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A GENERALIZATION OF CLARK´S IUH FOR FLATLAND AREAS WITH STRONG HUMAN INTERVENTIONS

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We present an improvement of Clark's Instantaneous Unit Hydrograph (IUH), which we call the "GAIUH", linking hydrological parameters and geomorphologic characteristics of flatland basins with strong human interventions. Partition of the effects caused by the direct runoff routing and attenuation due to natural hydrographic networks and structures like culverts are taken into account. A one parameter time area concentration curve as a function of concentration time of the basin is proposed for runoff routing. Two linear reservoirs are considered for attenuation effects. The linear storage coefficient for the culvert reservoir is obtained as a function of geomorphologic features of the reservoir, and hydraulic parameters of the culvert and inflow hydrograph. The proposed GAIUH model was applied to two hydrological systems of the flatland region in Argentina. Simulated and measured stream flow data are compared for a group of storms presenting a good correlation. The GAIUH model has given better results than other mathematical models applied to the hydrological systems.

INTRODUCTION

Drainage basins in flatland regions are characterized by low slopes and vast areas with significant storage capacity. In addition, man's interference through road construction, artificial channels, etc. produces radical changes in the natural hydrological response of the system. Because of that, and depending on the rainfall characteristics, flatland drainage basins can display a great diversity of runoff patterns.

Some concepts developed and tested in mountain basins cannot be directly transferred to understand the hydrological processes in flatland drainage systems. In fact, the instantaneous unit hydrograph (IUH) proposed by Rodriguez Iturbe et al. (1982) was applied to the Ludueña system (typical flatland basin of Santa Fe province, Argentina), and the simulated peak discharges were four times greater than observed (Zimmermann & Caamaño, 1996). The disagreement was mainly associated to the particular geomorphologic features of the basin and the huge anthropogenic intervention.

Low slopes and surface storage leads to important attenuation in the hydrographs. Overland flows arrive slowly and last enough time to add the contribution of flow from areas far from the channels, in contradiction with the concept of partial areas.

Moreover, since there is strong human interference (channelling, roads, culverts, etc.) in flatland systems the network is not a reflection of the climate. As a result, an important disparity between the reality and the hypothesis established in the model equations can be observed. To avoid this, the comparative hydrology concept (Falkenmark and Chapman, 1989) should be considered.

In this work, relationships between the IUH, and geomorphologic and "anthropo-geomorphological" characteristics (GA), are proposed for hydrological systems placed in flatland areas.

ASSUMPTIONS

The "anthropo-geomorphological" term is used here to represent man's interventions that can create serious constraints and cause radical changes in the natural hydrological response of the system. Usually they are not considered in the formulation of relationships between geomorphology and hydrology. In particular, the effects of storage caused by small culverts are studied here. Clark's theory of IUH (Clark 1945) is accepted. In this approach, the IUH is considered to be the product of routing a time area concentration diagram (TAC curve, as the distribution model) through a reservoir (as the attenuation model) having a linear storage coefficient (K) derived from geomorphologic characteristics.

The distribution and attenuation concepts are used in the present IUH formulation, which is termed the Geomorphologic and Anthropomorphologic Instantaneous Unit Hydrograph (GAIUH). Determination of a triangular TAC curve, derived from basin drainage parameters, and linear reservoirs having coefficients depending on network and flow control characteristics, is proposed.

The GAIUH takes into consideration distribution effects of runoff by means of a TAC curve, network storage effects by means of a linear reservoir (K_1) located at the outlet and flow control effects through a second linear reservoir (K_2), placed below of the first reservoir.

PROPOSED TAC CURVE

Assuming the TAC curve with a triangular shape, the following deductions can be made:

- The base of the triangle is equal to the concentration time (T_c) of the drainage basin. This can be stated because the last isochrone, which corresponds to the total basin area, is equal to its concentration time.
- Since the TAC diagram represents the IUH, its volume must be equal to the unity. In this case, the height of the diagram, q_p, is given by:

$$q_p = \frac{2}{T_c} \tag{1}$$

- In order to complete the curve, the temporal location of the peak (t_p) must be determined. This is done by using the following similarity equation (Rodriguez Iturbe et al., 1979):

$$q_p\, t_p = 0.58 \left(\frac{RB}{RA} \right)^{0.55} \tag{2}$$

where RB is the bifurcation ratio and RA the area ratio. According to Rodriguez Iturbe et al. (1979), a probabilistic value of 0.8 can be used for RB/RA. In this case, solving for t_p and using Equation (1), the following relation is obtained:

$$t_p = \frac{0.51}{2} T_c \approx \frac{T_c}{4} \tag{3}$$

It can be observed that all the parameters involved in the TAC curve depend on T_c. Clearly, it is important to correctly estimate this last parameter.

