundation map available). The parameters adjusted were Manning coefficients in beds and valleys, weir discharge coefficients and corrective bridge factors. The mean of statistical studies was defined as a flow peak of 1280 m^3 s^{-1} for a 50-year return period, 1488 m^3 s^{-1} for 100-year return period and 1990 m^3 s^{-1} for 500-year return period. The lag of maximum overflows is 44 h and base time of the hydrograph is larger than 100 h (Riccardi, 1994).

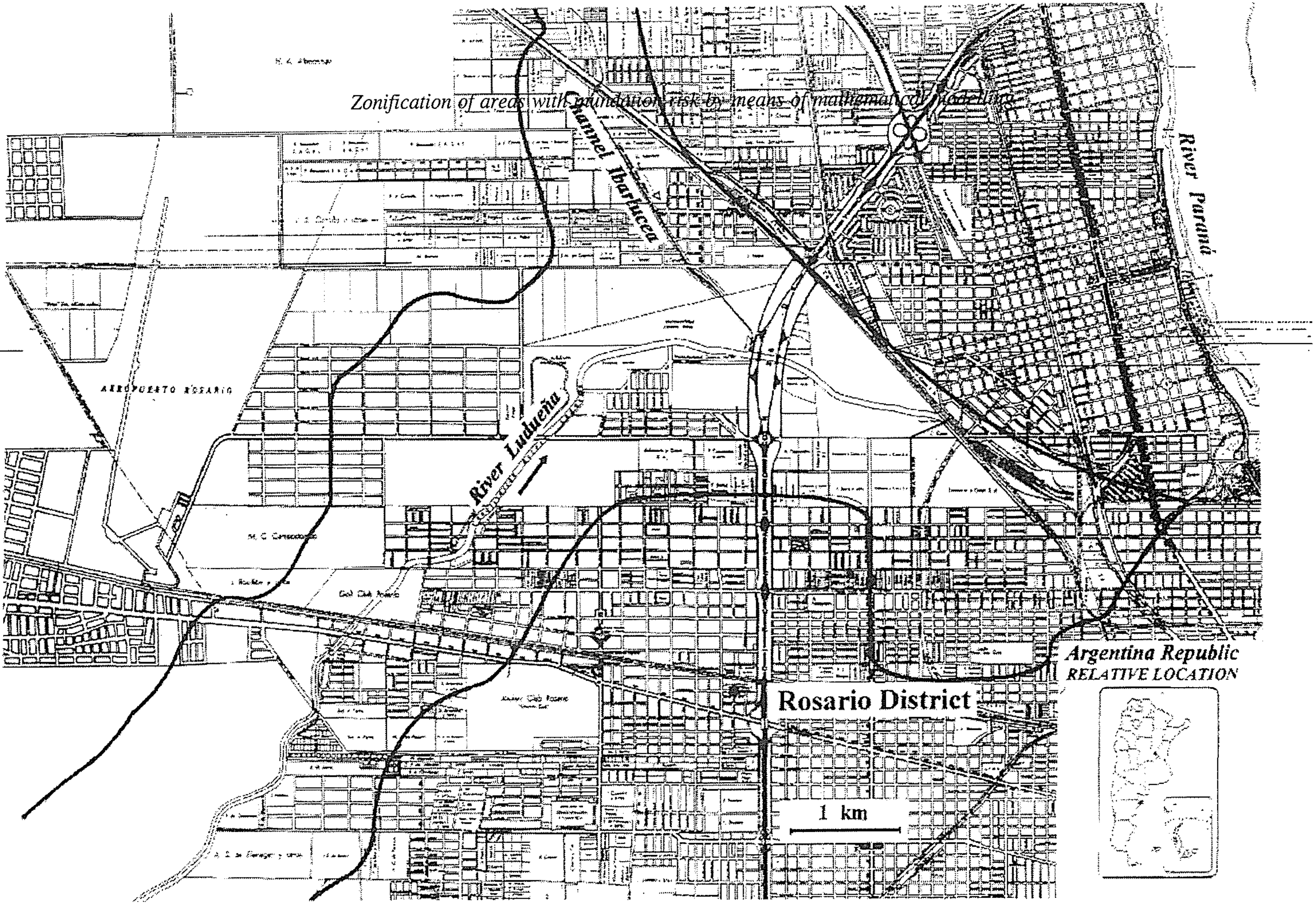

Fig. 16 River Ludueña study section.

River Ludueña

In this other application, the topological and spatial discretization was satisfied with 202 cells and 311 links. Fifty-four cells corresponded to open channel flow; 43 to conduit networks and 95 to flood plains of rural and semiurban zone and 10 to urban zone. Topographical information was considered over a grid of points of 100 m average distance. This information was digitized from 0.25 m contour maps. The hydraulic parameters considered in preliminary form: roughness, efficiency coefficients, etc. were taken into account for previous simulations (Riccardi, 1991, 1992, 1993, 1994).

The model was calibrated for different floods and mainly with one measured in 1986 with a 40-year return period. It was then run to map the probable maximum flood (PMF) as the catastrophic event. The values of the discharge peaks were 500 $m^3 s^{-1}$ for a 50-year return period, 700 $m^3 s^{-1}$ for a 100-year return period, 1000 $m^3 s^{-1}$ for a 500-year return period and 1700 $m^3 s^{-1}$ for the PMF.

RESULT USES

Based on the risk area delimitation described in this paper, state and local governments are planning non-structural rules and developing the associated legislation.

The projected legislation proposes severe restrictions of land-use for the risk zone for a 100-years return period. The inundation map for a 500-year return period was considered as the minimum risk zone. Variable restriction rules of land-use and occupation in the intermediate zone have been proposed. These rules diminish toward the limit of the inundation map for a 500-year return period.

CONCLUSIONS

The model developed is completely capable for a flow simulation with two-dimensional characteristics. It simulates satisfactorily the flow distribution on beds, flood plains, underground conduit networks and a major system of urban zones. The results of applications have demonstrated a very good approximation to the real phenomenon. Also, the model applications have been able to incorporate a powerful technological tool to the planning of regional water resources, allowing delimitation of zones with inundation risk by mean of computed results. These actions should be considered in strategies of sustainable development and hydroenvironmental equilibrium.

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