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A CELL MODEL FOR HYDROLOGICAL-HYDRAULIC MODELING

Gerardo Riccardi | Consejo de Investigaciones and CURIHAM
Facultad de Ciencias Exactas, Ingenieria y Agrimensura
Universidad Nacional del Rosario
Rosario, Argentina

A hydrological-hydraulic quasi-2D multilayer simulation model is presented. It models the dominant hydrological processes involved in the rainfall-runoff transformation in rural and urban environments. The modeling structure is based on the well-known schemes of interconnected cells, with the original addition of a large set of linking laws. These allow flow simulation in both surface runoff and networks of closed conduits. The model was developed to embrace different scales of rural and urban storm drainage from streets to river basins, and to cover different levels of detail, from river basins with a resolution of 0.10 elements/km^2 to drainage micro basins with 600 elements/km^2. The conceptual model and a synthesis of several relevant applications are presented here. The results obtained were satisfactory, and show that the real process of rainfall-runoff transformation flowing in a single layer and in two interconnected layers (surface runoff and networks of closed conduits) can be successfully modeled.

INTRODUCTION

The processes involved in the rainfall-runoff transformation in rural and urban environments have been widely studied by scientists, for a better understanding of the phenomena, and by engineers, for the hydraulic design of different structural components of drainage systems. A large number of systems exist with proven quality that allow the representation of such processes; among the most important are S11S (Abbott and Cunge,1981), MOUSE (Danish Hydraulics Institute, 1996), SWMM (Huber and Dickinson, 1998), ILLUDAS (Terstriep and Stall, 1974), OTTHYMO (Wisner, 1983). However, there are still limitations in modeling rainfall-runoff processes. Some suggested subjects of research and development are: development of reliable but not overly complicated hydraulic models, modeling with appropriate levels of detail, scale effects, modeling to small scale, integrated models, and continuous modeling (Cao et al., 1993; O'Loughlin et al., 1996). This study presents a system for hydrological-hydraulic numerical simulation for both rural and urban environments. The simulation structure is capable of simulating different levels of detail and to different scales, from small conduits to micro and macro urban basins, and rural basins.

MODEL FORMULATION

The modeling system is based on the scheme of cells originally proposed by Cunge (1975). In previous work (Riccardi, 1994, 1998) we have enlarged the field of application of this scheme. The system now allows the simulation of the rainfall-runoff processes with multidirectional and multi-layer flow dynamics. At each unit of subdivision of the surface layer it is possible to simulate precipitation, interception losses, depression storage, and infiltration. The runoff can be simulated by means of a set of discharges based on the kinematic approach of the momentum equation to an approach to the dynamic equation. This approach allows flow routing through rivers, channels, flood valleys, urban streets and networks of closed conduits. In addition the model incorporates discharge laws for bridges, weirs, storm inlets, junctions, changes of section, pumping stations, etc. The model was named CELDAS8. The governing equations are those of continuity and different simplifications of the momentum equation, which were transformed to obtain the discharge between the linked cells.

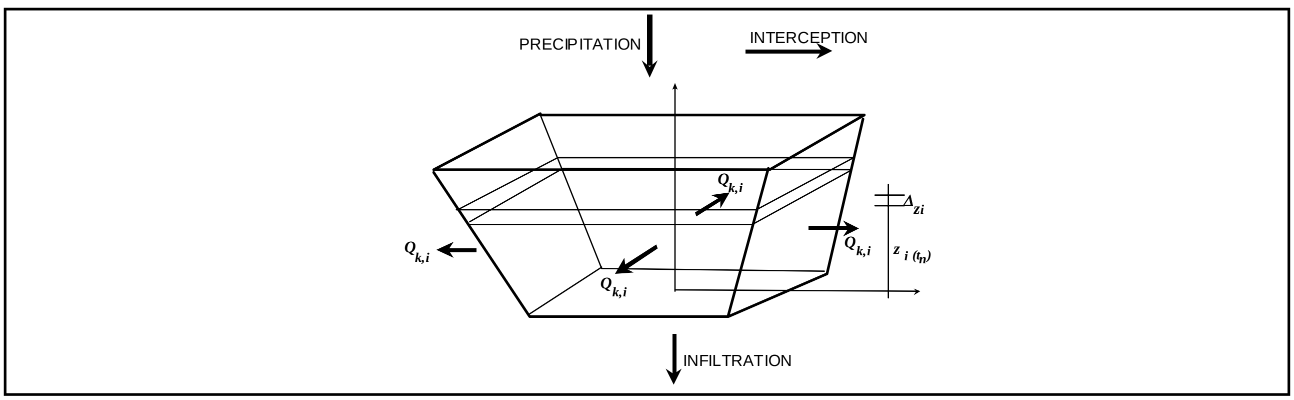

Figure 1. Continuity equation of a cell.

Continuity Equation

It is supposed that to every cell *i* corresponds a water level $z_i = z_i(t)$ the cell center (Figure 1) and that the water surface is horizontal and its value is z_i. The continuity equation can be written in differential form (Cunge, 1975):

$$A_{S_i} \frac{dz_i}{dt} = P_i(t) + \sum_{k=1}^{j} Q_{k,i}(t) \quad (1)$$

where $P_i(t)$ represents processes such as rainfall, interception, surface storage, infiltration and external exchange of flow.

Laws of Discharge Between the Cells

Simple river link

This link is used for flow with a preponderance of gravity, hydrostatic pressure, and friction forces. The discharge expression $Q_{k,i}^{(n)}$ is derived from the momentum equation assuming negligible inertia and using the Strickler-Manning resistance formula (Cunge, 1975):

$$Q_{k,i}^{(n)} = sign\,(z_k^{(n)} - z_i^{(n)})\,\frac{1/\eta\; R_{h\,k,i}^{2/3}\;\; At_{k,i}}{\sqrt{\Delta x_{k,i}}}\sqrt{|\,z_k^{(n)} - z_i^{(n)}|} \tag{2}$$

Quasi-dynamic link

This link is used when the forces originating in convective mechanisms are important and can produce large variations of wetted areas of the cross sections. The local acceleration term in the momentum equation is neglected (Riccardi, 1994, 1997a, 1997b):

$$Q_{k,i}^{(n)} = \pm\frac{K_{k,i}}{\sqrt{\Delta x_{k,i}}}\sqrt{ABS\left[\frac{z_k^{(n)} - z_i^{(n)}}{1+\left[K_{k,i}/\sqrt{\Delta x_{k,i}}\right]^2/2g\;\left(A_i^{-2} - A_k^{-2}\right)}\right]} \tag{3}$$

