# Derived flood frequency distribution and sensitivity analysis to variations in model parameters

Alejandra I. Vornetti and Rafael S. Seoane

## ABSTRACT

In this paper we propose a new derived flood frequency distribution approach using the Nash instantaneous unit hydrograph (IUH) as the catchment's response model and analyze the sensitivity of flood frequency distributions to variations in the model parameters. We analyzed the effect of applying the Nash IUH to estimate peak flows in two basins in the province of Buenos Aires (Argentina). For the new flood frequency distribution, the Nash IUH parameters were estimated using the method of moments and a technique that relates the IUH parameters to the basin geomorphologic and hydraulic features. After applying the proposed derived flood frequency distributions to the basins, it is concluded that the new model (which includes the Nash IUH) yields similar results to those attained with a model developed by Raines and Valdés; the new model parameters can be estimated using a technique for ungauged basins. Also observed is a change in the shape of the flood frequency curves according to the IUH parameter estimation technique and to the set of infiltration parameters used.

**Key words** | derived flood frequency distribution, instantaneous unit hydrograph, parameters, peak flows

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## INTRODUCTION

The derived density function applied by Eagleson (1972) incorporates three models (rainfall, infiltration and catchment response) and has been modified by Raines & Valdés (1993) and Seoane & Valdés (1993), among others.

Eagleson (1972) developed an expression for the flood frequency distribution by combining a rainfall model that includes an exponential distribution of rainfall intensity and duration, a constant temporal infiltration rate and kinematic wave theory as the catchment response model. This work had logical continuity from the models developed by Hebson & Wood (1982) and Díaz-Granados *et al.* (1984) that included the new theories of the geomorphologic (Rodríguez Iturbe & Valdés 1979) and geomorphoclimatic (Rodríguez Iturbe *et al.* 1982) instantaneous unit hydrographs (IUH) in the model initially proposed by Eagleson (1972).

The derived distribution approach proposed by Raines & Valdés (1993) uses a rainfall model similar to that of Eagleson; the Soil Conservation Service (SCS) model was used to obtain runoff and the geomorphoclimatic IUH as the catchment response.

The proposal of Raines & Valdés (1993) estimates precipitation excess with the SCS Curve Number (CN) method to derive the joint probability density function of effective rainfall intensity and duration from the exponential marginal probability density functions of total intensity and duration.

Finally, the authors developed an expression of the cumulative distribution function (CDF) of peak flow using the geomorphoclimatic IUH as the catchment response model and integrating the joint probability density function of effective rainfall intensity and duration from an approximation proposed by Díaz-Granados *et al.* (1984).

An antecedent in the analysis of uncertainty in the abstraction model parameters was provided by De Michele & Salvadori (2002), who present an analytical formulation of the derived distribution of peak flood. The SCS CN model is used to describe runoff processes and the authors analyze the

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influence of antecedent soil moisture conditions on the flood frequency distribution.

Gottschalk & Weingartner (1998) derive an expression for the distribution function of peak runoff, combining results of frequency analysis of rainfall volumes with runoff coefficients and the unit hydrograph. Rainfall volume is assumed to follow a gamma distribution, whereas a beta distribution is applied for the runoff coefficient.

Goel *et al.* (2000) developed a derived flood frequency distribution (DFFD) assuming that rainfall intensity and duration are correlated. Rainfall–runoff processes are modelled using a $\phi$-index infiltration model and a geomorphoclimatic IUH. The authors concluded that the correlation of rainfall intensity and duration has an important impact on the DFFD.

Fiorentino *et al.* (2007) analyzed the relative effects of different hydrological processes acting at the basin scale in different hydroclimatic areas of the world. The authors analyzed results of a theoretically DFFD obtained by Iacobellis & Fiorentino (2000) and compared these to the results of a simulation scheme that uses a distributed hydrological model in cascade with a rainfall generator. The performed simulations allowed a better understanding of the hydrological behaviour of basins during extreme events, with particular reference to the processes controlling runoff generation mechanisms and the spatial pattern of flood-peak contributing areas.

Gioia *et al.* (2008) generalized the analytical scheme for DFFD proposed by Iacobellis & Fiorentino (2000) and two different threshold-driven mechanisms of runoff generation, associated with ordinary and extraordinary events, were included in order to explain highly skewed annual maximum flood distributions. The model was tested on a group of basins in Southern Italy characterized by annual maximum flood distributions highly skewed. The application of the proposed model provided good results.

Kurothe *et al.* (2001) developed a physically based DFFD. This model uses bivariate exponential distribution for rainfall intensity and duration, and the SCS CN method for deriving the probability density function (PDF) of effective rainfall. The effective rainfall–runoff model is based on the kinematic wave theory. As the use of the CN method is simple and the data required for estimation of curve numbers are readily available, the authors conclude that the proposed model provides an alternative approach for flood frequency determination and produces reasonable results.

Loukas (2002) proposed an event-runoff simulation procedure based on the method of derived distributions for the estimation of flood frequencies for ungauged watersheds. The procedure uses a stochastic rainfall generation model and a rainfall–runoff watershed model. The Monte Carlo simulation is used for the generation of the various parameter values and simulation of the flood hydrographs. The frequencies of the hourly and daily peak flow and the flood volume are estimated. After applying the proposed procedure in British Columbia, the author found that the results compared well with the observed data and with the Extreme Value type I fitted probability distribution. The author concluded that the method is easy to apply, requires limited regional data and is reliable for small and medium forested watersheds.

Rahman *et al.* (2002) presented a Monte Carlo simulation technique that makes allowance for the probability distributed nature of the key flood-producing variables and the dependencies between them to determine the derived flood frequency curves. The new technique is easy to apply for catchments with good rainfall data and a limited streamflow record.

Arónica & Candela (2007) applied a Monte Carlo procedure for deriving frequency distributions of peak flows using a semi-distributed stochastic rainfall–runoff model. The rainfall–runoff model used is very simple, which makes it useful for poorly gauged catchments. The catchment response has been modelled by the SCS CN method in a semi-distributed form. The SCS CN method was implemented in probabilistic form with respect to prior-to-storm conditions. The procedure was tested on six case studies where synthetic flood frequency curves have been obtained. The application of this procedure showed how the Monte Carlo simulation technique can reproduce the observed flood frequency curves with reasonable accuracy over a wide range of return periods.

