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Simulation of the urbanization effect in flow

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ABSTRACT A consequence of urban development in a basin is the increase of peak discharge and surface flow which may cause flooding. Main channel design has to take into account future city development. A mathematical conceptual rainfall-runoff model was developed in order to simulate an urban basin at Porto Alegre (Brazil). The model has four algorithms: depression, interception losses, soil storage, basin routing and channel routing. Based on the Urban Development Master Plan, some of the model parameters were modified and evaluated, and a preliminary assessment of the effect of urbanization on flow hydrographs was made.

Simulation de l'effet de l'urbanisation sur l'écoulement
RESUME Une conséquence du développement de l'urbanisation sur l'hydrologie d'un bassin est l' augmentation du débit de pointe et de l'écoulement de surface qui peut provoquer des inondations. Le projet du canal principal doit tenir compte du développement futur de la ville. Un modèle mathématique conceptuel pluie-débit a été mis au point pour simuler l'écoulement sur un bassin urbain à Porto Alegre (Brésil). Le modèle présente quatre algorithmes: dépression, pertes par interception, stockage dans le sol, déplacement de l'écoulement à la surface du bassin et dans le réseau de conduites et de caniveaux. Sur la base du plan du développement urbain certains des paramètres ont été modifiés et précisés et on a pu faire une évaluation préliminaire de l'effet de l'urbanisation sur l'hydrogramme.

INTRODUCTION

The drainage system of a natural basin is modified by urbanization, becoming a network of pipes and channels, which reduces concentration time and infiltration volume. Since urbanization often extends from downstream to upstream in a basin, the design of the main channel must take into account future land use. This is even more important when the basin is large and there is no land use regulation.

Many models have been developed in order to design storm sewers

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and evaluate new development: MITCAT (Harley *et al.*, 1970); SWMM (Metcalf & Eddy, 1971); WASSP (National Water Council, 1981) among others. These models represent the basin in detail, simulating each city block, but there are models which represent the basin as a lumped system: HEC-1 (HEC, 1973); STANFORD IV (Crawford & Linsley, 1966); TR-20 (USDA, 1969). The former type of model requires a large amount of data and involves high computation cost, but provides a better solution. Lumped models are useful to simulate large urban basins because they use less data and only a small amount of computation. The basic difficulty with this type of model is the prior evaluation of the parameters for the period after expected urbanization has taken place.

When urbanization plans are still under an overall regulation it is difficult to use a model which represents the area in detail. In addition, such detailed formulation becomes expensive for large urban basins. A conceptual urban model was developed in order to simulate urbanization effects in the rainfall-runoff process, based on the land use planned for a basin. The first and second versions of this model (IPH I and II) have been used in many basins in South America (Muñoz & Tucci, 1974; Tucci *et al.*, 1981) and were employed in a real time forecasting problem (Tucci & Clarke, 1980). Those versions did not take channel routing into account. A new version of the model routes the channel flow through a kinematic wave formulation with the lateral flow calculated by the IPH II version. A brief description of the model is given in the following section.

THE MODEL

Basin algorithm

Interception loss Interception due to depression or vegetation is modelled by a reservoir which has a maximum capacity of R_{max}, a model parameter. The interception reservoir is used at the beginning of a rainfall until it fills. This reservoir loses volume through evaporation (which is disregarded in urban simulation).

Rainfall in the basin can fall on impervious surfaces such as building roofs or car parks, or on lawns or other pervious surfaces where it can infiltrate. The percentage of impervious area, AINP, is used as a model parameter in order to compute part of the effective volume. In these areas water flows over the impervious surfaces until it reaches the channel.

Soil storage loss The effective rainfall or surface volume of the area subject to infiltration is computed based on the Horton equation:

$$I = I_b + (I_o - I_b)\, e^{-kt} \qquad (1)$$

where I is the infiltration capacity; I_o, I_b and k are parameters; and t is time.

Percolation is simulated by a similar equation:

$$T = I_b\ (1 - e^{-kt}) \tag{2}$$

where T is percolation.

The continuity equation of the upper soil zone is:

$$\frac{dS}{dt} = I - T \tag{3}$$

where S is soil storage.

Substituting equations (1) and (2) in (3) and integrating results in:

$$S = \frac{I_o}{\ln h}\ (e^{-kt} - 1) \tag{4}$$

where $h = e^{-k}$.

Using equations (1) and (2) in equation (4) results in:

$$S = \frac{-\ I_o^2}{\ln h\ (I_o - I_b)} + \frac{I_o}{\ln h\ (I_o - I_b)} \cdot I \tag{5}$$

$$S = \frac{-I_o}{\ln hI_b}\ T \tag{6}$$

Considering rainfall P in an interval t to t + Δt, three situations can occur, depending on the relationship of rainfall rate to infiltration rates at times t and t + Δt:

(a) When rainfall (P) is greater than infiltration capacity (I_t), effective rainfall can be calculated by integrating the Horton equation in the interval (Fig.1(a)):

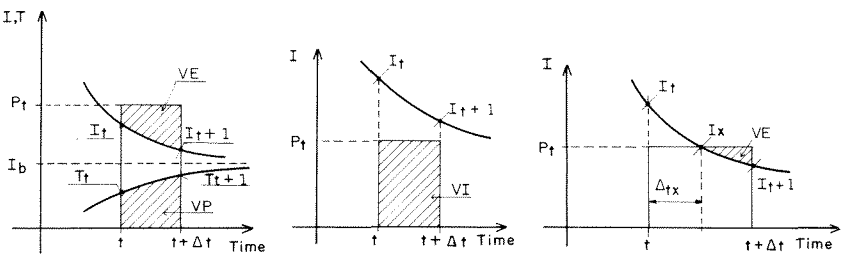


Fig. 1 Infiltration algorithm conditions: (a) when $P_t > I_t$, (b) when $I_t > P_t$, and (c) when $I_t > P_t$ and $I_{t+1} < P_t$.

$$V = \Delta t\ (P - I_b) - \frac{(I_t - I_b)}{\ln h}\ (h^{\Delta t} - 1) \tag{7}$$

and the variables I_{t+1}, S_{t+1} and T_{t+1} are calculated by equations (1), (4) and (6) respectively.