Dynamic link

This link is used when all the physical processes described in the momentum equation are important. Starting from the complete momentum equation, an approximate expression of second degree in the discharge variable can be derived (Riccardi, 1998):

$$a_1\; Q_{k,i}^{\;2} \;+\; a_2\;|Q_{k,i}|\,Q_{k,i} \;+\; b\;Q_{k,i} \;+\; c \;=\; 0 \tag{4}$$

$$Q_{k,i}^{(n)} \;= -b + \frac{\sqrt{b^2 - 4(a_1 + a_2)c}}{2(a_1 + a_2)} \quad \text{if } c \;<\; 0 \tag{5.a}$$

$$Q_{k,i}^{(n)} \;= -b - \frac{\sqrt{b^2 - 4(a_1 - a_2)c}}{2(a_1 - a_2)} \quad \text{if } c \;> 0 \tag{5.b}$$

Kinematic link

This link is only used when the hydrodynamic information is carried downstream (Riccardi,1998). The discharge is computed as a function of the upstream cell level:

$$Q_{k,i}^{(n)} = K_{k,i}\; S_{0_{k,i}}^{\;\;1/2} \tag{6}$$

Weir link

This link is used to represent links between cells with physical limits: cells separated by highway or railway embankments, and connections between the main stream and flood plains. The discharge formulation used is the classical expression of the wide crest weir: free (Equation 8.a) and submerged (Equation 8.b) (Cunge, 1975):

$$Q_{k,i}^{(n)} = \mu_1 b \sqrt{2g} \, (z_k^{(n)} - z_{(i)}^{(n)})^{3/2} \tag{8.a}$$

$$Q_{k,i}^{(n)} = \mu_2 b \sqrt{2g} \, (z_k^{(n)} - z_w) \sqrt{z_k^{(n)} - z_i^{(n)}} \tag{8.b}$$

Bridge link

This link is the weir link assuming bottom step zero and a discharge coefficient according to Ven Te Chow for flow through constrictions. In addition, this link can be used for big culverts (Riccardi, 1994).

Head loss or control section link

This link is suitable for flow singularities with head loss due to abrupt changes in the cross section. Two flow conditions are possible, free (Equation 9.a) and submerged (Equation 9.b)(Riccardi, 1998):

$$Q_{k,i}^{(n)} = \sqrt{2g \, (z_k^{(n)} - z_{cri}) / (Cd^{-2} \; A_{Cri}^{-2} - At_k^{-2})} \tag{9.a}$$

$$Q_{k,i}^{(n)} = \sqrt{2g(z_k^{(n)} - z_i^{(n)}) \, / \, (Cd_S^{-2} \; A_{Sc}^{-2} - At_k^{-2})} \tag{9.b}$$

Conduit link

This link is used for connections between cells of closed conduits. The continuity and discharge equations for free surface flow are of the same type as those outlined in the approaches for diffusive wave, quasi-dynamic, and dynamic (Riccardi, 1998). In flow under surcharge, the surface area in Equation (1) is calculated as a minimum value corresponding to the surface that results from the Preissmann slot (Riccardi, 1998) along the conduit.

Storm inlet link

This link is used to represent the storm inlet of closed conduit components of drainage systems. The model allows simulating curb, gutter and combination inlets, undepressed and depressed, with and without grating (Riccardi, 1998). The discharge is calculated by means of weir and hole laws, under free and submerged flow conditions:

$$Q_{k,i}^{(n)} = C_s \; \mu_l \; L_{su} \; \left(z_k^{(n)} - z_{su} \right)^{1.5} \tag{10.a}$$

$$Q_{k,i}^{(n)} = C_s \; \mu_a \; L_{su} \left(z_k^{(n)} - z_{su} \right) \left(z_k^{(n)} - z_i^{(n)} \right)^{0.5} \tag{10.b}$$

Pumping link

This link is used to represent elements of flow elevation by means of addition of external energy; which is the case of single pump or pumping stations (Riccardi, 1997b). It is necessary to specify the pumping sequence as a function of time, t_1-$Q_{k,i\,1}$, t_2-$Q_{k,i\,2}$, t_3-$Q_{k,i\,3}$,or as a function of the level at the upstream cell, $z_{k\,1}$-$Q_{k,i\,1}$, $z_{k\,2}$-$Q_{k,i\,2}$.

Numerical Formulation, Boundary and Initial Conditions

An implicit method of finite difference for the numerical solution is used (Cunge, 1975):

$$A_{S_i}\frac{\Delta z_i}{\Delta t} = P_i + \sum_{k=1}^{j} Q_{k,i}^{(n)} + \sum_{k=1}^{j} \frac{\partial Q_{k,i}^{(n)}}{\partial z_i}\Delta z_i + \sum_{k=1}^{j} \frac{\partial Q_{k,i}^{(n)}}{\partial z_j}\Delta z_k \qquad (11)$$

A_S, P_i and $Q_{k,i}$ are known at time $t = n\,\Delta t$, and Δz_i and Δz_k are unknowns. For N cells, the system is established for N unknown functions z_i of the independent variable t. The solution exists and is unique (Cunge, 1975) if the initial conditions $z_i\,(t)$ can be prescribed. The numerical solution is carried out using an algorithm based on matrix resolution by the method of Gauss-Seidel. Three types of boundary conditions can be represented: a) *water level as function of time*: $z(t)$; b) *discharge as a function of time*: $Q(t)$; and c) *a level-discharge relationship*: $Q = f(z)$. The model requires water levels at all the cells at the initial time. For dynamic links, initial discharges should be defined at such links. The term $P_i\,(t)$ in Equation (1) allows the addition or extraction of external flow at each cell. Usually it is used to represent effluents, overflows, extraction for irrigation, etc.

APPLICATIONS

Urban basin

The simulation of the so-called urban basin called “Conduit 27” located in Rosario City (Figure 2) is described (Riccardi, 1998). The basin has a 4.23 km^2 area and slopes ranging between 1 and 3 ‰. The drainage system is combined. Runoff corresponding to rainfalls of recurrence smaller than that of the design drains, flows through the combined sewer. In events of medium and large storms, the runoff drains by the combined sewer and by streets toward the storm inlets of the trunk conduit. The underground system of closed conduits consist of two trunk conduits and multiple secondary branches. The linking between the surface and underground systems consists of approximately 300 m of storm inlets. There is an 85 percent impervious area. A continuous increase of the impervious area of the basin has produced a decrease of the maximum storm that the system supports without floods from 1 to 2 years of recurrence.

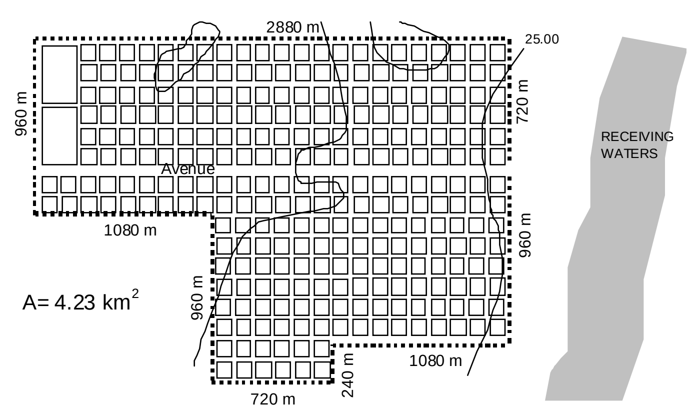

Figure 2. Urban drainage basin.