In this paper the authors have developed a new version of the DFFD approach that includes the Nash (1960) instantaneous unit hydrograph (IUH) as the catchment's response, and have applied it to the Tapalqué and Azul river basins located in the centre of the province of Buenos Aires (Argentina). A comparison is made of the goodness-of-fit with the empirical CDF of the new model and of the derived distribution approach proposed by Raines & Valdés (1993) for the above-mentioned basins.

The authors analyzed the sensitivity of the shape of flood frequency distributions to variations in infiltration and catchment response model parameters. Two versions of the DFFD were used: Raines & Valdés's model (1993) and the new proposal, which includes the Nash IUH. A sensitivity index was estimated to quantify the effect of known parameter variations on the estimated exceedance probabilities associated with observed peak flows.

The effect of applying the Nash IUH to estimate peak flows in these basins was then analyzed. Model parameters were estimated using the method of moments and a technique developed by Rosso (1984) that relates them to the catchment geomorphologic and hydraulic features. The results of applying the linear regression model to observed and estimated flows in these areas are presented.

## METHODOLOGY

From the proposal of Raines & Valdés (1993) which uses the geomorphoclimatic IUH as the catchment response model, a new flood frequency distribution was developed using the Nash IUH (1960).

The joint PDF of effective rainfall intensity and duration was obtained by Raines and Valdés and is presented in two parts. The probability of null runoff is:

$$\mathrm{Prob}\,[i_e = 0, t_e = 0] \approx 1 - \exp(-\sigma)\Gamma(\sigma+1)\sigma^{-\sigma} \tag{1}$$

The continuous part of the joint PDF of $i_e$ and $t_e$ is:

$$\begin{aligned} f_{i_e,t_e}(i_e, t_e) = {} & 0.77642\beta^*\lambda \exp(-\lambda t_e - \sigma)\sigma^{-\sigma} \\ & \times \Gamma(\sigma+1) S^{0.44161} t_e^{-0.44161} i_e^{-0.44161} \\ & \times \exp(-1.39047\beta^* S^{0.44161} t_e^{-0.44161} i_e^{0.55839}) \end{aligned} \tag{2}$$

where

$$\sigma = \lambda\left[\frac{0.2\beta^* S}{\lambda}\right]^{1/2}$$

$i_e$ is effective rainfall intensity (cm h$^{-1}$), $t_e$ is effective rainfall duration (h), $\lambda$ is the inverse of mean rainfall duration (h$^{-1}$), $\beta$ is the inverse mean point rainfall intensity (h cm$^{-1}$) and $K$ is a factor that converts point rainfall intensity to areal average rainfall intensity (Eagleson 1972), defined as:

$$K = 1 - \exp(-1.1\lambda^{-0.25}) + \exp(-1.1\lambda^{-0.25} - 0.003861A_\Omega)$$

$$\beta^* = \frac{\beta}{K}$$

where $A_\Omega$ is basin area (km$^2$) and $S$, the maximum potential surface retention (mm), is defined as:

$$S = \frac{25400}{\mathrm{CN}} - 254$$

where CN is curve number.

In order to estimate the peak flow ($Q_{max}$) exceedance probability, it is necessary to find the CDF of $Q_{max}$ which is given by:

$$F_{Q_{max}}(Q_{max}) = \iint_R f_{i_e,t_e}(i_e, t_e)\,\mathrm{d}i_e\,\mathrm{d}t_e \tag{3}$$

where $f_{i_e,t_e}(i_e, t_e)$ is the joint PDF of effective rainfall intensity and duration and $R$ is the region of the $i_e$, $t_e$ plane, where the convolution of $i_e$, $t_e$ with the catchment response model produces peak flows less than or equal to $Q_{max}$.

By using the Nash IUH as the catchment response model, integrating the joint PDF (Equations (1) and (2)) according to the approximation proposed by Díaz-Granados *et al.* (1984) and using a triangular IUH (Henderson 1963), the following expression of the CDF of peak flows is obtained:

$$F_{Q_{max}}(Q_{max}) = 1 - \lambda\exp(-\sigma)\Gamma(\sigma+1)\sigma^{-\sigma}(G + \Sigma H_i) \tag{4}$$

where

$$G = \int_{T^*}^{\infty} \exp(-\lambda t_e - 1.39047\beta^* S^{0.44161} Q_{max}^{*0.55839} t_e^{-0.44161})\,\mathrm{d}t_e$$

$$H_i = \int_{a_i T^*}^{b_i T^*} \exp\left(-\lambda t_e - 1.39047\beta^* S^{0.44161} \times \left[\frac{T^*(c_i Q^*_{max})^{d_i}}{t_e}\right]^{0.55839/d_i} t_e^{-0.44161}\right) dt_e$$

$$Q^*_{max} = \frac{Q_{max}}{A_\Omega}$$

$$T^* = \frac{2k\Gamma(n)}{(n-1)^{n-1}\exp(1-n)}$$

where $n$ is the shape parameter of the Nash IUH, $k$ is the scale parameter of the Nash IUH (h), $i = 1,\ldots,4$ and $a_i$, $b_i$, $c_i$, $d_i$ are coefficients of $H_i$ (listed in Table 1).

A sensitivity index is then used to estimate the sensitivity of the flood frequency distribution to variations in the estimated parameters for the infiltration and the IUH model. The proposed index is given by the equation (Raines & Valdés 1993):

$$\text{IS}\ (\%) = \frac{Q_0 - Q_{new}}{Q_0} 100 \qquad (5)$$

where IS is the sensitivity index (%), $Q_0$ is the 90% flood peak quantile for the initial set of parameters (m$^3$ s$^{-1}$) and $Q_{new}$ is the 90% flood-peak quantile for the new set of parameters (m$^3$ s$^{-1}$).

The experiment consists of introducing known variations to each parameter individually and estimating the proposed index without changing the other parameters. At this stage of the analysis we applied a 10% increase and the same percent reduction to the CN infiltration parameter value. The experiment also includes a 50% increase and a 50% reduction of the IUH parameter values, representing the kinematic-wave parameter $\alpha_\Omega$ for the geomorphoclimatic IUH used in the model of Raines & Valdés (1993), as well as in the scale and shape parameters $n$ and $k$ for the Nash IUH used in the new DFFD.