(b) When P < I_t, all of the rainfall infiltrates and the effective rainfall is zero. Using equation (6) in (3) and putting

$I = P$ results in:

$$S_{t+1} = (S_t - Pb)e^{-t/b} + Pb \tag{8}$$

where $b = - I_o/\ln hI_b$. I_{t+1} is calculated by equation (5). If I_{t+1} is greater than the rainfall (Fig.1(b)) it is only necessary to calculate T_{t+1} by equation (6).

(c) When $P > I_{t+1}$, an intersection of the infiltration capacity curve with the rainfall curve occurs (Fig.1(c)). Using equation (8) for $I = P$, the intersection time is:

$$\Delta t_x = - b \ln[(S_{t+1} - Pb)/(S_t - Pb)] \tag{9}$$

The interval $(t, \Delta t_x)$ is calculated according to case (b), and the interval $(\Delta t_x, t + \Delta t)$ according to situation (a). When the percolation volume is required, a balance of continuity is performed in order to calculate it.

Basin routing Clark's method is used to route the effective rainfall. This method uses a time-area histogram and a linear reservoir procedure. First, the time-area procedure is used:

$$V_t^* = \sum_{j=1}^{t_c} V_{t-j+1} f_j \tag{10}$$

where the f_j are the histogram ordinates and t_c is time of concentration in units of Δt.

The time-area histogram may be estimated by methods based on basin physical characteristics, or the synthetic time-area histogram (HEC, 1973) based on the length of the main stream may be employed.

Finally, the effective rainfall is routed through a linear reservoir:

$$Q_t^b = Q_{t-1}^b \; e^{-t/k_s} + V_t^* \; (1 - e^{-t/k_s}) \tag{11}$$

where Q^b is the basin flow, and k_s a parameter.

Groundwater routing is done using equation (11) with percolation volume instead of V_t^* and another value for k_s. It is important only for continuous simulation in rural basins.

Channel routing Channel routing is performed by the kinematic wave method, using the continuity equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \tag{12}$$

where Q is the channel discharge, A the channel cross-section area, $q = Q^b/L$, and L the length of channel reach to which the basin contributes.

In this method, the dynamic equation is simplified by making the friction slope equal to the bed slope. This equation is often expressed as:

$$A = \alpha Q^{\beta} \tag{13}$$

where α and β are parameters.

Li *et al.* (1975) presented a nonlinear implicit scheme of finite differences to solve these equations. The numerical scheme is:

$$\frac{\partial Q}{\partial x} = (Q_{j+1}^{t+1} - Q_{j}^{t+1})/\Delta x$$

$$\frac{\partial A}{\partial t} = (A_{j+1}^{t+1} - A_{j+1}^{t})/\Delta t \tag{14}$$

$$q = (q_{j+1}^{t+1} + q_{j+1}^{t})/2$$

Substituting equation (12) in (13) and applying the numerical scheme, a nonlinear equation results. This is solved using the Newton method with a second-order approximation.

Initial conditions The percolation volume, V_p, when $P = 0$, is

$$V_p = S_t - S_{t+1} = S_t\ (1 - e^{-t/b_t}) \tag{15}$$

Using equation (15) in (11) (groundwater routing) results in:

$$S_t = \frac{\Delta t(Q_{t+1} - Q_t e^{-\Delta t/k_b})}{(1 - e^{-\Delta t/b_t})\ (1 - e^{-\Delta t/k_b})} \tag{16}$$

where k_b is the parameter for the baseflow routing.

The estimation of the initial condition, S_t, requires the discharge of two steps, Q_{t+1} and Q_t. It was assumed that there is no surface flow which requires several time steps without rain. If it is assumed that $Q_t \cong Q_{t+1} \cong Q_o$, equation (16) results in:

$$S_o = Q_o\Delta t/(1 - e^{-t/b_t}) \tag{17}$$

The infiltration capacity and percolation at t = 0 can be calculated by equations (5) and (6).

When there are many sub-basins in a reach, the initial discharge of each sub-basin is not known. Since the upstream inflow in the reach (Q_v) is given, the lateral inflow is the difference $Q_L = Q_d - Q_v$ where Q_d is the downstream reach flow. Assuming that all sub-basins have the same discharge in units of depth, that discharge is calculated by $Q_o = Q_L/A_L$ where A_L is the total lateral contributing area.