LINEAR RESERVOIR ASSOCIATED WITH NETWORK ATTENUATIONS

To determine the reservoir routing constant, K_1, we investigated available methodologies that explicitly use local geomorphologic characteristics. In addition, those methods requiring observed data for their calibration were excluded from the analysis since, in general, we wish to infer the hydrological behavior of the system using only the dominant geomorphologic features. When there is no flow data, empirical formulas, e.g. K/T_c = 0.5-0.8, are recommended. O'Kelly (1955) arrived at the following relationships for T_c, based on the triangular IUH, and for storage coefficient K in the linear reservoir:

$$T_c = aA^{\frac{1}{4}}S^{\frac{1}{2}} \tag{4}$$

$$K = bA^{\frac{1}{4}}S^{\frac{1}{2}} \tag{5}$$

where a and b are constants, A is the drainage area (in km^2), and S is the slope (in percent). Combining the preceding equations shows that K/T_c is a constant equal to the ratio b/a. O'Kelly presented values of a and b for a range of regional basin slopes. Using the O'Kelly data for the regional slope of the hydrological system studied, which is about 1 m/km, the ratio a/b is equal to 1.3. Based on the O'Kelly value, the K/T_c ratio can range from 0.5 to 1.3.

LINEAR RESERVOIR ASSOCIATED WITH CULVERT ATTENUATION

Attenuation effects caused by flow controls, like small culverts, are simulated by means of a simple linear reservoir formulation.

The degree of attenuation will depend on the characteristics of the inflow hydrograph, topographic characteristics (storage curve) and the characteristics of the flow control structures (bridges, culverts, etc.). All these effects should be collapsed into the constant K for a linear reservoir.

The routing of a hydrograph through a reservoir is usually solved numerically due to the difficulty of defining it analytically as a time function. The solution must satisfy the continuity equation, under an initial condition, together with a boundary condition, such as the stage-discharge relationship of the culvert, $Q_s(z)$, and a geometric internal relationship such as the storage curve $V(z)$:

$$\frac{dV(t)}{dt} = Q_i - Q_s \tag{6}$$

$$V = V(z) = \phi_1(z) \tag{7}$$

$$Q_s = Q_s(z) = \phi_2(z) \tag{8}$$

where $V(z)$ is the storage in the reservoir, t the time, Q_i and Q_s are the inflows and outflows respectively, and $z(t)$ is the water elevation in the reservoir at time t. Putting the equations as a function of z, and discretizing Equation (6) in time steps Δt, one gets:

$$\phi_1\left(z^{n+1}\right) = \frac{Q_i^n + Q_i^{n+1}}{2}\Delta t - \frac{\phi_2\left(z^n\right) + \phi_2\left(z^{n+1}\right)}{2}\Delta t + \phi_1\left(z^n\right) \tag{9a}$$

with the initial conditions,

$$V_0 = V\left(z^0\right) = \phi_1\left(z^0\right) \tag{9b}$$

where n= 1,2,3... represent the level of time $t = n\, t$. Since the equation is implicit in z^{n+1}, it is solved iteratively. With known z^{n+1}, we can evaluate the variables $V= \phi_1(z)$ and $Q_s = \phi_2(z)$.

The ϕ_1 and ϕ_2 functions can be proposed by means of the following relationships:

$$\phi_1(z) = A\left(z - z_0\right)^B \tag{10}$$

$$\phi_2(z) = \mu\sqrt{2g}\,F\sqrt{z - z_v} = C\sqrt{z - z_v} \quad where\ z > H + z_v \tag{11}$$

where z_0 is the bottom elevation of the reservoir, z_v is the bottom elevation of the culvert, F is the wetted area, μ is a discharge coefficient, g is the gravity acceleration, and A and B are constants. Chow (1959) used a power relationship such as Equation (10) for reservoir routing. Equation (11) corresponds to a discharge curve of a submerged culvert with upstream control (French, 1988).

A theoretical approach was taken with the purpose of establishing an analytic relationship between the constant K_2 and the remaining parameters. It was not possible to find a theoretical expression as a function of the discharge law for the culvert or the storage function. Accordingly, K_2 should be obtained numerically, preserving only some characteristics of the outflow hydrograph, such as the peak flow.

Numerical determination of K_2

The constant K_2 was derived numerically by routing of the inflow series, produced by basins with different geomorphologic parameters, through reservoirs with diverse characteristics of storage and control structures. At the same time, equivalent K_2 values, which minimized the errors in the calculated outflow peaks, were derived (Figure 1). The numerical experiments were characterized by:

- an inflow hydrograph with the following parameters: T_p as time to peak, Q_p as inflow peak, and T_b as base time;
- an elevation-storage relationship defined by a potential function of parameters A, B and z_0 (although z_0 was proposed as 0), in accordance with Equation (10);
- a boundary condition defined by the discharge curve of the control structure (e.g. culverts, bridges, etc.) having the area, F, and the discharge coefficient, m, as fundamental parameters;
- an initial condition represented by an empty reservoir; and
- a numerical solution of Equations (9a) and (9b).