CELDAS8 was implemented with 68 cells and 114 links (Figures 3 and 4). The level of detail was 16 elements/ km^2. The calibration was made with a group of measured values of rainfall and water level at the main conduit. The calibrated Manning coefficient for runoff over tributary areas was $\eta = 0.100\text{-}0.200$. Into the secondary combined sewer the resistance was $\eta = 0.020$, and by avenue and

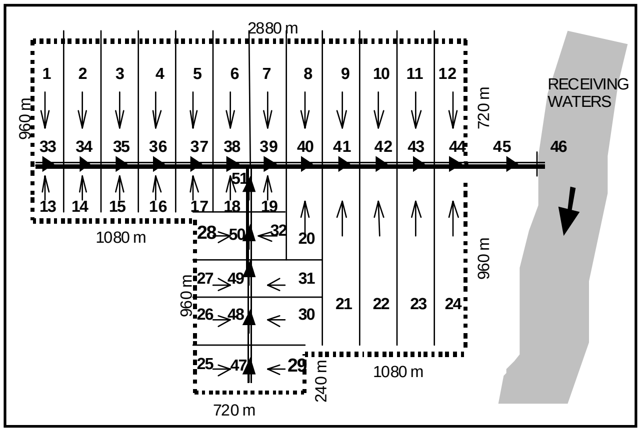

Figure 3. Topological discretization of surface layer.

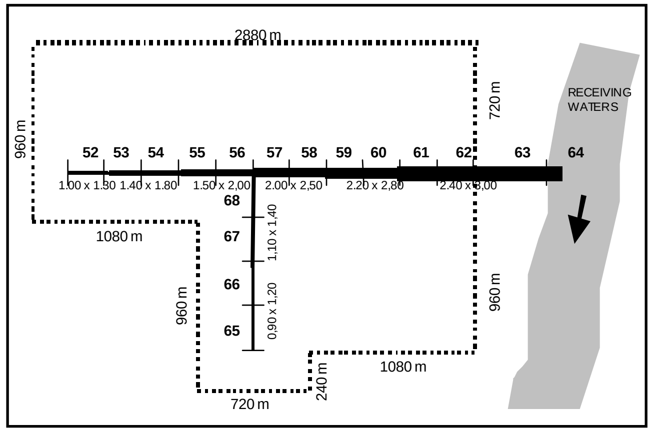

Figure 4. Primary network of closed conduits.

main conduits, η = 0.016. The discharge coefficient of the storm inlets was 0.60. The model was used to quantify the improvement of the capacity of the drainage system by means of the substitution hypothesis of 30 percent of the impervious area (sidewalks, courtyards and secondary streets) for semipervious surfaces. The new porous surfaces were considered 30 percent impervious and 70 percent permeable. In Figure 5 the net rainfall and downstream hydrographs for 5 and 10 years recurrence storms in the current state and with of substitution of impervious for porous surfaces are illustrated. The maximum water levels calculated in the trunk conduit and over the avenue are shown in Figures 6 and 7.

Determination of flood risk zones over the Ludueña River floodplain

In this application the determination of flood risk maps over the Ludueña River (Rosario, Argentina) is described (Riccardi, 1997a, 1997b, 1998). This watercourse drains a 800 km^2 area,

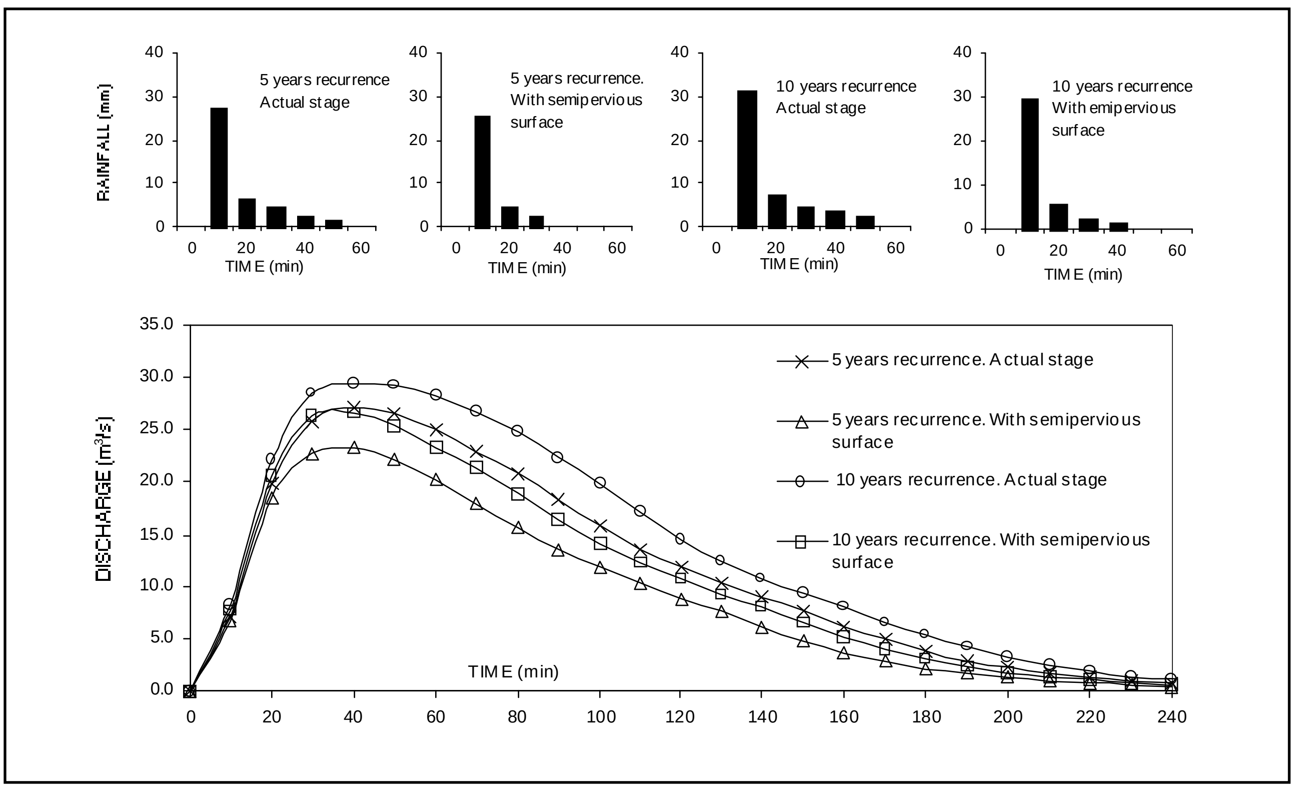

Figure 5. Calculated net hyetographs and hydrographs. Actual and with semipervious surface stages.