RESULTS

Diluvio Creek

The Diluvio Creek basin is approximately 80 km^2 in area. Part of

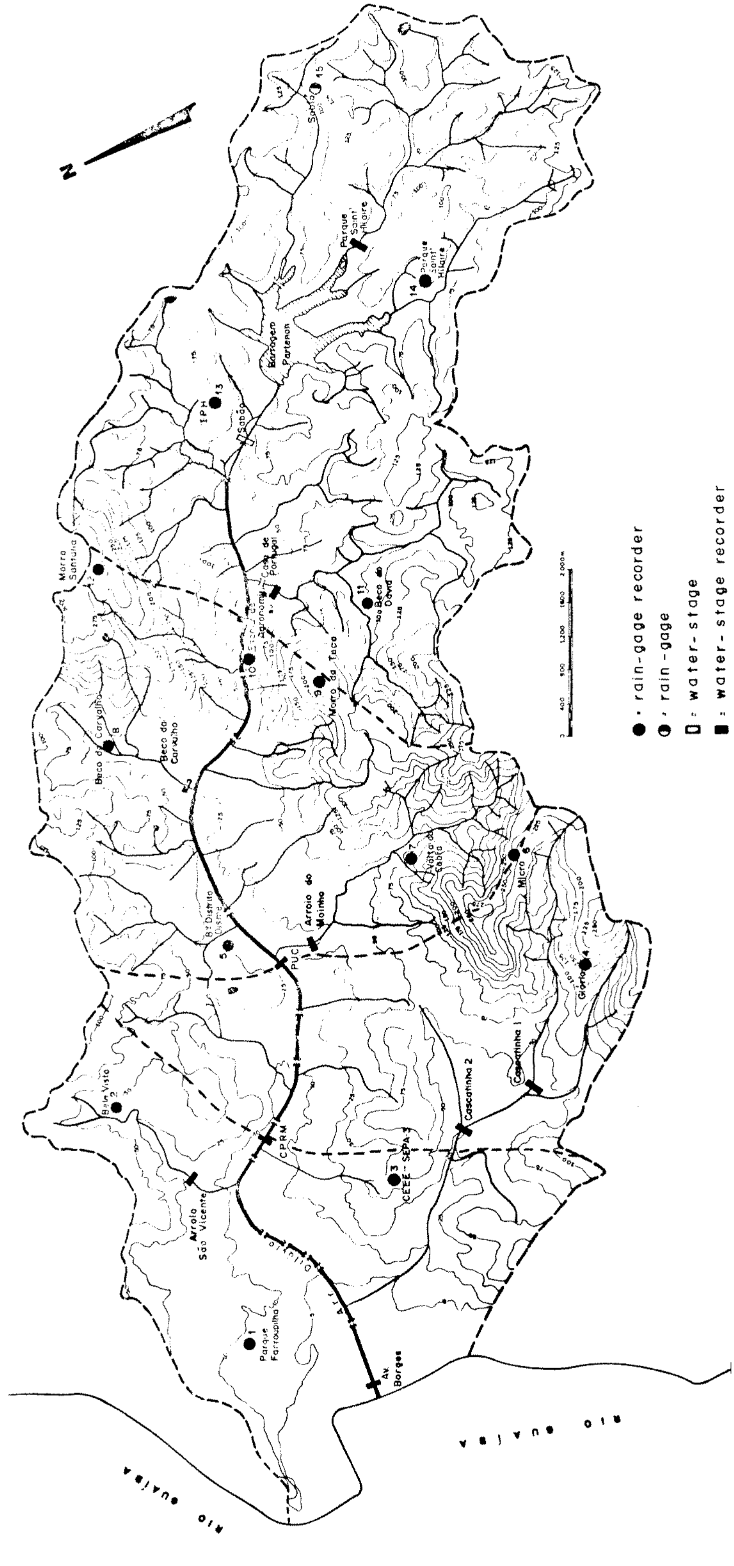


Fig. 2 Map of Diluvio Creek basin.

Porto Alegre, a city of 1.5 million inhabitants in southern Brazil, is in this basin. The Creek is about 13 km in length.

Basin urbanization takes place from downstream to upstream, and flood problems occur in the urbanized part of the basin. Two sub-reaches were used in this study, draining an area of approximately 25 km^2. The degree of urbanization varies, but land use is mainly residential. Figure 2 is the basin map, with the basins of the study reaches shown by dashed lines.

There are three flowgauges in the reach (Agronomy, PUC and CPRM) and 14 raingauges, all with automatic recorders. In another study (Alvares & Sanchez, 1979) the basin was divided in sub-basins based on the city drainage pattern and on physical characteristics such as slope, area, drainage length etc., which were measured. In addition, occupation characteristics, such as type of surface and population per hectare, were determined. The former data were obtained through orthophotography.

A new Master Plan was approved by the county authorities to regulate city land use. This Plan specifies the type of housing and the population density of the area.

Discretization

The reach was subdivided in order to fit the model. Since there are two flowgauges, it was divided into two main reaches (Agronomy-PUC and PUC-CPRM). The first reach is 2.4 km long and has a basin area of 19 km^2. Seven sections were chosen for the channel routing and eight sub-basins. The main purpose of the basin subdivision was to simulate the difference in the planned land use. Figure 3 shows the reach discretization. The second reach (PUC-CPRM) is also 2.4 km long but has a 6 km^2 basin area. Four sections were used for channel routing and only one lateral contribution basin.

Eight flood events were selected, based on the extreme flows. The rainfall and the discharge were abstracted at 30-minute time intervals. For each event an isohyetal map was prepared with the rainfall totals of the raingauges. The individual rain for each time step was calculated using the time distribution of the rain-gauge nearest to the sub-basin.

Fitting parameters

In the first reach, the upstream inflows are fairly small and the main effect is the lateral contribution from the basin, as can be seen in Fig.4. This Figure shows model results but in Fig.5(b) it can be seen that the model simulation of this event was in good agreement with recorded values.

Five events were used to fit the parameters for the Agronomy-PUC reach. Some parameters were estimated based on basin characteristics such as time of concentration and k_s. The initial channel parameters α and β were estimated based on the Manning equation using the channel physical characteristics. Those parameters were fitted using the recorded hydrographs at the PUC flowgauge. Table 1 gives the obtained values.