**Table 1** | Ht$_i$ coefficients, proposed new model

| *I* | $a_i$ | $b_i$ | $c_i$ | $d_i$ |
|---|---|---|---|---|
| 1 | 0.0000 | 0.1024 | 0.5000 | 1.0000 |
| 2 | 0.1024 | 0.2890 | 0.6529 | 1.1081 |
| 3 | 0.2890 | 0.5722 | 0.8048 | 1.3640 |
| 4 | 0.5722 | 1.0000 | 1.0000 | 3.1358 |

Finally we analyzed the effect of the Nash IUH on peak flow determination. Flows were estimated by means of different parameter estimation techniques by the method of moments and by the method developed by Rosso (1984):

$$n = 3.29\left(\frac{R_B}{R_A}\right)^{0.78} R_L^{0.07} \qquad (6)$$

$$k = 0.70\left(\frac{R_A}{R_B R_L}\right)^{0.48} v^{-1} L_\Omega \qquad (7)$$

where $R_B$ is basin bifurcation ratio, $R_A$ is basin area ratio, $R_L$ is basin length ratio, $v$ is expected peak velocity in the stream network (m s$^{-1}$) and $L_\Omega$ is the length of the highest-order stream (km).

Rosso's paper (Rosso 1984) is one of the different studies that propose to define theoretical relationships between hydrology and geomorphology and intend to find the average response of a watershed with particular geomorphologic characteristics. Rosso obtained an equation for parameter $n$ in the Nash IUH model by applying a numerical technique that follows an iterative scheme proposed by Croley (1977). This procedure was carried out for 126 combinations of the Horton (1945) relationships in the intervals $2.5 \leq R_B \leq 5.0$, $3.0 \leq R_A \leq 6.0$ and $1.5 \leq R_L \leq 4.1$, which are values usually found in nature. These results were finally processed by means of a multiple regression analysis in the logarithmic space to relate the dependent variable $n$ to the independent variables $R_A$, $R_B$ and $R_L$. This provides the equation proposed by the author which allows us to estimate the shape parameter of the Nash IUH from the Horton relationships in a watershed. The regression analysis performed produced excellent results.

This approach and the limits used in the regression analysis are related to the results presented in this paper. Both watersheds analyzed here have Horton relationships included in the above-mentioned intervals; we are therefore able to assume that it is possible to apply the Rosso method to estimate the Nash IUH parameters in the studied watersheds.

In order to analyze the goodness-of-fit between observed and estimated flows when the Nash model is used, we applied the linear regression model and estimated the confidence interval of the slope of the best-fit straight line to determine whether or not the slope was significantly different from 1. This technique is used when discharges are estimated using mean and median values of the parameters obtained with the method of moments in each particular event, and when the proposed technique of Rosso (1984) is applied.

## APPLICATION OF THE PROPOSED METHODOLOGY

The proposed methodology was applied to the Tapalqué and Azul river basins located in the centre of the province of Buenos Aires (Argentina). These basins cover areas of 1,560 and 1,102 $km^2$, respectively, with mean slopes between 1.2 and 5.7‰. Mean annual rainfall in the region was 884 mm during the 1994–2001 period. The general geographic location of the basins is shown in Figure 1. Schematic representations of the studied watersheds are shown in Figure 2(a) and (b), where the recording precipitation gauge located in the Tapalqué river basin and the flow measurement station in each basin are depicted.

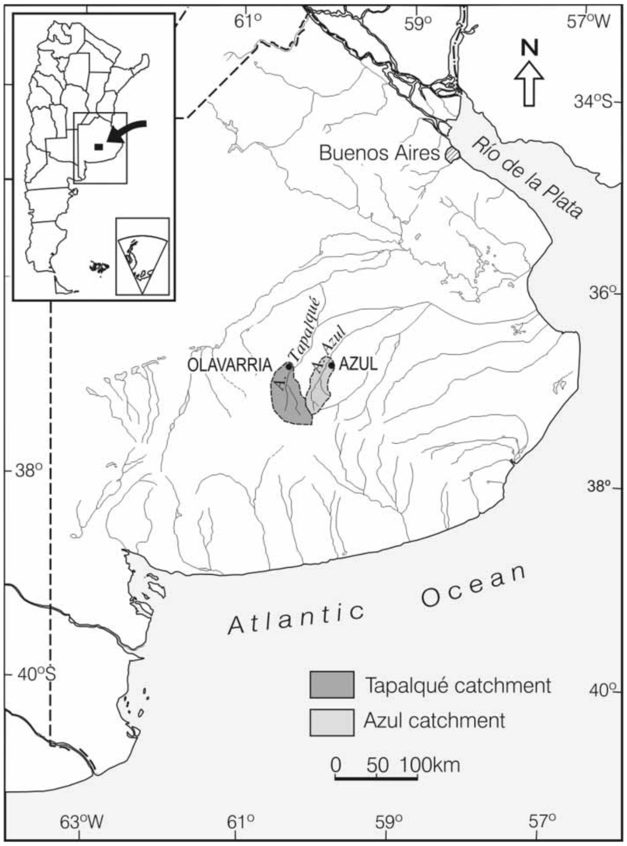


**Figure 1** | General geographic location of the basins.

### Geomorphologic parameters

Geomorphologic parameters were estimated at a scale of 1:100,000 with the Horton stream network classification. Bifurcation, length and area ratios are shown in Table 2.

### Analysis of annual peak flows

Annual peak flow data from two discharge measurement stations of the *Dirección de Hidráulica de la Provincia de Buenos Aires* at the outlet of the Tapalqué and Azul river basins were used. Mean daily stream flows in each basin are 2.86 and 1.69 $m^3 s^{-1}$, respectively. As these flows are much smaller with respect to peak flows from extreme events, base flow contribution to total flow is negligible. In the Tapalqué river watershed peak flow data are available for the period 1963–1997; in the Azul river watershed data are available for the period 1963–2001. The descriptive statistics of annual daily peak flows are presented in Table 3.

### Estimating rainfall model parameters

The parameters of the joint PDF of effective rainfall intensity and duration were estimated from available rainfall data of the National Meteorological Service at Olavarría Airport. The independent rainfall events were identified by applying the method proposed by Córdova & Bras (1981) that define a period of 12 hours without rainfall between independent events. In this analysis, rainfall events less than 2.5 mm are not considered. Table 4 presents the estimated means and standard deviations of rainfall intensity and duration in these basins and the parameter values of the rainfall model.