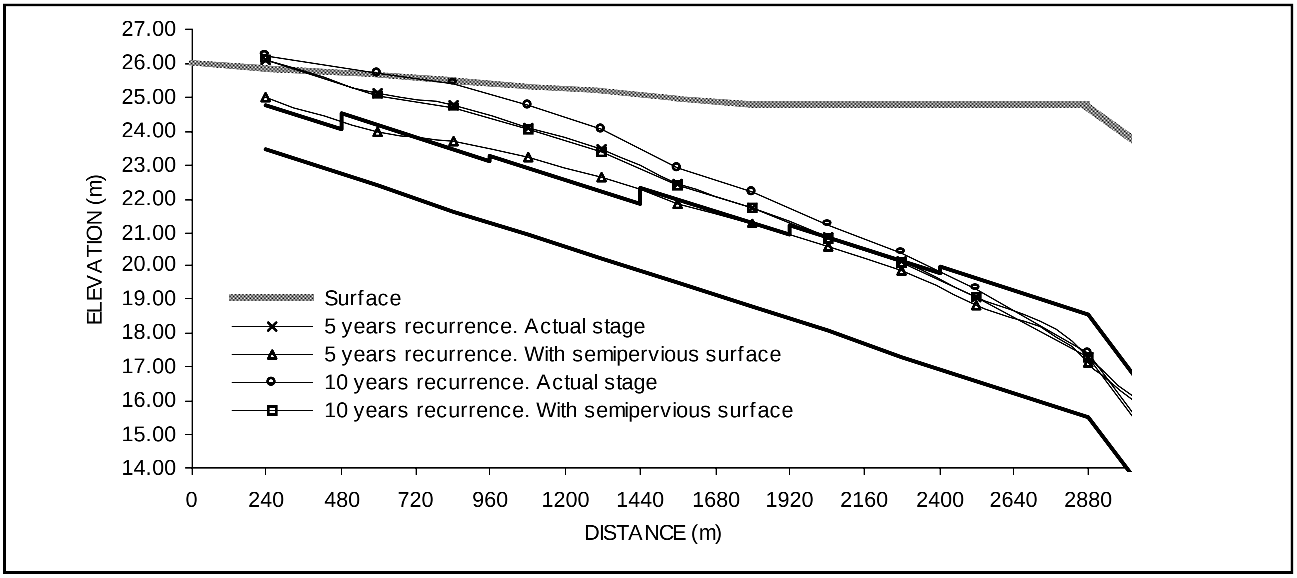

Figure 6. Calculated water levels in trunk conduit.

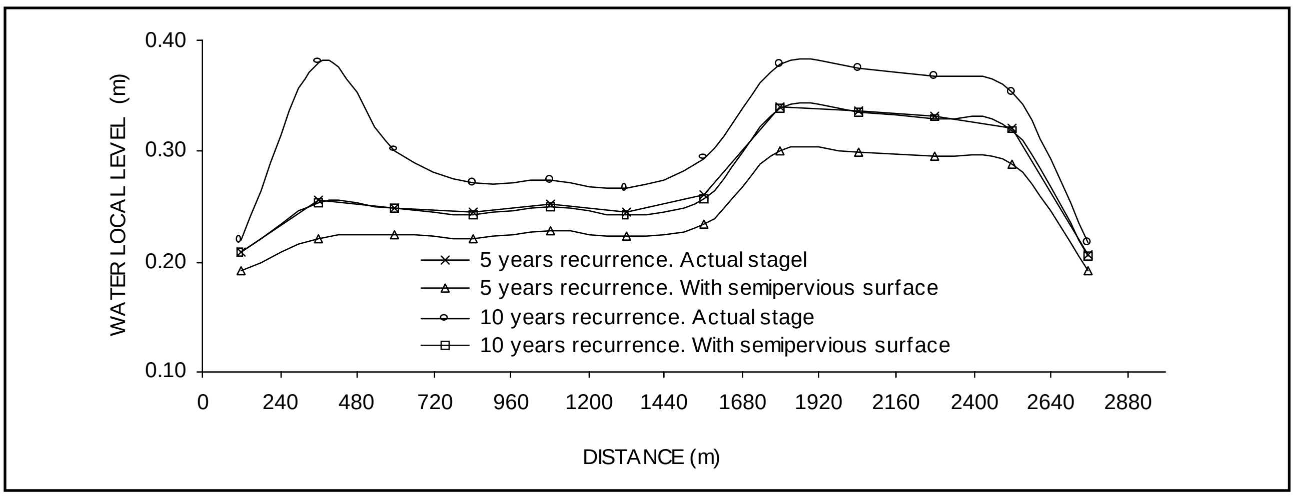

Figure 7. Calculated water levels over the main avenue.

flowing toward the Paraná River. The floodplain under study includes a superficial 50 km^2 area over the low basin. Seventy-five percent of it is rural, 15 percent semiurbanized and 10 percent is fully inhabited. There are around 300,000 inhabitants. The river length including its tributaries is 19 km, with a 1.2‰ average slope. In the main stream, overflow occurs at 80 m^3/s. In a reach 1.5 km in length the watercourse is piped in five closed conduits. These conduits have a 74 m^2 cross section, and the discharge maximum capacity is 350 m^3/s. The main goal of the study was to analyze the hydraulic behavior of the system to determine the flood risk maps for the natural state (without works) and projected state (with structural works such as an upstream retention dam and new closed conduits). Those maps were determined for 50, 100 and 500 years of recurrence. In addition, a mapping for the maximum probable flood (MPF) as a catastrophic event was also carried out. The topological and spatial discretization was done with 202 cells and 311 links (Figure 8). The level of detail was 4 elements/km^2. The model was calibrated using data from historic floods, mainly with one measured in 1986 of 50 years recurrence.

During the model execution, the input peak discharges were 500 m^3/s (RP= 50 years), 700 m^3/s (RP= 100 years), 1,000 m^3/s (RP= 500 years) and 1,700 m^3/s for the MPF. For the flood of 500 years recurrence the maximum relative error in water level over the main stream was 11 percent and over