The more sensitive parameters, I_o and k in the Horton equation, were fitted using the flow recorded at the PUC flow gauge. In

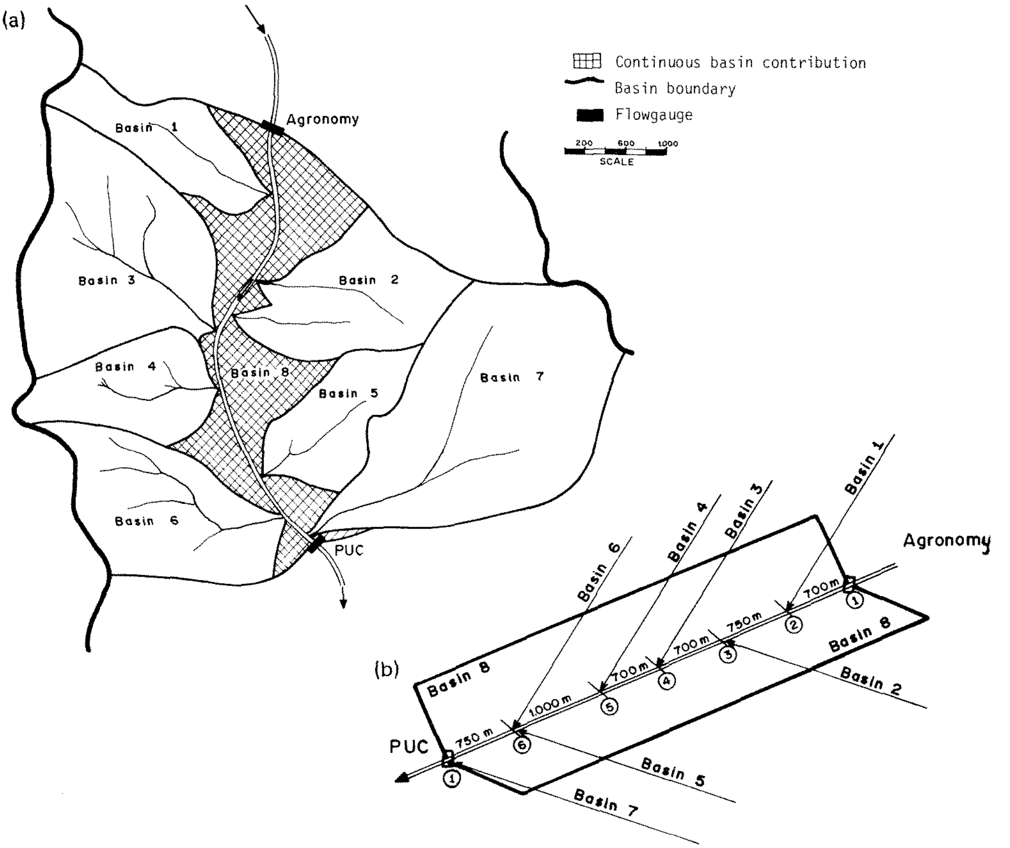


Fig. 3 The reach between Agronony and PUC: (a) plan of the sub-basins; (b) model schematization.

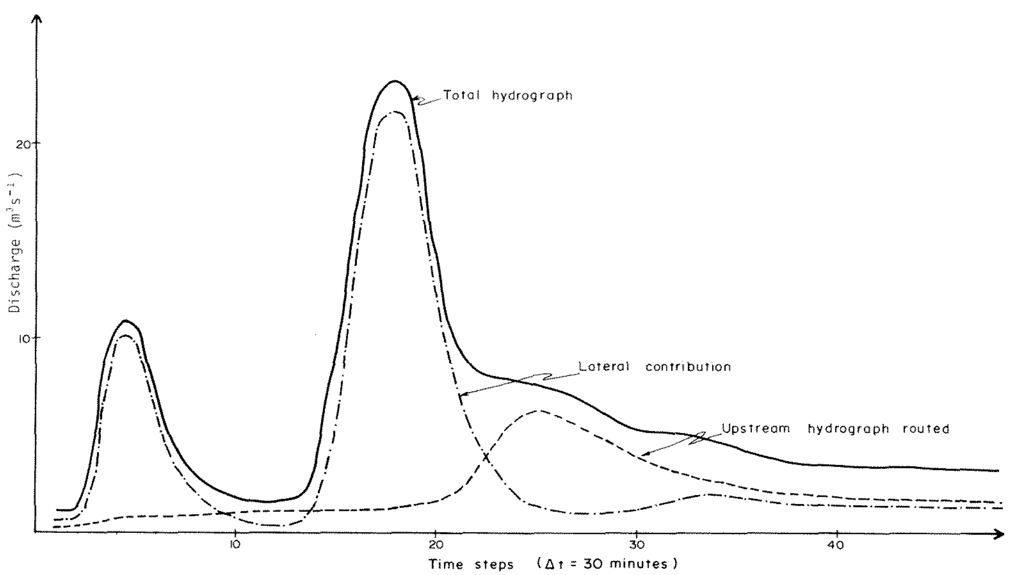


Fig. 4 Lateral contribution and upstream flow: model results.