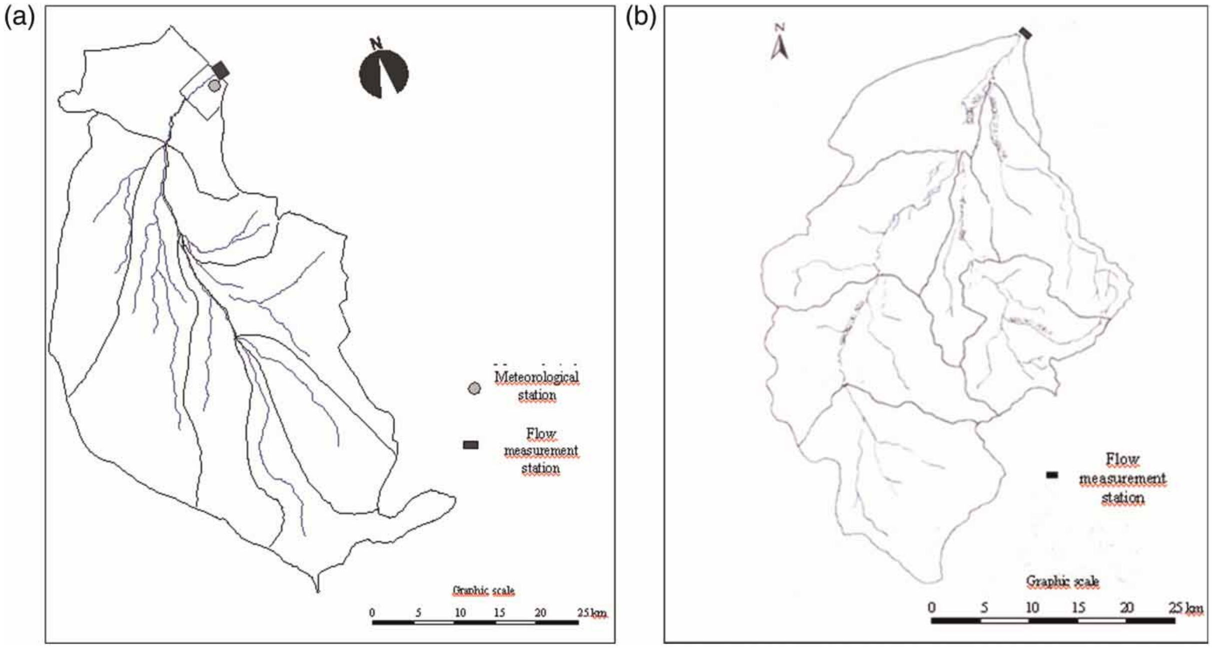


**Figure 2** | Schematic representation of the (a) Tapalqué and (b) Azul river basins.

**Table 2** | Horton's classification (where $\Omega$ is catchment order, $R_B$ is basin bifurcation ratio, $R_L$ is length ratio and $R_A$ is area ratio)

| Catchment | $\Omega$ | $R_B$ | $R_L$ | $R_A$ |
|---|---|---|---|---|
| Tapalqué | 4 | 3.065 | 1.671 | 3.529 |
| Azul | 4 | 3.975 | 1.727 | 5.421 |

**Table 3** | Descriptive statistics of peak annual flows

| River | Period | Mean ($m^3 s^{-1}$) | Standard deviation ($m^3 s^{-1}$) | Skew |
|---|---|---|---|---|
| Tapalqué | 1963–1997 | 75.6 | 101.7 | 2.2 |
| Azul | 1963–2001 | 60.8 | 95.6 | 2.4 |

## Estimating the Nash IUH parameters

The Nash model parameters were estimated using two different techniques. The first is the method of moments, which is used when effective rainfall hyetographs and observed flow hydrographs are available. The other is an indirect technique developed by Rosso (1984), which relates the Nash model to the geomorphologic IUH.

**Table 4** | Rainfall characteristics and rainfall model parameters (where $i$ is rainfall intensity and $t$ is rainfall duration)

| | $i$ ($cm\ h^{-1}$) | $t$ (h) | Parameters $\beta$ ($h\ cm^{-1}$) | $\lambda$ ($h^{-1}$) |
|---|---|---|---|---|
| Mean | 0.99 | 3.92 | 1.01 | 0.25 |
| Standard deviation | 1.41 | 3.72 | | |

Note that Rosso's methodology is effective when there are little or no observed rainfall or flow data; this is because the expressions for estimating the Nash IUH parameters depend on the catchment morphological features and on the average flow velocity in the drainage system.

## MAIN RESULTS

### Application of the new DFFD approach

The new DFFD approach was applied to the above-mentioned basins and results are presented in Figures 3 and 4. The figures show the empirical CDF, estimated by

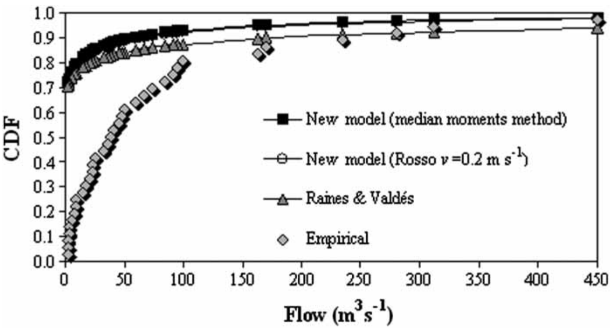


**Figure 3** | CDFs, Tapalqué river.

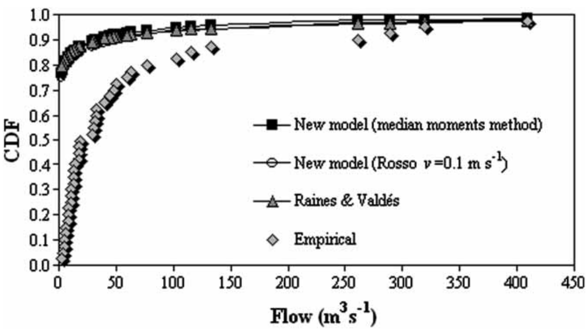


**Figure 4** | CDFs, Azul river.

Equation (8), and the CDFs obtained with the proposed new model and with the derived distribution approach proposed by Raines & Valdés (1993), which uses the geomorphoclimatic IUH as the catchment's response model:

$$F_x(X) = 1 - \frac{m}{n+1} \tag{8}$$

where $X$ is observed peak flow, $F_x(X)$ is the CDF of peak flows, $m$ is the number of order (being 1 for the largest value in the sample and $n$ for the smallest value) and $n$ is sample size.

Kolmogorov–Smirnov distances were estimated in order to quantify the goodness-of-fit obtained with each model (results are listed in Table 5):

$$D_n = \max_{j=1}^{n}\left[\left|\frac{j}{n} - F_x(X_j)\right|\right] \tag{9}$$

where $D_n$ is Kolmogorov–Smirnov distance, $X_j$ is observed peak flow, $j$ is the number of order and $n$ is sample size.