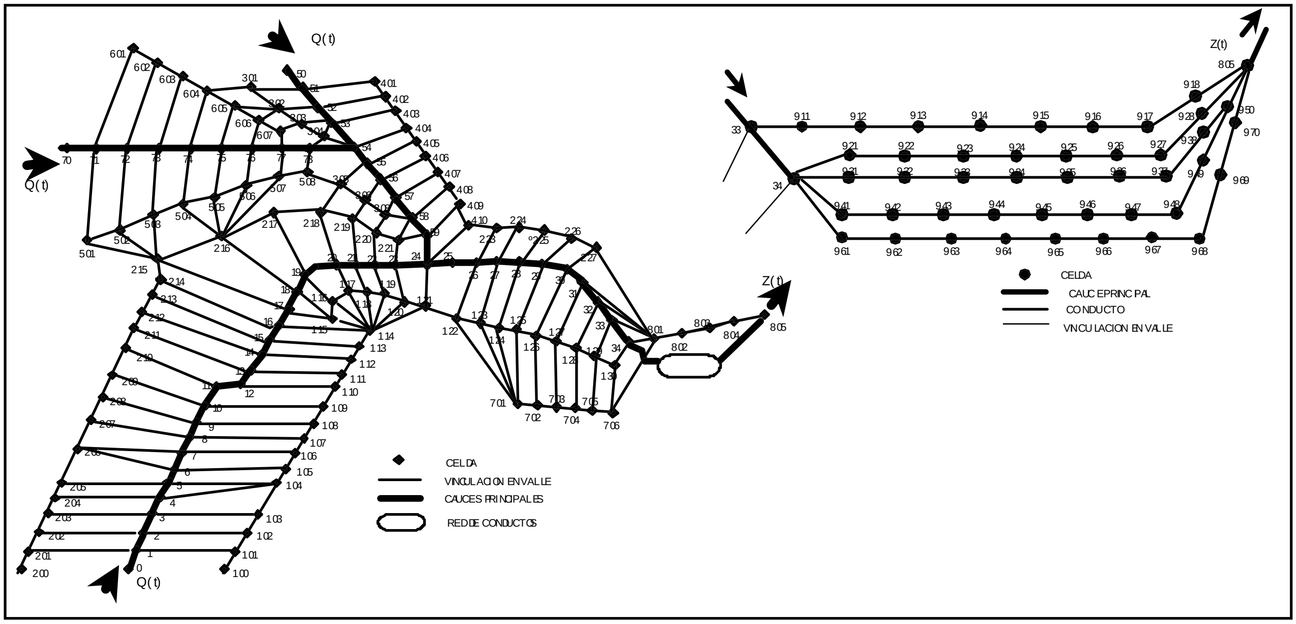

Figure 8. Topological model representation. (a) Complete zone; (b) Closed conduit network.

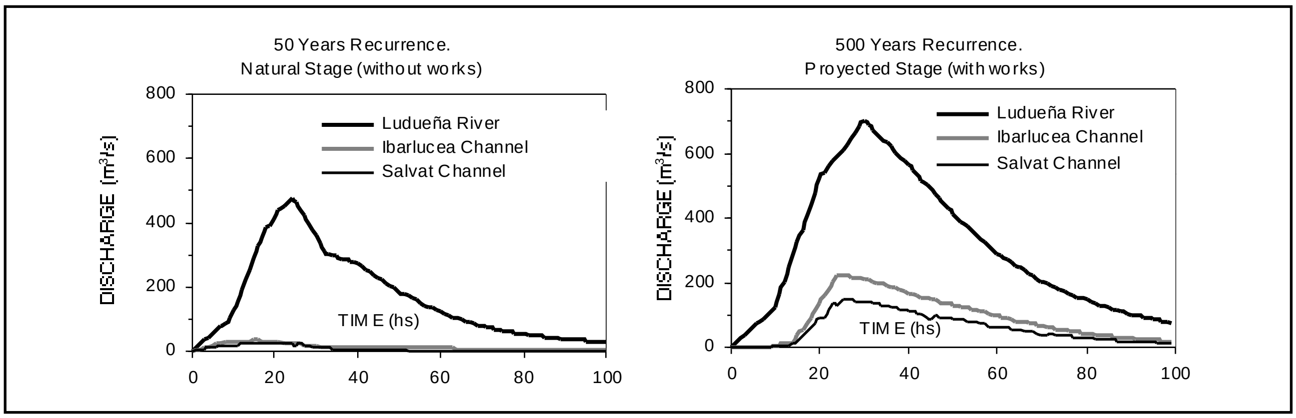

Figure 9. Input hydrographs. 50 and 500 years recurrence.

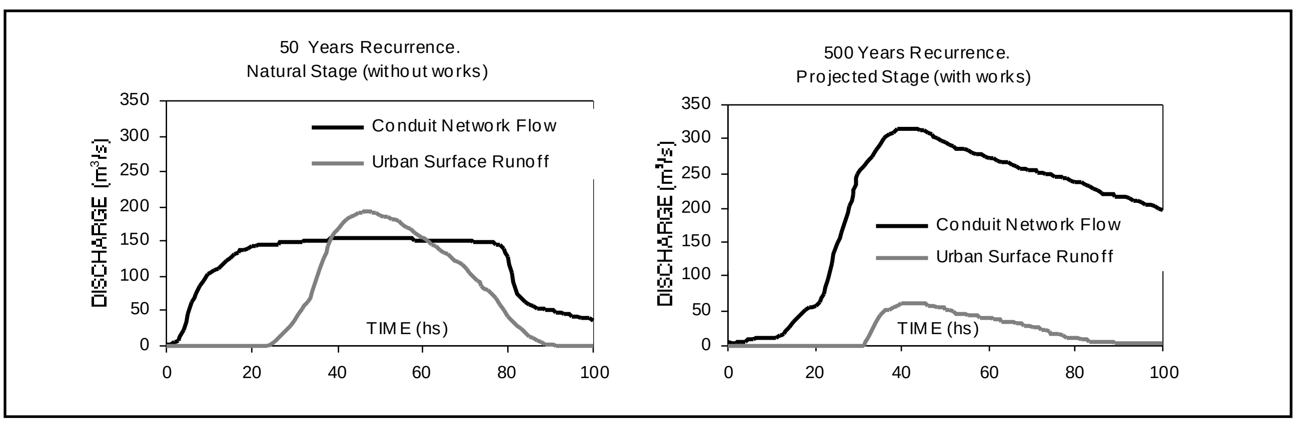

Figure 10. Downstream hydrograph. 50 and 500 years recurrence.

the flood plain 19 percent. The results for a 50 year recurrence without works and a 500 year recurrence with works are presented. The inflow from the high basin is shown in Figure 9. The computed hydrographs at the downstream boundary over the urban zone, routing by the major system and by the conduits are illustrated in Figure 10. The corresponding flood maps are shown in Figure 11. From the results obtained in both simulations the positive effect of the structural works