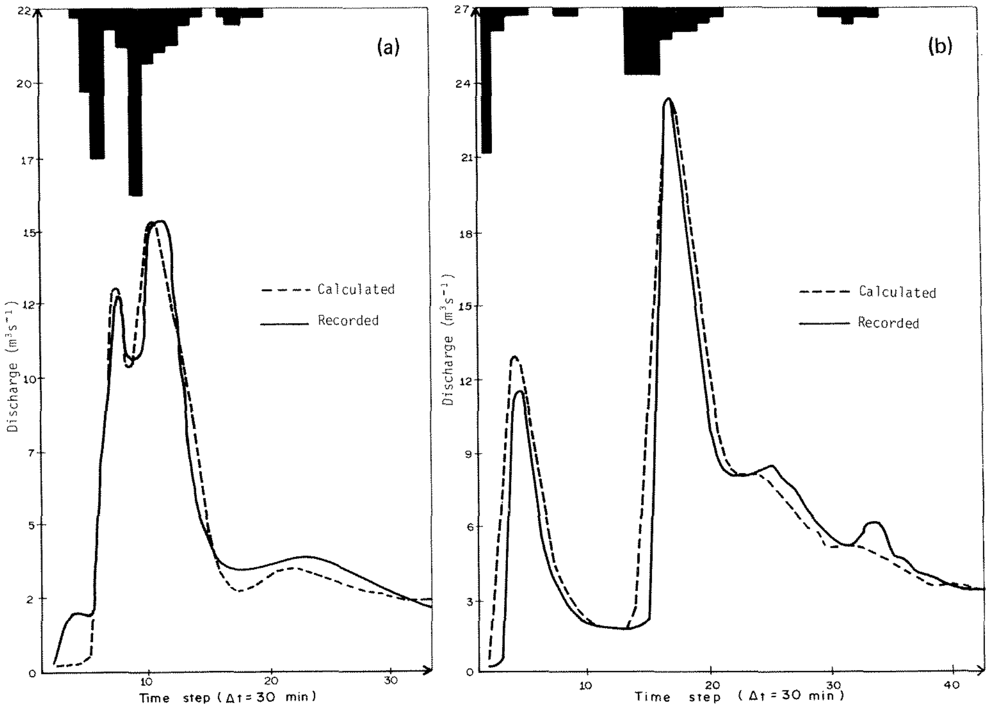


Fig. 5 Fitted events for the reach Agonomy-PUC: (a) hydrograph at PUC on 7 November 1979, (b) hydrograph at PUC on 24 September 1979.

Fig.5, two hydrographs showing these results are presented. The parameters did not vary much in fitting all five events (Table 2). Using mean values as an estimate of the parameters and repeating the simulations for those events, the results were good, giving all

Table 1 Channel parameter variations

	Agronomy-PUC		PUC-CPRM	
Hydrograph	α	β	α	β
1	1.8-1.45*	0.6	–	–
2	1.4	0.6	–	–
3	1.25	0.6	–	–
4	1.25	0.6	–	–
5	1.6-1.3*	0.42	2.0	0.4
6	–	–	2.0	0.4
7	–	–	2.3	0.6
8	–	–	2.3	0.6
Mean parameters	1.4	0.55	2.15	0.50
Conservative parameters	1.25	0.4	2.0	0.4

* α variation from section 1 + 07.

Table 2 Basin parameters of the reach Agronomy-PUC

	Basins 1, 2, 4, 5, 6, 8							Basin 3							Basin 7						
Event	I_o (mm/ ½ h)	I_b (mm/ ½ h)	h *	k_s (l/h)	k_{SUB} (l/h)	R_{max} (mm)	T_c (h)	I_o (mm/ ½ h)	I_b (mm/ ½ h)	h *	k_s (l/h)	k_{SUB} (l/h)	R_{max} (mm)	T_c (h)	I_o (mm/ ½ h)	I_b (mm/ ½ h)	h *	k_s (l/h)	k_{SUB} (l/h)	R_{max} (mm)	T_c (h)
1	13.0	0.35	0.80	1.0	40	7.0	1.0	11.0	0.3	0.79	3.0	40	6.0	1.0	12.0	0.3	0.79	3.0	50.0	6.0	1.0
2	13.0	0.29	0.78	1.0	30	6.5	1.5	10.0	0.22	0.78	3.0	30	5.0	1.0	11.0	0.25	0.78	3.0	30.0	5.0	1.0
3	13.0	0.29	0.82	1.0	40	6.5	2.0	11.0	0.22	0.80	3.0	40	5.0	0.5	12.0	0.25	0.80	3.0	50.0	6.0	0.5
4	13.0	0.27	0.80	1.0	40	6.5	1.0	10.0	0.22	0.78	3.0	30	5.0	0.5	11.0	0.35	0.78	3.0	30.0	5.0	0.5
5	13.0	0.36	0.79	0.6	30	7.0	0.5	10.0	0.3	0.80	3.0	30	5.0	0.5	11.0	0.3	0.80	3.0	30.0	6.0	0.5
Conservatives parameters	13.0	0.27	0.78	0.6	30	6.5	0.5	10.0	0.22	0.78	3.0	30	5.0	0.5	11.0	0.25	0.78	3.0	30.0	5.0	0.5
Mean parameters	13.0	0.31	0.8	1.0	36	6.6	1.0	10.4	0.25	0.79	3.0	34	5.2	0.5	11.4	0.27	0.79	3.0	38.0	5.8	0.5

* $h = e^{-k}$ where k has time units.

determination coefficients greater than 0.8 (0.8-0.92) (Table 4).

The second reach (PUC-CPRM) presented no difficulties in fitting the hydrographs, since the reach is short and the lateral basin is small. Table 3 gives the parameter values for four events and

Table 3 Basin parameters of the reach PUC-CPRM

Event	I_o (mm/ ½ h)	I_b (mm/ ½ h)	h *	k_s (l/h)	k_{SUB} (l/h)	R_{max} (mm)	T_c (h)
5	10.0	0.25	0.8	3.0	40	5.0	1.0
6	9.0	0.25	0.78	3.0	20	4.0	1.0
7	10.5	0.26	0.8	3.0	30	4.5	1.0
8	10.0	0.25	0.8	3.0	20	4.0	1.0
Conservative parameters	9.0	0.25	0.78	3.0	20	4.0	1.0
Mean parameters	10.0	0.25	0.80	3.0	30	4.5	1.0

* $h = e^{-k}$ where k has time units.