Results show that, in these basins, the IUH model does not significantly affect estimates made with the DFFD approach. With floods greater than 250 $m^3\,s^{-1}$, the proposed new model provides a slightly better fit to empirical frequencies than the model developed by Raines and Valdés. This result is more evident in the Tapalqué river catchment; the Kolmogorov–Smirnov distances confirm numerically what the figures show. For low flows on the other hand, Raines and Valdés's model shows a better fit to empirical frequencies even though both models are not able to capture the whole shape of the empirical CDFs.

## Sensitivity of DFFDs to infiltration model parameters

The estimation of the flood frequency curves with the two versions of the DFFD approach, using CN values with a 10% increase and a 10% reduction with respect to the initial parameter values, was performed for both basins. An example of this experiment, including the flood frequency curves for the Tapalqué river basin, is depicted in Figures 5 and 6 (see also Table 6).

At this stage, changes in the SCS CN parameter were amplified in the estimates of floods associated with different non-exceedance probabilities. For the 90% flood-peak quantiles used to define the proposed sensitivity index, changes of up to 160% and down to 60% were detected with respect to floods estimated with the above-mentioned initial parameter values.

Results also show, both numerically and graphically, a greater sensitivity to this parameter for the Raines & Valdés (1993) model in both basins under study.

## Sensitivity of DFFDs to the IUH parameters

We analyzed the sensitivity of flood frequency curves estimated with the new DFFD to changes in the Nash IUH parameters $n$ and $k$. Results obtained for the Tapalqué river basin are shown in Figure 7; sensitivity index values for both basins under consideration are listed in Table 7.

From an analysis of the above-mentioned results, it may be concluded that changes in the Nash IUH parameters exert a similar effect on both basins. An aspect to be

**Table 5** | Kolmogorov–Smirnov distances

| Model | Parameter estimation method | $D_n$ Tapalqué catchment $Q_{max} > 20$ m$^3$ s$^{-1}$ | $Q_{max} > 250$ m$^3$ s$^{-1}$ | Azul catchment $Q_{max} > 20$ m$^3$ s$^{-1}$ | $Q_{max} > 250$ m$^3$ s$^{-1}$ |
|---|---|---|---|---|---|
| Raines & Valdés | | 0.466 | 0.057 | 0.297 | 0.060 |
| Proposed new model | Median moments | 0.495 | 0.028 | 0.358 | 0.030 |
| | Rosso technique | 0.487 | 0.024 | 0.347 | 0.026 |

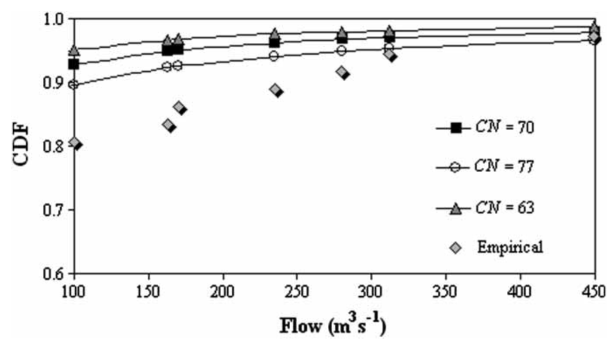


**Figure 5** | Sensitivity of the new model to CN, Tapalqué river.

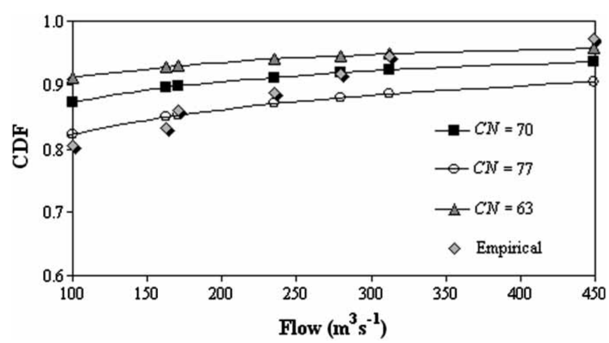


**Figure 6** | Sensitivity of Raines and Valdés model to CN, Tapalqué river.

**Table 6** | Sensitivity index to infiltration parameter CN

| Catchment | DFFD model | Changes in CN (%) | SI (%) |
|---|---|---|---|
| Tapalqué | New model | +10 | −68.9 |
| | | −10 | 45.0 |
| | Raines & Valdés | +10 | −163.8 |
| | | −10 | 61.1 |
| Azul | New model | +10 | −81.3 |
| | | −10 | 46.9 |
| | Raines & Valdés | +10 | −134.4 |
| | | −10 | 60.6 |

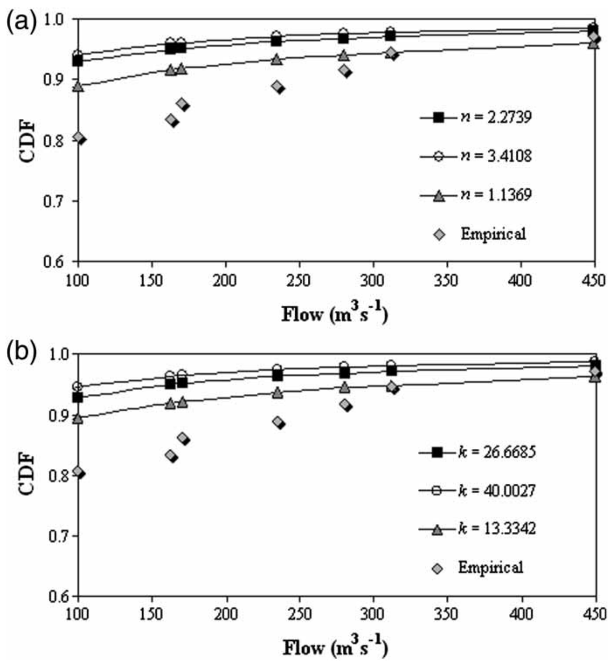


**Figure 7** | Sensitivity of the new model to Nash IUH parameters, Tapalqué river: (a) $n$ and (b) $k$.

**Table 7** | Sensitivity index to Nash IUH parameters in the proposed new model

| Catchment | Parameter | Changes in the parameters (%) | SI (%) |
|---|---|---|---|
| Tapalqué | $n$ | +50 | 23.1 |
| | | −50 | −68.9 |
| | $k$ | +50 | 27.9 |
| | | −50 | −99.6 |
| Azul | $n$ | +50 | 11.7 |
| | | −50 | −94.2 |
| | $k$ | +50 | 11.7 |
| | | −50 | −94.2 |

emphasized is the fact that higher absolute values of the sensitivity index correspond to lower parameter values.