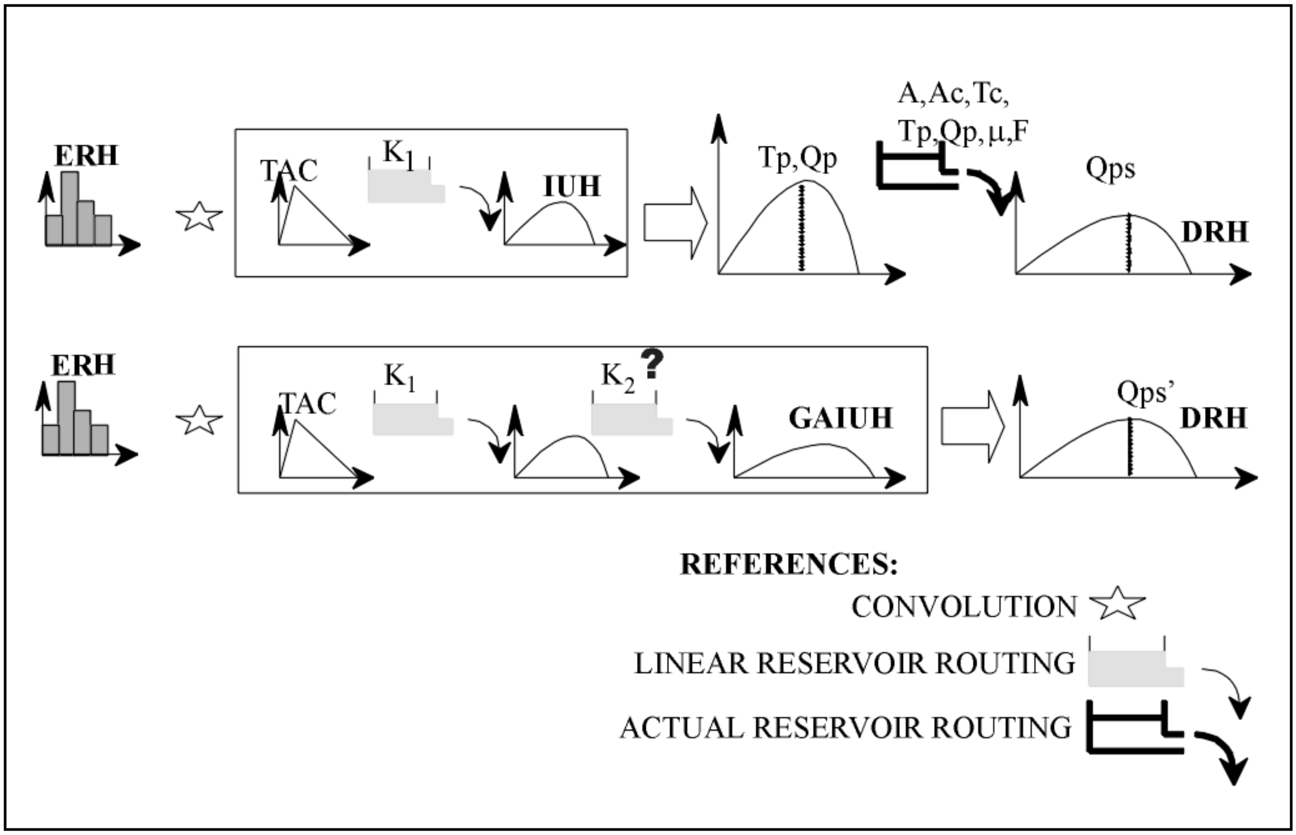

Figure 1. Instantaneous unit hydrographs.

The time step used was equal to $T_p/20$, in order to guarantee a reasonable number of points in the rising segment of hydrograph. The iteration process included, for each time step, calculation of reservoir elevation, and iteration was stopped when successive values for the calculated reservoir level showed a relative difference of less than 1/10000.

Inflow Hydrographs

Inflow hydrographs were generated in relation to the geomorphologic characteristics of hypothetical basins using rainfalls as inputs. The geomorphologic characteristics were similar to those found in the flatland basin hydrological systems under study. The features considered for the basins are shown in Table 1.

Table 1. Mean Features of Basins Considered in the Numerical Simulation

Basin N.	Area km^2	Slope m/km	T_c hs	t_p hs	q_p hs^{-1}	K_1 hs
1	10	3.60	3	0.75	0.666	3.0
2	30	2.70	6	1.5	0.333	6.0
3	80	1.80	12	3.0	0.166	12.0
4	180	1.00	24	6.0	0.083	24.0
5	450	0.65	48	12.0	0.042	48.0

Concentration times were calculated using Ventura's formula for simplicity: $T_c = \alpha\,(A_c/S)^{0.5}$, where A_c is the drainage area (in km^2), S is the slope (in m/km) and α is a coefficient. A coefficient α equal to 0.06 was used. This value was based on estimations of T_c using recession curves of observed hydrographs (Zimmermann ,1997). The values of q_p and t_p were obtained by applying Equations (1) and (3), and K_1 was obtained by assuming $K_1 = T_c$. The set of basins represents channel-networks from order 1 to order 4, in accordance with the characteristics of the actual systems studied. The runoff from precipitation was considered under two extreme events: a) maximum event with 50-yr return period and 48 hr duration. b) minimum event with 2-yr return period and 3 hr duration. In both situations, the hyetograph was constructed using the Chicago method for storm design. The time interval of hyetographs was equal to two hours. For the hyetograph construction, the depth-frequency-duration curves of Rosario Aero (series 1942-1985), were used. Effective rainfall hyetographs (ERH) and runoff were estimated using the U.S. Soil Conservation Service (SCS) method, with CN = 70. This agrees with the regional soils under antecedent moisture condition II (Zimmermann, 1988). Convolutions between the ERH and the Clark´s IUH, whose parameters were defined in Table 1, gave the flood hydrographs shown in Table 2.