Table 4 shows the determination coefficient of the simulation for the four events with mean parameter values. Figure 6 shows calculated and recorded hydrographs at CPRM. Figure 7 shows the simulation of event 5 both with mean and with conservative parameters. Mean parameters simulated the hydrographs better as can be seen in Fig.7 and Table 4.

Table 4 Determination coefficient of the verification simulation

Event	Agronomy-PUC		PUC-CPRM	
	Mean	Conservative	Mean	Conservative
1	0.89	0.70	–	–
2	0.79	0.89	–	–
3	0.92	0.80	–	–
4	0.79	0.86	–	–
5	0.84	0.60	0.92	0.90
6	–	–	0.90	0.92
7	–	–	0.92	0.89
8	–	–	0.89	0.89

The computation of initial conditions by equations (16) or (17) allowed a good start to the fitting process. In the simulation of isolated hydrographs, these estimates have a great effect on the results and, using this procedure, it is not necessary to fit initial conditions. In the simulation of design conditions one can choose the soil saturation level.

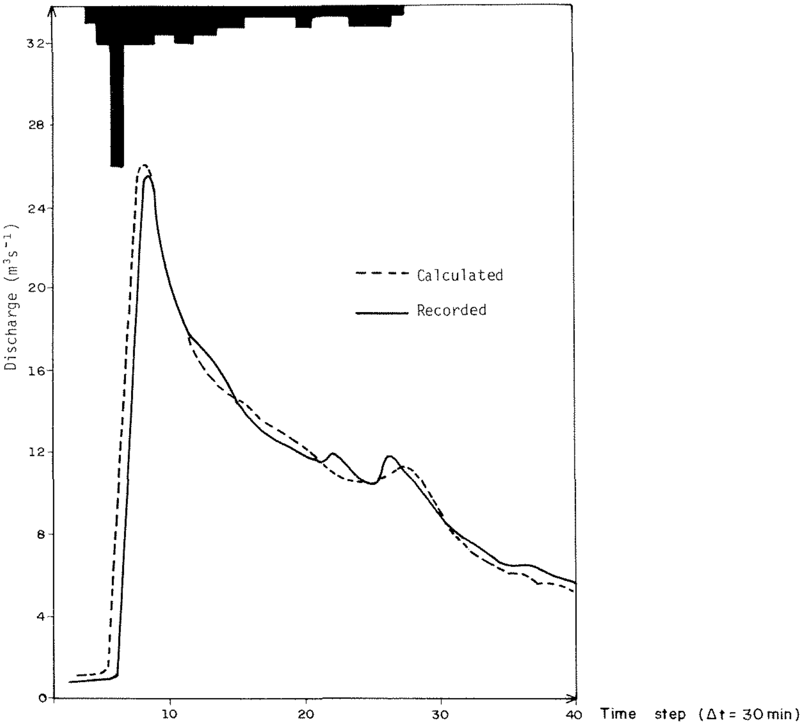


Fig. 6 Recorded and simulated hydrographs at CPRM.

Simulation of future urbanization

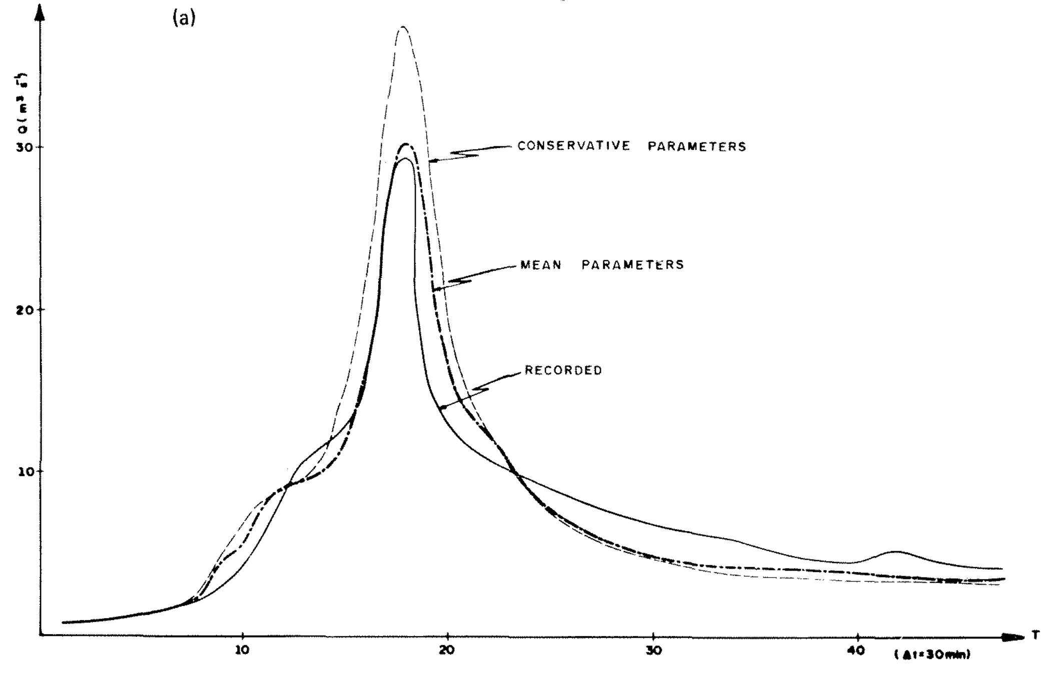
In order to evaluate changes in the flow behaviour due to future basin urbanization it is necessary to estimate changes in model parameters.

In relating the habitation density to the impervious area for the Diluvio basin, two tendencies were obtained (left and right side of the river) (Fig.8). These tendencies occur mainly for high density values and can be explained by the fact that on the right-hand side of the basin there are more houses while on the left-hand side there are more apartment buildings which increase the density for the same impervious area. The high density values in this figure occur in the downstream portion of the basin which is more urbanized.