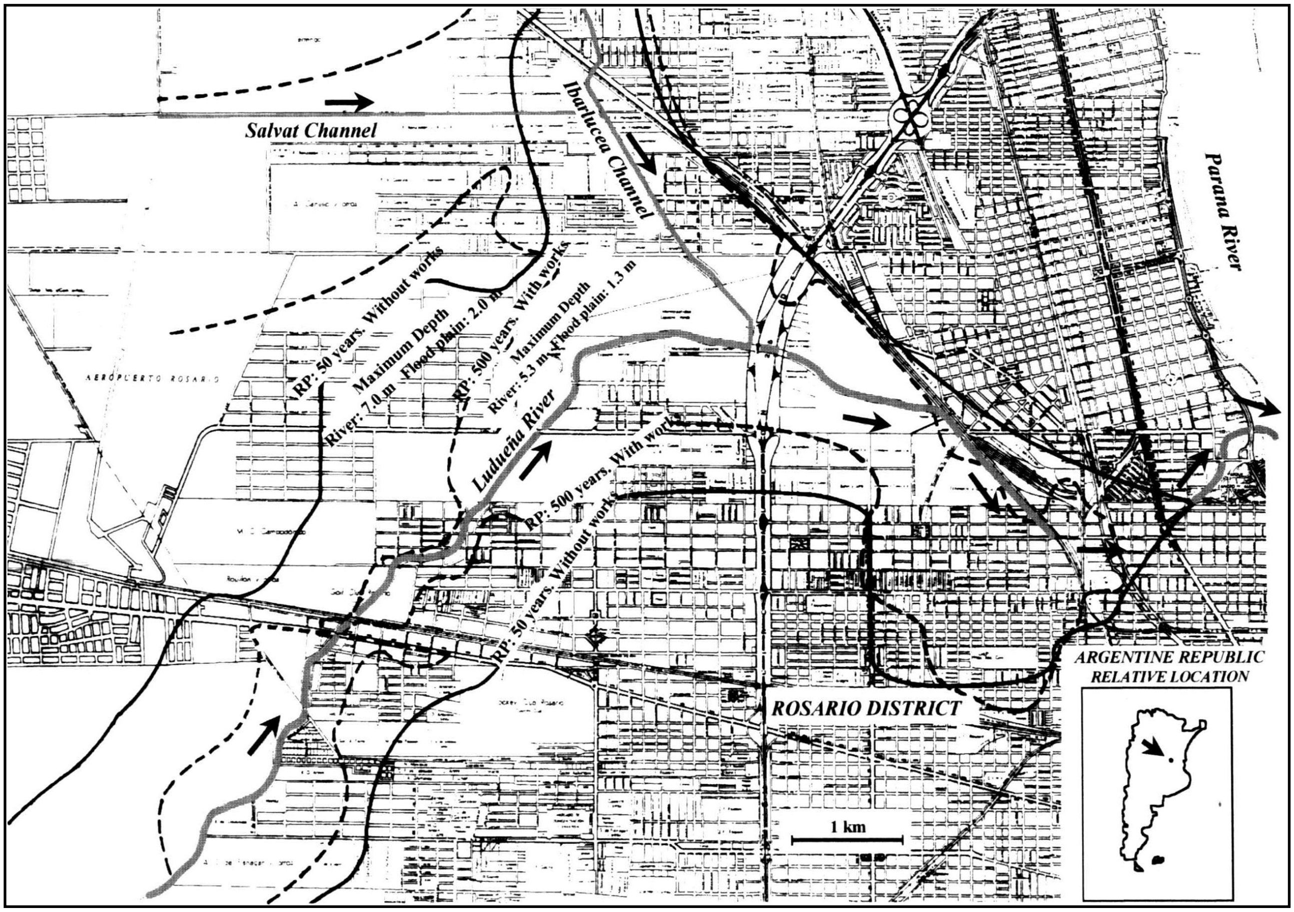

Figure 11. Ludueña river flood map.

can be observed. Based on the risk area delimitation described, state and local government have planned the non-structural rules and are developing the associated legislation.

The Ludueña River basin

This application used the CELDAS8 model as a subsystem of flow surface propagation in an integral system of hydrological simulation called SHAL (Zimmermann and Riccardi, 1995). This integral system is capable of simulating hydrological processes such as: precipitation, evapotranspiration, surface storage, infiltration, surface runoff, percolation, flow in non-saturated zones and groundwater flow.

The Ludueña river basin was modeled. It is located south east of Santa Fe province (Figure 12). The length of the hydrographic network (including permanent and transitory watercourses) is 140 km and the basin area is 700 km^2 with an average slope of 1.0 ‰. The base flow is 0.50 m^3/s, under ordinary flow condition it is 80 m^3/s and in extraordinary events (more than 50 years recurrence) discharges larger than 400 m^3/s have been observed.

The discretization chosen was constituted by 100 cells and 111 links, reaching a mean level of detail of 0.14 elements/ km^2 (Figure 13). At the downstream boundary a level-discharge relationship under a big road bridge was considered. The links between tributary cells and river cells were kinematic and the links between watercourse cells were simple river and bridges (culverts). The considered detail definition has allowed the fine representation of multiple geometric and hydraulic parameters of the components of the system.

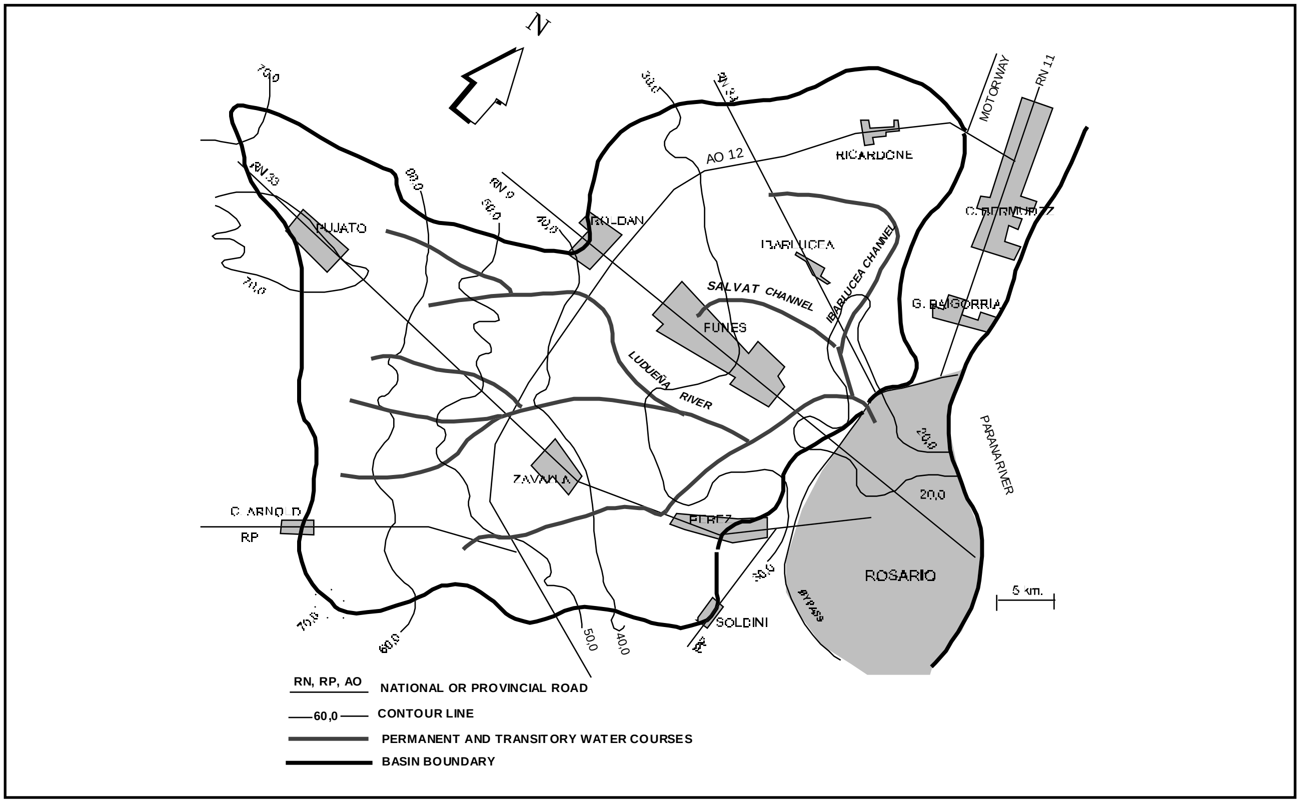

Figure 12. The Ludueña River basin.