We experimented with a 50% increase and with the same percent reduction with respect to the initial values of the $n$ and $k$ parameters in both basins because, unlike the SCS CN parameter, smaller variations exerted very little effect on flood frequency curves.

An experiment similar to that described above was carried out for the kinematic-wave parameter of the Geomorphoclimatic Instantaneous Unit Hydrograph (GcIUH), which is the catchment response model in the flood frequency distribution of Raines & Valdés. The results obtained for the Tapalqué river basin are shown graphically in Figure 8; the sensitivity index values for both basins under study are quantified and listed in Table 8.

The above shows that sensitivity index values are similar for both basins. Note also that positive variations of the kinematic-wave parameter yielded negative values of the sensitivity index.

## Analysis of the effect of applying the Nash IUH to estimate peak flows

Peak flows in the above basins were estimated using the Nash IUH. The IUH parameters were estimated using the method of moments using available rainfall and flow data of 14 events in the Tapalqué river basin and 13 events in the Azul river basin. These events were the most rigorous observed in both basins in the last 25 years. The rainfall

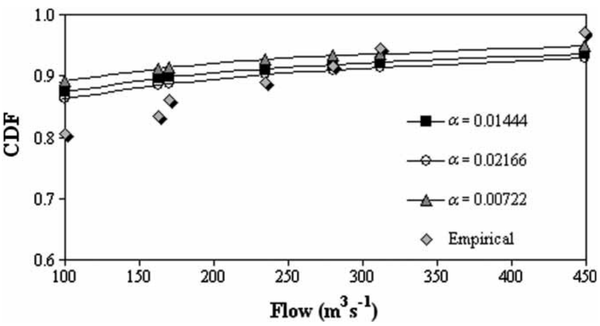


**Figure 8** | Sensitivity of Raines and Valdés model to the GcIUH parameter, Tapalqué river.

**Table 8** | Sensitivity index to GcIUH parameter in the Raines & Valdés model

| Catchment | Parameter | Changes in the parameter (%) | SI (%) |
|---|---|---|---|
| Tapalqué | $\alpha_\Omega$ | +50 | −37.8 |
| | | −50 | 41.3 |
| Azul | $\alpha_\Omega$ | +50 | −34.6 |
| | | −50 | 34.4 |

data for these events were available at a 6-h scale from nine stations in the Tapalqué river basin and from eight stations in the Azul river basin; we were therefore able to estimate the mean areal rainfall depth and the mean rainfall duration and intensity. For the 14 rainfall events in the Tapalqué river basin, the mean values of rainfall depth, duration and intensity are 7.68 cm, 19.3 h and 0.5 cm $h^{-1}$ and for the 13 rainfall events in the Azul river basin these values are 6.29 cm, 21.2 h and 0.4 cm $h^{-1}$. These results show that the mean values of the rainfall variables for severe storm are similar in both basins.

Given the geomorphological characteristics of these basins, selected mean flow velocities in each basin were used when Rosso's technique was applied. The mean and median values of the parameters estimated by the method of moments as well as the parameters estimated by Rosso's technique are presented in Table 9. Observed and estimated hydrographs for two flood events in each studied watershed are shown in Figures 9 and 10. These figures show that, for these events, the linear model is capable of representing not only the catchment response during peak flows but also the shape of the hydrograph when the parameters are estimated by moments method. From these figures it is also worth noting that Rosso's technique is a good choice when there are no observed rainfall and flow data in these two watersheds with similar meteorological and geomorphological characteristics in Argentina.

Figures 11 and 12 depict the scatter plots of estimated over observed peak flows when flows are calculated from (1) median parameter values of the Nash model estimated from observed rainfall–runoff events by the method of moments and (2) the parameters estimated by applying Rosso's technique.

A straight line of best fit was drawn in these figures in order to study the correlation between the observed and

**Table 9** | Nash IUH parameters

| Catchment | Nash IUH parameter | Method of moments Mean | Method of moments Median | Rosso technique |
|---|---|---|---|---|
| Tapalqué | $n$ | 2.30 | 2.27 | 3.06 |
| | $k$ (h) | 28.92 | 26.67 | 18.13 |
| Azul | $n$ | 2.90 | 2.49 | 2.64 |
| | $k$ (h) | 20.30 | 17.49 | 13.67 |

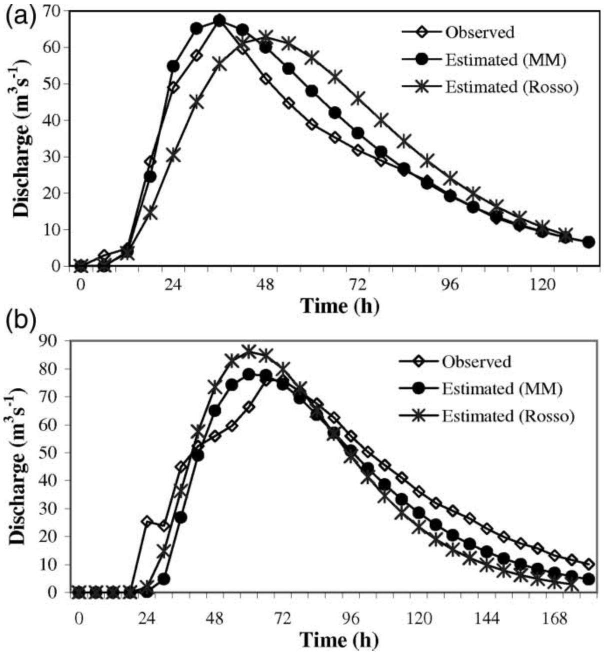


**Figure 9** | Observed and estimated hydrographs, Tapalqué river, event (a) 2 June 1991 and (b) 3 March 1987.

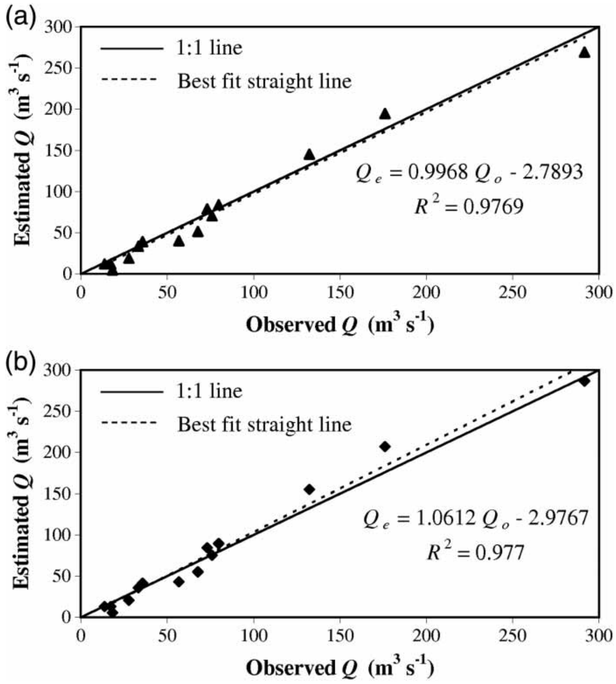


**Figure 11** | Scatter plot of estimated over observed peak flows, Tapalqué river, flows estimated using (a) the median value of parameters obtained by the method of moments and (b) parameters obtained from Rosso technique.