Table 5 gives the present densities used for fitting the model and the future density based on the Urban Master Plan (PMPA, 1979). The impervious areas were computed from Fig.8 and the Plan specifications (type of housing allowed for each area).

The surface routing parameters change due to the increase both in drainage capacity and in quick-flow surfaces. The time of concentration and the k_s parameters were modified based on the individual characteristics of the basin and on their values in the highly urbanized areas of the Diluvio basin. For example, the modification for k_s used was:

$$k_{s(i)} = \frac{k_s^* \, L^* \, A_{(i)}}{A^* \, L_{(i)}} \tag{18}$$



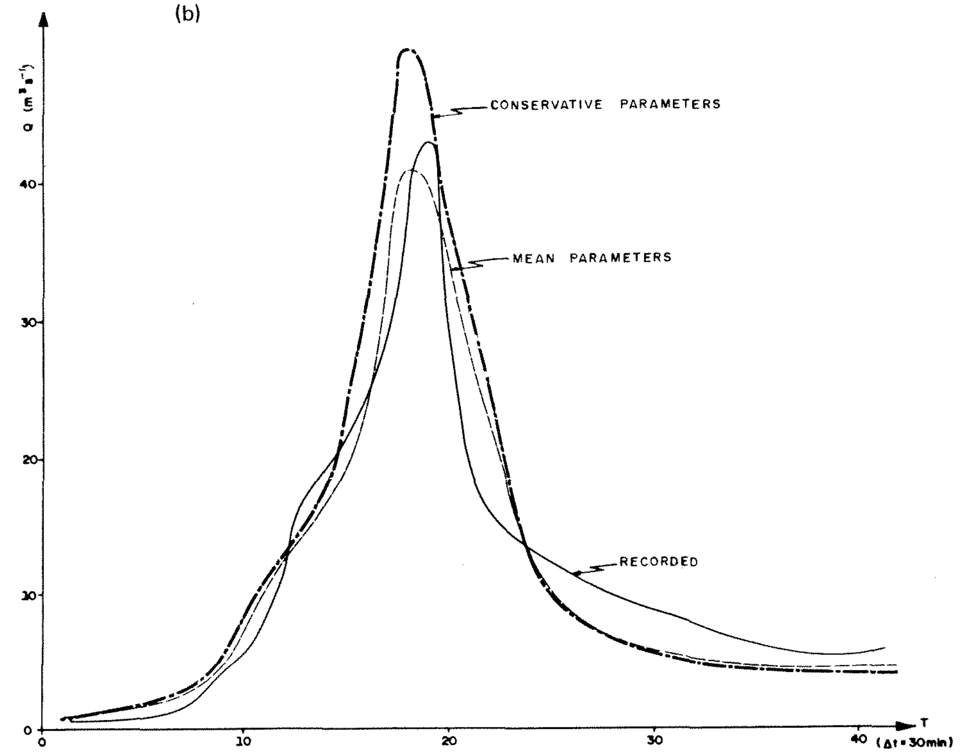


Fig. 7 Comparison of recorded and simulated hydrographs: (a) PUC flowgauge, (b) CPRM flowgauge.

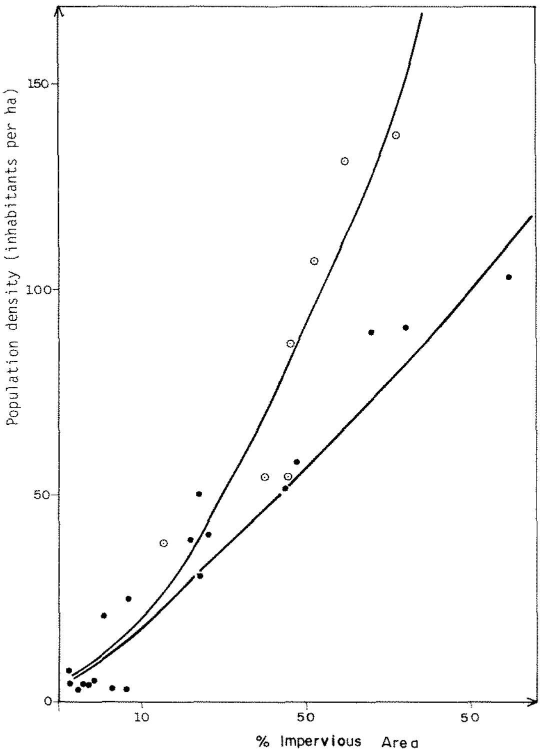


Fig. 8 Population density *vs.* impervious area.

where k_s^*, L^*, A^* are respectively the k_s parameter, the length of the main drainage and the basin area of the highly urbanized basin, and $k_{s(i)}$, $L_{(i)}$, $A_{(i)}$ are the same parameters of basin i.

These give only a rough estimate of the simulation, but for a large urban basin for which there is not a complete design of storm sewers, streets, blocks, etc., such estimates may be used to examine the main drainage behaviour.