Table 2. Characteristics of Events Used in the Numerical Simulation

Basin	Minimum Storm T = 2-year)			Maximum Storm (T = 50-year)		
	Q_p m^3/s	T_p hs	Volume m^3	Q_p m^3/s	T_p hs	Volume m^3
1	6.3	3.7	90,000	60.0	6.9	1,122,000
2	11.3	5.3	270,000	116.9	8.5	3,366,000
3	15.8	9.0	720,000	183.4	12.7	8,976,000
4	18.0	17.4	1,620,000	220.4	21.0	20,196,000
5	22.5	33.0	4,050,000	279.4	36.9	50,490,000

Propagation through reservoirs

Inflow hydrographs were propagated through reservoirs with different geomorphologic and hydraulic parameters (see upper branch in Figure 1). The established range for these parameters was related to the characteristics of the modeling systems. The transverse areas of culverts ranged between 1 m^2 (small rural culverts) to 50 m^2 (bridges). Two additional parameters were also included: the aspect ratio, *RELH,* and discharge coefficient, μ. By varying the reservoir parameters A and B (Equation 10), a wide range of volumes were considered.

In summary, nine independent variables were considered as reservoir parameters: Q_p and T_p (peak flow and peak time of inflow hydrograph), T_c and A_c (time of concentration and basin area), F, μ and *RELH* (transverse area, discharge coefficient and aspect ratio), and A and B (coefficients of reservoir geometry). This group of independent variables was identified as Option A. An alternative expression, called Option B, was proposed, where μ, *RELH*, and F were grouped into one parameter, called full discharge (Q_f), defined as a discharge of culvert working with submerged conditions upstream and free flow downstream. The ranges considered for all parameters are shown in Table 3.

Table 3. Range of Parameters

Parameter	Dimension	Minimum Value	Maximum Value
A	-	100,000	1,700,000
B	-	1	3
μ	dimensionless	0.3	0.8
RELH	dimensionless	1	10
F	L^2	1 m^2	50 m2
Q_f	L^3T^{-1}	0.42 m^3/s	1,253 m^3/s
A_c	L^2	10 km^2	450 km^2
T_C	T	3 hs.	48 hs.
T_p	T	3.7 hs	36.9 hs
Q_p	L^3T^{-1}	6.3 m^3/s	279.4 m^3/s

The lower branch in Figure 1 shows the internal procedure in the GAIUH; the hydrograph is obtained by means of convolution between ERH and GAIUH. The GAIUH was obtained by propagating a TAC curve through two linear reservoirs in series, one representing geomorphologic attenuation (with constant K_1), and the other representing culvert attenuation (with constant K_2). The optimal constant K_2 was selected by minimizing the absolute errors of peak flows (Q_{ps} and Q'_{ps}) obtained as output of propagation through actual and linear reservoirs. Since this process implied the search of roots of an objective function, a bisection algorithm was used with a tolerance of 0.01 m^3/s.

Discretizing the range of parameters (Table 3) and taking the possible combinations between them, a group of 650 values of parameters and optimal K_2 was generated. Firstly, a multiple linear regression showed poor results. Sensitivity analyses with non-linear regressions, showed that the parameters *RELH* and *B* are not significant in the process and consequently were removed.

Dimensional Analysis

In order to guarantee dimensional homogeneity, the π-theorem was applied. The relationship ϕ_1 (Equation 10) was normalized. In nature, this relationship is a quadratic or cubic polynomial. In this work, a cubic exponent was adopted, and consequently, A is a dimensionless coefficient. In a previous work (Zimmermann, 1997) this coefficient was reciprocally connected to the longitudinal and transverse slopes of the reservoir geometry.

Five dimensionless parameters for Option A and four for Option B were determined as a result of π-theorem application. These variables were separated into dependent and independent groups. Multiple non-linear correlation between dimensionless variables was carried out. The following equations were fitted from 650 pairs of dependent and independent variables:

$$\frac{K_2}{T_c} = 0.0412 \left(\frac{F}{A_c}\right)^{0.7114} \left(\frac{T_c Q_p}{F^{3/2}}\right)^{1.0668} \left(\frac{T_p}{T_c}\right)^{1.3732} A^{0.1753} \mu^{-1.0248} \tag{12}$$

for Option A $\left(r^2 = 0.9775\right)$, and

$$\frac{K_2}{T_c} = 0.2828\left(\frac{T_f T_c}{A_c^{3/2}}\right)^{0.0574}\left(\frac{T_p}{T_c}\right)^{0.67} A^{0.1719}\left(\frac{Q_p}{Q_f}\right)^{0.8341} \tag{13}$$

for Option B $\left(r^2 = 0.9193\right)$.