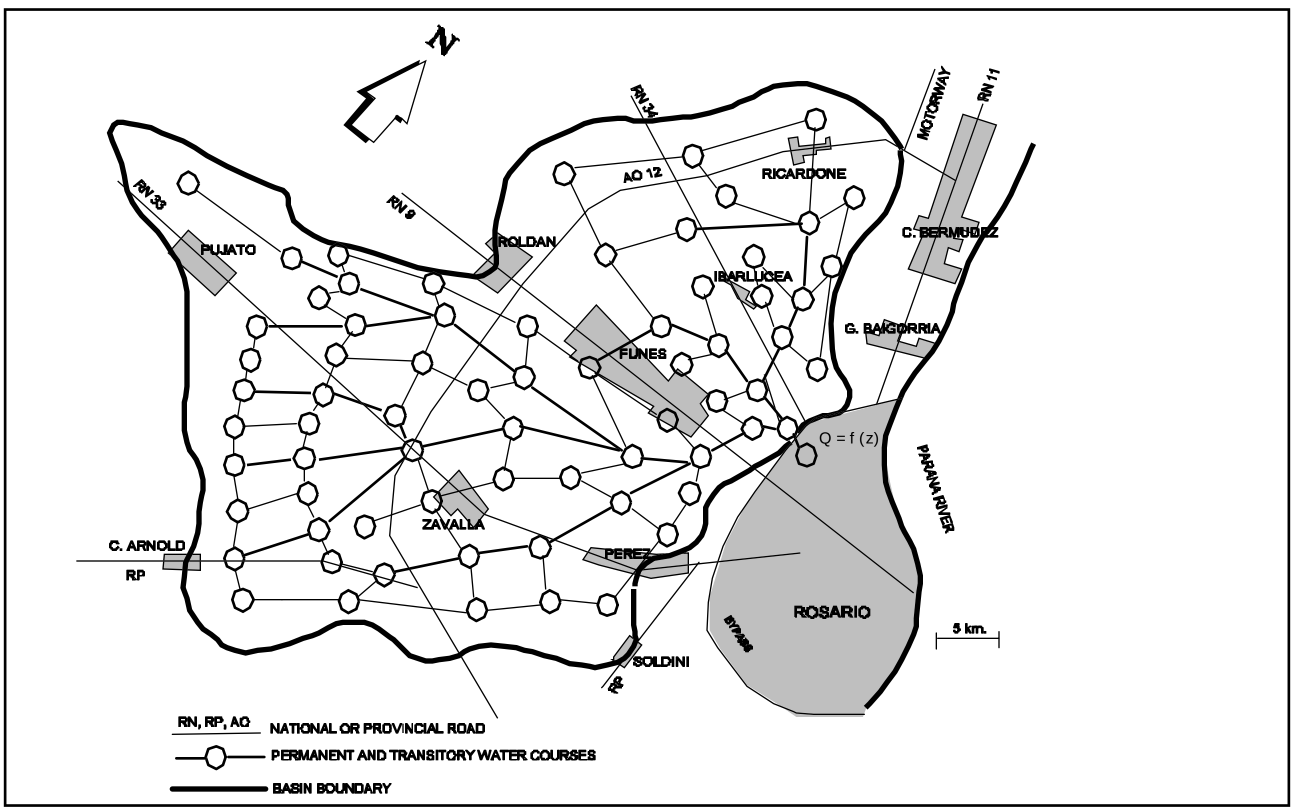

Figure 13. Topological model representation of the Ludueña river basin.

In each simulated event the CELDAS8 model was loaded with the net rainfall computed by means of the subsystem of losses of the SHAL. The model was calibrated considering observed data from a series of events. There are two meteorological stations with rainfall and hydrological parameters recorders. In addition, there are two measurement stations for water level discharges under bridges. The comparison between observed and calculated hydrographs at the downstream boundary is shown in Figures 14 and 15, corresponding to typical floods without and with floodplain.

The CELDAS8 model as a component of the SHAL was exploited for the analysis of diverse runoff alternatives. In this way, the model application has been able to incorporate a technological tool into the planning of regional water resources.

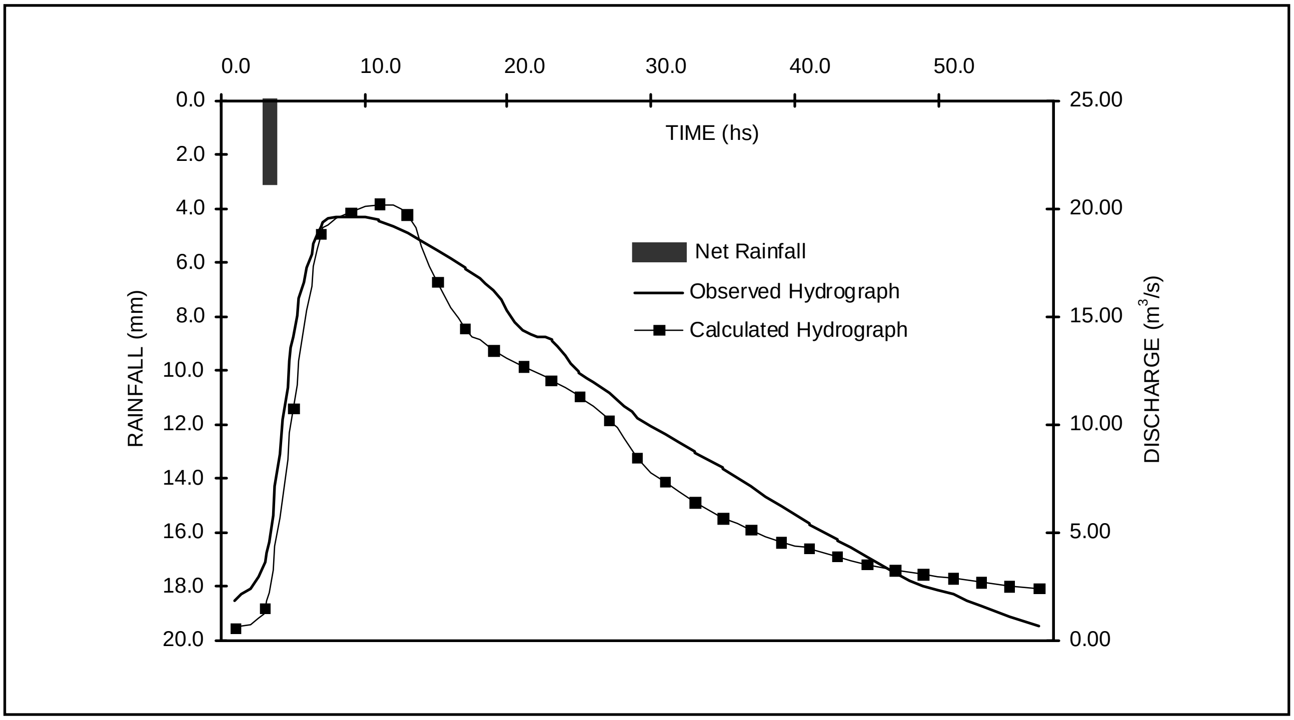

Figure 14. Observed and calculated hydrographs for the Ludueña River basin. Event without overflow.