In Fig.9, the hydrographs for the section CPRM with both the present and future conditions can be seen. This event has a

Table 5 Impervious areas and population density (inhabitants per hectare)

	Present		Future (Master Plan)	
Basin	Density	Impervious area (%)	Density	Impervious area (%)
1	3	2	120	36.5
2	21	6	125	61.0
3	25	12	125	45.0
4	31	27	170	46.0
5	52	15	172	75.5
6	40	18	175	42.5
7	50	20	150	61.0
8	39	7	125	61.0

rainfall distributed in time which shows a significant effect on the surface volume. The intensity of this event is not high. However, another event with high intensity rainfall (approximately 50-year return period, 95 mm h^{-1}) was also simulated giving much smaller

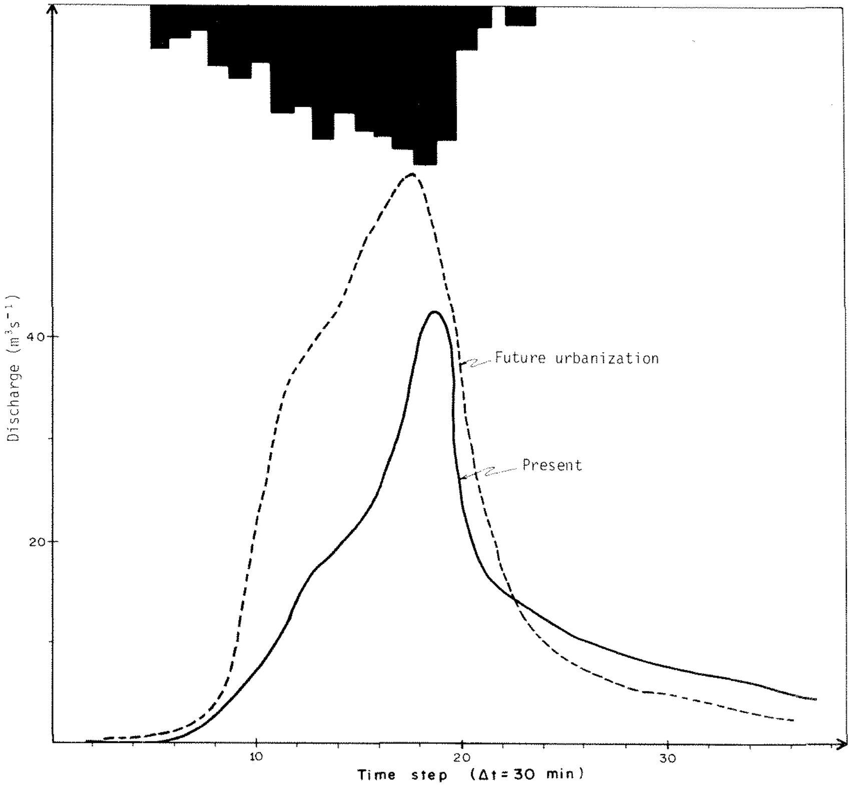


Fig. 9 Hydrographs of actual and future urbanization at CPRM.

differences between present and future hydrographs (5% in peak and 7% in surface volume) because the rainfall was concentrated in two time steps after low intensity rainfall which saturated the soil.

CONCLUSION

The model used is a simplified approach to the complex problem of simulating the rainfall-runoff process in an urban basin. A more complete model requires more data than are usually available. A simplified model requires sound procedures to estimate future changes in basin parameters due to urbanization.

It is necessary to know the effect of urbanization in order to prevent future flood problems and to prepare solutions which can be implemented before the urbanization takes place.

The model used is valuable because it studies the flood behaviour using a reasonable distribution procedure, thus allowing a good fitting of the recorded data. In order to simulate the effects of urbanization in the basin, some estimates have to be made for parameter modifications. In this type of model, the changes in impervious area and in the basin routing parameters were estimated based on the development proposed for the area.

The results of the recorded rainfall events showed that the effect of urbanization on flow distribution depends both on rainfall intensity and on its distribution.

For the reach studied there is no out-of-banks flow with an event of the order of a 50-year return period. The reach downstream from the CPRM flowgauge will also be studied in order to evaluate future consequences.

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REFERENCES

Alvarez, C. & Sanchez, O.J. (1979) Estudos hidrológicos da urbanizacão da bacia do Arroio Dilúvio; Relatório final.

Crawford, N.H. & Linsley, R.H. (1966) Digital simulation in hydrology; Stanford watershed model IV. *Univ. Stanford, Dept Civ. Engng, Tech. Report 39.*

Harley, B.M., Perkins, F.E. & Eagleson, P.S. (1970) A modular distributed model of catchment dynamics. *MIT, Ralph M.Parsons Lab., Report 133.*

HEC (1983) *HEC-1 Flood Hydrograph Package.* Hydrologic Engineering Center, Davis, California.

Li, R.M., Simon, D.B. & Stevens, M.A. (1975) Non linear kinematic wave approximation for water routing. *Wat. Resour. Res.* 11 (2), 245-252.

Metcalf & Eddy Inc./Univ. Florida/Water Resources Engineers Inc. (1971) *Environmental Protection Agency Storm Water Management Model* (4 vols). EPA, Washington, Report 11024 DOC-07/71.

Muñoz, R.H. & Tucci, C. (1974) Modelo matemático do Arroio Chasqueiro, Relatório Técnico, Instituto de Pesquisas Hidráulicas, UFRGS, Porto Alegre, RS, Brasil.

National Water Council (1981) *Design and Analysis of Urban Storm Drainage, the Wallingford Procedure.* Department of Environment, National Water Council, London.

Tucci, C.E.M. & Clarke, R.T. (1980) Adaptive forecasting with a conceptual rainfall-runoff model. In: *Hydrological Forecasting* (Proc. Oxford Symp., April 1980), 445-454. IAHS Publ. no. 129.

Tucci, C., Sanchez, J. & Simoes Lopes, M. (1981) Modelo matemático precipitacão-vazão IPH II, Recursos Hídricos. *Publ. no. 3, Instituto de Pesquisas Hidráulicas, Universidade Federal do Rio Grande do Sul.*

USDA (1969) Computer program for project formulation hydrology. *USDA Tech. Release SCS-TR-55, Washington, DC.*

Received 10 November 1982; accepted 29 July 1983.