REGIONAL APPLICATION OF THE GAIUH MODEL

In order to verify the GAIUH model's ability to predict the hydrological response of real systems, simulated hydrographs were compared with stream flow data measured in the Ludueña and Pavón basins, which are representative of flatland regions of Argentina.

Ludueña Basin

The Ludueña basin covers an area of 700 km^2 in southern Santa Fe province, Argentina (61°W, 32°S) (Figure 2). The average basin slope is 1.4 m/km. The annual mean precipitation is about to 950 mm. Stream flow data are available (Zimmermann, 1988) for three periods: 1969-1971 and 1982-1984 at the Circunvalación gauging station, and 1978-1979 at the Golf Club gauging station. Daily data from five precipitation measurement stations and hourly data from two recording gages were used. Sixteen storms and their respective flow discharges in the outlet of basin were selected. Effective rainfall hyetographs, with time steps of 2 hrs, were estimated applying the SCS method, where the curve numbers (CN) were fitted with runoff measurements. The selected events are shown in

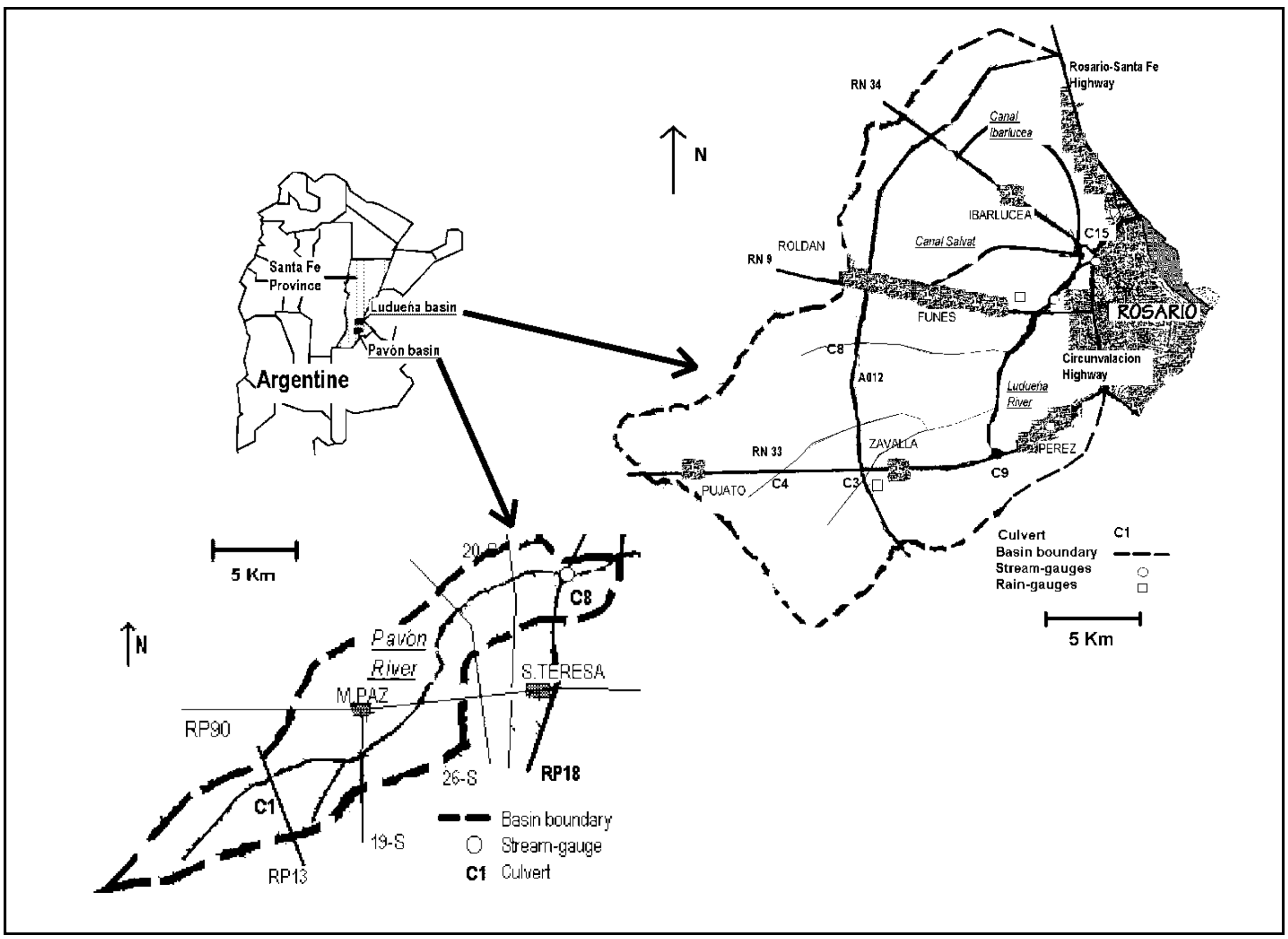

Figure 2. Ludueña and Pavón basin locations.