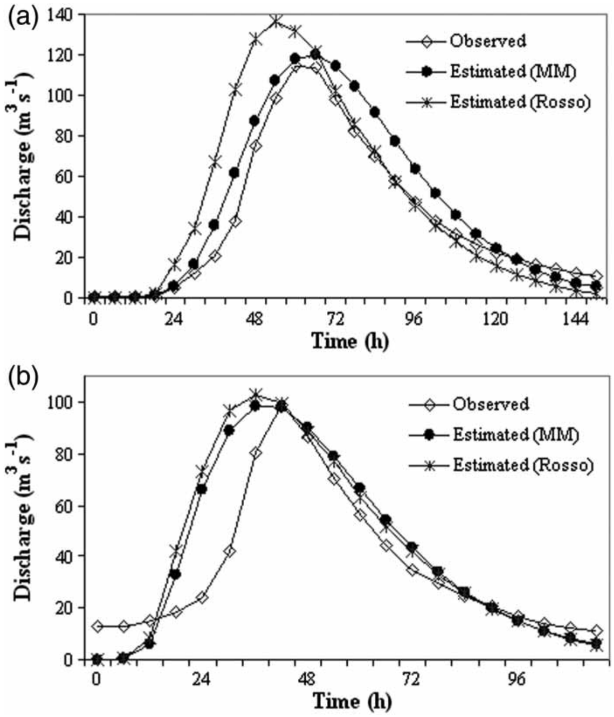


**Figure 10** | Observed and estimated hydrographs, Azul river, event (a) 26 September 1998 and (b) 8 October 2000.

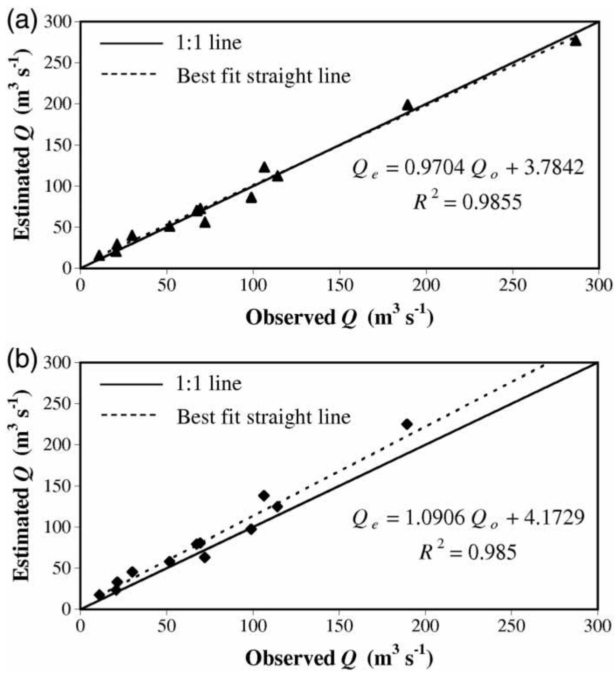


**Figure 12** | Scatter plot of estimated over observed peak flows, Azul river, flows estimated using (a) the median value of parameters obtained by the method of moments and (b) parameters obtained from Rosso technique.

**Table 10** | Confidence intervals of the slopes of best-fit straight lines and correlation coefficients

| Catchment | Median method of moments Lower confidence limit | Slope | Upper confidence limit | R | Rosso technique Lower confidence limit | Slope | Upper confidence limit | R |
|---|---|---|---|---|---|---|---|---|
| Tapalqué | 0.900 | 0.997 | 1.093 | 0.988 | 0.959 | 1.061 | 1.164 | 0.988 |
| Azul | 0.892 | 0.970 | 1.048 | 0.992 | 0.995 | 1.090 | 1.186 | 0.992 |

estimated flows. The equations for the correlation determined by linear regression are also presented; it is shown that the correlation coefficients between observed and estimated flows in these basins range between 0.988 and 0.992 (Table 10).

An identity line (i.e. a 1:1 line) was drawn as a reference in the scatter plots to see how observed and estimated flows agree with each other and we tested the statistical significance of the slopes of the regression lines from the 1:1 line. The tests performed show that the slope of the best-fit straight line is not significantly different from 1, at a 95% confidence level when flows are calculated from median parameter values of the Nash model estimated by the method of moments; a similar result is obtained when flows are estimated from parameter values obtained with Rosso's technique. The 95% confidence intervals of the slope of the best-fit straight lines are listed in Table 10.

## DISCUSSION AND CONCLUSIONS

Application of the new flood frequency distribution approach to the Tapalqué and Azul river basins shows that the use of the Nash IUH model in the new DFFD defines a goodness-of-fit with the empirical frequency curve similar to that obtained with a more complex catchment response model, such as the geomorphoclimatic IUH (e.g. Raines & Valdés 1993), even though both models are not able to represent the whole shape of the empirical CDFs. The significance of the new approach is that the Nash IUH model is widely used; it is important to note that application of the new proposed model improves flood estimates for high non-exceedance probabilities, which are required for engineering applications in the estimation of design parameters of small hydraulic structures. Both new and Raines & Valdés's DFFDs described the infiltration process by the SCS CN method, which provides a zero probability that is extremely high in the basins considered in this study. This could be the reason why the empirical and theoretical curves do not coincide in the first part of the CDF.

Results presented in this paper show that, for peak flood flows greater than 250 $m^3 s^{-1}$ associated with greater non-exceedance probabilities, Kolmogorov–Smirnov distances between the empirical flood frequency curve and that estimated with the new proposed model (which uses Nash IUH) are about 50% lower than those estimated from the Raines & Valdés model (which uses GcUIH).