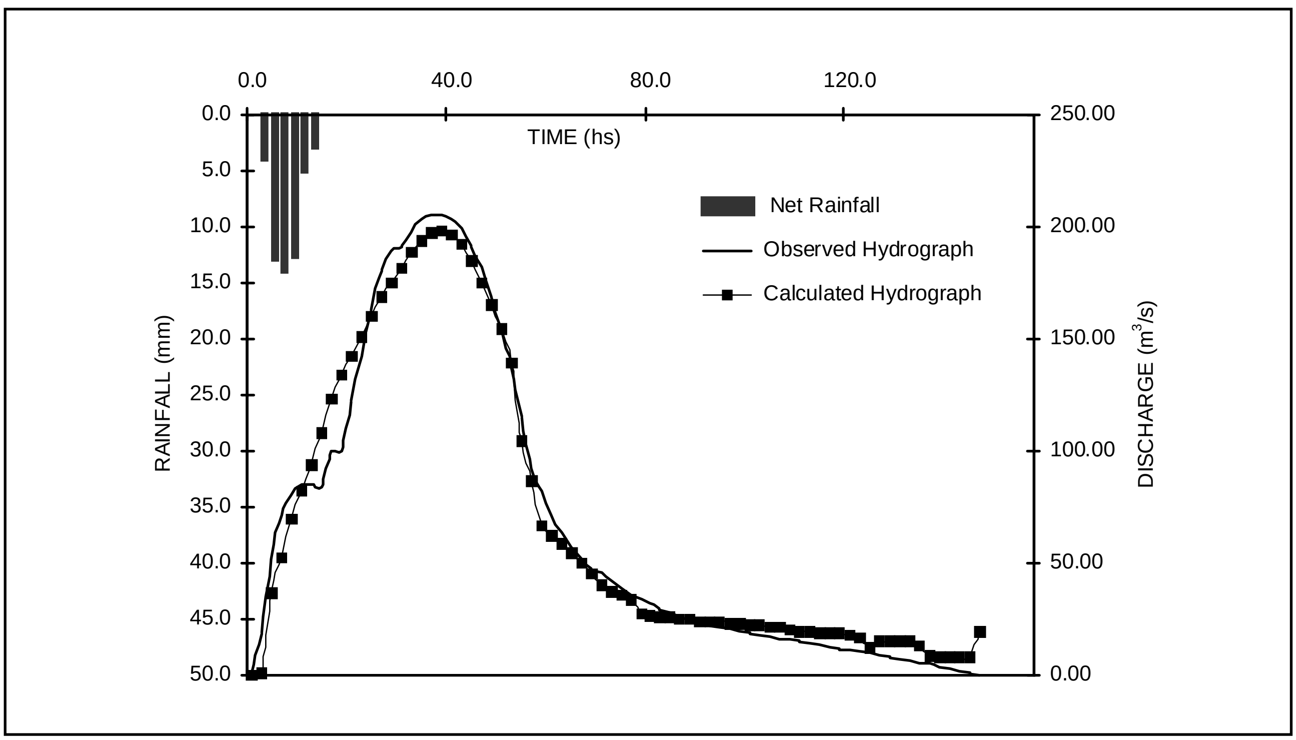

Figure 15. Observed and calculated hydrograph for the Ludueña River basin. Event with overflow.

CONCLUSIONS

The modeling system presented here has demonstrated a satisfactory capability to reproduce the multiple processes involved in the rainfall-runoff transformation. It can simulate quasi-2D flow in a single layer and flow over two interconnected layers. The simulation structure allows the representation of different levels of detail, allowing the maximum possible subdivision of the physical components with geometric and hydraulic parameters consistent with the available data. The calibration parameters (in all tests and applications) were in agreement with the range of standard values encountered in the literature for flow resistance.

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NOTATION

A_{Si}	Superficial wetted area of the cell *i* (m^2)
z_i and z_k	Water level at the cells *i* and *k* (m)
$Q_{k,i}$	Discharge between cells *k* and *i* ($m^3 s^{-1}$)
$\Delta x_{k,i}$	Distance between centers of cells *k* and *i* (m)
$K_{k,i,}$	Conveyance coefficient of the link between cells *k* and *i* ($m^3 s^{-1}$)

$Rh_{k,i}$	Hydraulic ratio of the link between cells k and i (m)
$At_{k,i}$	Cross sectional area of the link between cells k and i (m^2)
η	Manning resistance coefficient ($s\ m^{-4/3}$)
At_i, At_k	Cross section of wetted areas of the cells k and i (m^2)
a_1, a_2, b, c	Coefficients, depending on $Q_{k,i}^{(n-1)}$, $z_i^{(n-1)}$, $z_k^{(n-1)}$ (m^{-3}), (s^{-1}), ($m^3\ s^{-2}$)
z_w	Elevation bottom weir (m)
μ_1, μ_2	Weir discharge coefficients for free and submerged flow
b	Weir width (m)
z_{cri}	Critical level in control section (m)
A_{cri}	Wetted area of the control section for critical level (m^2)
A_{Sc}	Wetted area in the control section (m^2)
Cd	Discharge coefficient in control section
g	Acceleration of gravity ($m\ s^{-2}$)
A_{cll}	Full cross section of closed conduit (m^2)
μ_l, μ_a	Discharge coefficients of storm inlets (weir or hole) for free and submerged flow
L_{su}	Opening length (m)
Cs	Coefficient of geometric and hydraulic characteristics in storm inlet
z_{su}	Elevation bottom of gutter (m)
Q_{ll}	Discharge in full section conduit ($m^3\ s^{-1}$)
$Q_{p;}\ Q_b$	Peak and base discharge ($m^3\ s^{-1}$)
$t_{p;}\ t_b$	Peak and base time (s)
D; L	Diameter and length of conduit (m)
i_l	Longitudinal slope
RP	Return period (years)
t	Time (s)

ADDRESS FOR CORRESPONDENCE
Gerardo A. Riccardi
Consejo de Investigaciones and CURIHAM
Facultad de Ciencias Exactas, Inngeniería y Agrimensura
Universidad Nacional del Rosario
José Hernandez 986
2000 Rosario
Argentina

E-mail: riccardi@fceia.unr.edu.ar