Table 4. Storm and Discharge Characteristics for the Ludueña Basin

Storm N.	Date	Duration hrs	Runoff mm	Fitted CN	Q_p obs m^3/s
1	26.11.69	2	4	67.1	12.6
2	04.02.70	4	0.86	64.3	2.7
3	15.03.70	14	2.3	47.4	13
4	13.05.70	4	0.3	53.6	3.4
5	27.09.70	5	1.3	56.2	7.8
6	01.10.70	4	2.3	74.2	10.2
7	30.10.70	8	3.4	64.5	16.5
8	27.12.70	8	4	58.4	12.1
9	01.12.78	10	9.9	44.5	25.0
10	03.12.78	8	16.5	58.3	35.0
11	04.03.79	24	32.3	43.8	60.0
12	09.11.83	6	7.8	64.4	30
13	12.11.83	6	5.6	71.3	32.5
14	21.02.84	10	5	43.2	16.8
15	28.02.84	14	46.6	75.8	205.0
16	15.03.84	4	2.8	67.5	19.6

Table 4.

The identification of the culverts that cause obstructions to flow was carried out. For each culvert, geometric and hydraulic parameters (μ, F and Q_f), hydrological parameters of its drainage sub-basin (A_c and T_c), and a geomorphologic parameter of its reservoir (A), were obtained. For each storm and each culvert, hydraulic parameters of the inflow (T_p and Q_p) were obtained.

The discharge coefficient and full discharge of the culverts were calculated by means of the Federal Highway Administration (USA) methodology, where overtopping situations were contemplated. The HYDRAIN-HY8 computer program was used (GKY & Associates, Inc., 1992). Drainage areas of culverts and storage laws of reservoirs were defined using maps on a scale of 1:50 000. Concentration times were calculated by means of the Ventura equation. Five culverts were considered as effective restrictions to flow.

The coefficients of the cubic storage relationship, A, were estimated by means of integration of reservoir volumes (V) over maps on scale 1:50 000 and forcing a cubic relationship:

$$A = \frac{V}{\left(z_{\max} - z_v\right)^3} \tag{14}$$

Table 5. Culvert and Sub-basin Parameters for the Ludueña Basin

Sub Basin	Outlet	Roads	A_c km^2	T_c hs	F m^2	A Dimensionless	Q_f m^3/s	μ Dimensionless
1	Culvert 3	A012	51.3	7.9	8.0	66,400	11.0	0.44
2	Culvert 4	RN 33	15.6	11.3	1.1	81,600	1.4	0.20
3	Culvert 8	A012	114.9	14.4	14.4	432,000	13.7	0.54
4	Culvert 9	RN 33	77.9	10.6	6.0	1,344,000	21.5	0.51
5	Bridge RN 9	RN 9	190.3	22.1	163.2	1,562,500	500.0	0.85
6	Culvert 15	RN 34	175.3	25.7	4.6	93,300	16.6	0.53

where z_{max} and z_v are the roadway elevation and bottom elevation of culvert, respectively. The parameters are shown in Table 5.

Pavón Basin

This basin covers an area of 452 km^2 in southern of Santa Fe province (61°W, 33°30'S). Average basin slope is 0.6 m/km. Annual mean precipitation is about to 950 mm (Figure 2). Stream flow data are available (Postiglione et al., 1989) for one period, 1978-1984 at the RP18 gauging station with daily steps. Daily data from five precipitation measurement stations and hourly data from one recording gauge were used. Fourteen storms and their respective flow discharges in the outlet of basin were selected. Effective rainfall hyetographs, with six-hour intervals, were estimated applying the SCS method. The selected events are shown in Table 6.

Table 6. Storm and Discharge Characteristics for the Pavón Basin

Storm N°.	Date	Duration hs.	Runoff mm	Fitted CN	Q_p estimated m^3/s
1	25.01.78	12	38.4	73.5	90
2	12.03.78	6	8.5	43.9	22
3	21.03.78	18	63.5	85.7	90
4	23.09.78	12	42.3	75.0	65
5	01.12.78	12	45.3	76.0	85
6	03.03.79	6	11.8	75.0	14
7	09.12.79	18	51.8	67.3	85
8	12.05.80	6	56.2	85.0	63
9	31.01.81	12	42.3	69.5	88
10	07.02.81	6	11.0	65.4	25
11	07.05.81	18	44.7	87.0	103
12	19.02.82	6	11.3	66.2	70
13	08.09.82	12	7.7	57.0	21
14	29.01.84	6	16.7	51.3	100

Table 7. Culvert and Sub-Basin Parameters for the Pavón Basin

Sub Basin	Outlet	Roads	A_c Km^2	T_c hs	F m^2	A Dimensionless	Q_f m^3/s	μ Dimensionless
1	Culvert 1	RP 13	86.1	18.8	28.0	106,000	34.0	0.33
2	Culvert 8	RN 90	366.2	27.6	260.1	112,000	750.4	0.29

The flow control structures were identified. All the parameters were calculated using the method applied to the Ludueña basin. One culvert was considered as an effective flow restriction. Its parameters are shown in Table 7.

Operative methodology

In both applications, to the Ludueña and Pavón basins, after the flow control culverts were identified and their parameters were estimated, the sequence of calculation was:

- For each rainfall selected event and each sub-basin, hydrographs were generated by convolution between ERH and IUH, without culvert attenuation. Then, peak flow Q_p was compared with the full discharge of culvert, Q_f.

- If $Q_p > Q_f$ then a second linear reservoir with constant K_2 was considered in the IUH structure and a new convolution was done. If not, only the linear reservoir with constant K_1 representative of network attenuation was considered.
- Outflow hydrographs of sub-basins were summed in order to determine the hydrograph at the outlet of the whole basin.

RESULTS AND DISCUSSION

Different constant K_1 values were proposed in order to determine the best identification between model and prototype. Six alternatives were checked combining two options for K_2 estimation (Equation 12 or Equation 13) and three options for K_1 estimation ($K_1 = 0.5T_c$; $K_1 = T_c$ and $K_1 = 1.3T_c$).

For comparison purposes, relative (RAE) and quadratic (QAE) average errors in the approach for peak flows were used as follow:

Table 8. Results for the Ludueña Basin

	RAE (%)			QAE (m^3/s)		
	$K_1 = 0.5\ T_c$	$K_1 = T_c$	$K_1 = 1.3\ T_c$	$K_1 = 0.5\ T_c$	$K_1 = T_c$	$K_1 = 1.3\ T_c$
K_2 with eq. 12	98.0	29.4	36.3	18.9	7.1	9.5
K_2 with eq. 13	96.0	28.3	31.9	17.5	6.2	6.5

Table 9. Results for the Pavón Basin

	RAE (%)			QAE (m^3/s)		
	$K_1 = 0.5\ T_c$	$K_1 = T_c$	$K_1 = 1.3\ T_c$	$K_1 = 0.5\ T_c$	$K_1 = T_c$	$K_1 = 1.3\ T_c$
K_2 with eq. 12	52.7	27.6	24.6	41.6	25.7	24.8
K_2 with eq. 13	51.3	27.8	24.7	41.0	25.8	24.8

$$RAE(Qp) = \frac{1}{N}\sum_{i=1}^{N}\frac{abs(Qp_{ci} - Qp_{oi})}{Qp_{oi}} \qquad Quae(Qp) = \sqrt{\frac{1}{N}\sum_{i=1}^{N}(Qp_{ci} - Qp_{oi})^2} \qquad (15)$$

where *ci* and *oi* are calculated and observed values for the storm *i*, respectively, and *N* is the total of the events. The results are summarized in Tables 8 and 9.

In Tables 8 and 9, an important conclusion about K_1 can be observed. In all cases, a linear reservoir with constant K_1 in the range from T_c to $1.3T_c$ generates the best results. This indicated a regional tendency for the parameter. For the Ludueña basin, storm 15 (maximum recorded) was studied separately from the others. Overtopping in culverts 3, 4, 9 and 15 occurred in this event. This effect was simulated in the model by means of an increase of the full discharges Q_f, calculating the flow over embankment tops.

Figure 3 shows comparisons between simulated and observed hydrographs.

The evaluation of the constant K_2 with Equation 13 did not show a clear supremacy over Equation 12, but it offers some strategic advantages:

- it concentrates geometric and hydraulic characteristics of the culverts in an only variable, Q_f, simplifying the regression equation;
- it allows overtopping of a culvert (like in the storm 15, in Ludueña's application); and
- it allows different structures of flow control, knowing their discharge capacity.

Table 10 shows a comparison with other models which were applied to Ludueña and Pavón basins.

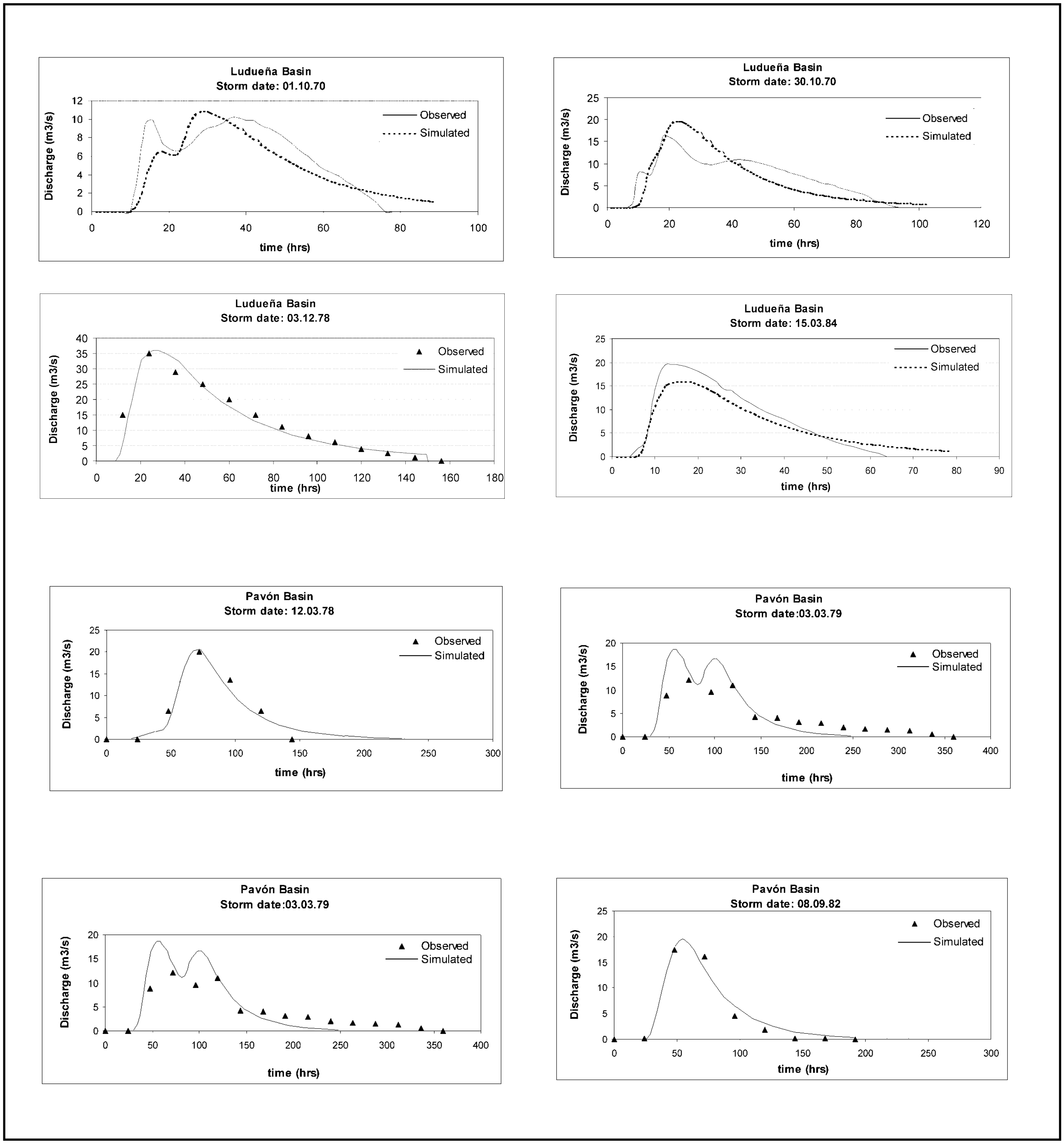

Figure 3. Comparisons of simulated and observed hydrographs.

CONCLUSIONS

A generalization of Clark's IUH, adequate for flatland areas with strong human interventions, called the Geomorphologic and Anthropomorphologic Instantaneous Unit Hydrograph (GAIUH) was proposed. Characteristic elements of Clark's IUH, such as the TAC curve and linear reservoirs in series, were used in order to quantify the effective rainfall routing, and network and culvert attenuation effects. All parameters were deduced from geomorphologic and anthropogenic variables. The TAC curve proposed, of triangular shape, has only one parameter, T_c, which is the base of the triangle. Ventura's formula was used to estimate T_c, and the linear reservoir, which represents network attenuation, was proposed with a constant K_1 ranging from 0.5 T_c to 1.3 T_c.

Table 10. Results for Different Models Applied to the Hydrological Systems Studied

Basin	Model	QAE (Q_p) m^3/s	Reference	Observations
Ludueña	SOGREAH (BILYK)	54.7	Zimmermann (1988)	Calibrated parameters
	OCINE (Schaake, 1971)	9.9	Zimmermann (1988)	Calibrated parameters
	GAIUH	6.2	Zimmermann (1997)	Without calibration
Pavón	HYMO	32.7	Navarro (1993)	Calibrated parameters
	GAIUH	24.8	Zimmermann (1997)	Without calibration

A linear reservoir with constant K_2, in series with the previous one, was adopted for culvert attenuation. The constant K_2 was connected to laws of discharge and storage of an actual reservoir caused by the culvert, which represents a flow control structure. In order to obtain an optimal constant for a linear "equivalent" reservoir, numerical routing was carried out for actual reservoirs, which included a wide range of drainage areas and climatic events. A dimensional analysis was done for the set of variables and dimensionless parameters were proposed. Multiple non-linear regression between dimensionless parameters was carried out, and two equations, including different parameters, were selected as the best representing a good degree of correlation.

The comparison of the GAIUH hydrological response with stream flow data of two typical systems of flatland areas was presented. The GAIUH application in the Ludueña and Pavón basins showed a good result for the prototype model.

A regional tendency for K_1 was indicated, which ranged from T_c to $1.3T_c$. It was shown that is suitable to define the constant K_2 in terms of the culvert full discharge, since it presents an operative simplicity and it can be adapted to different flow control structures.

Finally, the GAIUH model gave better results than other models previously applied in these hydrological systems. Considering these results, the GAIUH model constitutes a contribution to the knowledge of the relationships between geomorphology and hydrology for flatland areas and with an important degree of man's intervention.

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