In addition, IUH model parameters for the new DFFD can be estimated with observed rainfall and flow data. When such data are lacking, Rosso's technique is a suitable approach for estimating DFFDs. This is important in developing countries that have ungauged basins or more rainfall than flow records.

An innovative result was obtained from a numerical experiment conducted using observed precipitation and flow data and Nash IUH parameters estimated from the Rosso method. This result shows numerical information about the agreement in peak flood flow estimations attained when using the Nash IUH in two basins of low slope and temperate climate. Previous studies had not explicitly taken into account the evaluation of the adjustment of IUH and had focused on the evaluation of the complete derived flood frequency model.

The new proposed flood frequency curve is in agreement with the empirical curve similar to that presented in the international literature. Among the papers reviewed, Rahman *et al.* (2002) developed a derived flood frequency curve and applied the proposed methodology in three small Australian watersheds. Arónica & Candela (2007) have also presented flood frequency curves obtained by simulation; the developed procedure is applied to six Sicilian watersheds whose areas are between 20 and 800 $km^2$. Results presented in this paper show discrepancies between the derived and the empirical flood frequency curves, similar to that observed in the previously mentioned papers for high flows associated with low probability values. The analytical approach developed here does not result in such a good performance for low flows as that of numerical approaches proposed in the international literature. To overcome this problem, the flood probability distribution could be derived numerically.

Note that both basins where the proposed methodology was applied are in low slope regions and have areas greater than 1,100 $km^2$ while the watersheds analyzed in the international literature reviewed are smaller. One of the basins analyzed by Arónica & Candela (2007) is of a similar size to those considered in this study. These authors report a minor agreement for the largest catchments used in the study. This fact emphasizes the importance of the results in establishing a range of watershed sizes where this research helps to define the application possibilities of the method since the uncertainty of model parameter estimations increases with watershed area.

Note that the studied watershed sizes are within the limits of application of the unit hydrograph theory. Furthermore, as the slopes are low, there is no influence of the orography on the precipitation and we assume that there is no great spatial variation of rainfall. Note that the most important rainfall events which produced the greatest floods were registered in both catchments. In particular, we mention the floods of April 1980 and November 1985 which caused important damage to the rural sector of both catchments and also to the cities of Olavarría and Azul. These events were not analyzed in this paper because rainfall and flow data were not available for them. From the analysis of the severe storm events included in this study, it can be said that the mean values of the rainfall variables (depth, duration and intensity) are similar in the two catchments.

In this study we have not taken into account the variability of antecedent moisture conditions and the rainfall–runoff transform model used is not a distributed model. Including such aspects would refine the representation of runoff generation processes. Another matter to be considered is that we have not included the storage effects in the modelling; these can be significant due to the low slopes of both watersheds.

From the different experiments conducted to quantify the sensitivity of flood frequency curves to variations in infiltration and IUH parameters, the following conclusions were drawn.

Variations in the SCS CN parameter, used by both models of the DFFD approach, have the greatest effect on estimated floods associated with different probabilities and therefore yield the highest sensitivity index values. Note also that the Raines and Valdés model is more sensitive to this parameter in both basins. This result suggests that the use of the Nash IUH as the watershed response model produces more consistent flood frequency curves since precipitation and infiltration models are common to both DFFDs.

The effect of variations in the shape and scale parameters of the Nash IUH on flood estimates is similar for both parameters and in both basins under study. The influence of the kinematic-wave parameter on the Raines and Valdés flood frequency curves is similar for both the Tapalqué and Azul river basins.

The presented results suggest some topics for further research, such as taking into account the fact that the infiltration process description is very important in the representation of rainfall–runoff transform process. A possible area of future work is to analyze the uncertainty associated with flood flow estimation by studying the changes in the shape of the new DFFD when applying different infiltration models.

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## REFERENCES

Arónica, G. T. & Candela, A. 2007 Derivation of flood frequency curves in poorly gauged Mediterranean catchments using a simple stochastic hydrological rainfall–runoff model. *J. Hydrol.* **347**, 132–142.

Córdova, J. R. & Bras, R. L. 1981 Physically based probabilistic models of infiltration, soil moisture and actual evapotranspiration. *Water Resour. Res.* **17** (1), 93–106.

Croley, T. E. 1977 *Hydrologic and Hydraulic Computation on Small Programmable Calculators*. Report, Iowa Institute of Hydraulic Research, University of Iowa, Iowa City.

De Michele, C. & Salvadori, G. 2002 On the derived flood frequency distribution: analytical formulation and the influence of antecedent moisture condition. *J. Hydrol.* **262**, 245–258.

Díaz-Granados, M. A., Valdés, J. B. & Bras, R. L. 1984 A physically-based flood frequency distribution. *Water Resour. Res.* **20** (7), 995–1002.

Eagleson, P. S. 1972 Dynamics of flood frequency. *Water Resour. Res.* **8** (4), 878–898.

Fiorentino, M., Manfreda, S. & Iacobellis, V. 2007 Peak runoff contributing area as hydrological signature of the probability distribution of floods. *Adv. Water Resour.* **30**, 2123–2134.

Gioia, A., Iacobellis, V., Manfreda, S. & Fiorentino, M. 2008 Runoff thresholds in derived flood frequency distributions. *Hydrol. Earth Syst. Sci.* **12**, 1295–1307.

Goel, N. K., Kurothe, R. S., Mathur, B. S. & Vogel, R. M. 2000 A derived flood frequency distribution for correlated rainfall intensity and duration. *J. Hydrol.* **228**, 56–67.

Gottschalk, L. & Weingartner, R. 1998 Distribution of peak flow derived from a distribution of rainfall volume and runoff coefficient, and a unit hydrograph. *J. Hydrol.* **208**, 148–162.

Hebson, C. & Wood, E. F. 1982 A derived flood frequency distribution using Horton order ratios. *Water Resour. Res.* **18** (5), 1509–1518.

Henderson, F. M. 1963 Some properties of the unit hydrograph. *J. Geophys. Res.* **68** (16), 4785–4793.

Horton, R. E. 1945 Erosional development of stream and their drainage basin: hydrophysical approach to quantitative morphology. *Geol. Soc. Am. Bull.* **56**, 275–370.

Iacobellis, V. & Fiorentino, M. 2000 Derived distribution of floods based on the concept of partial area coverage with a climatic appeal. *Water Resour. Res.* **36** (2), 469–482.

Kurothe, R. S., Goel, N. K. & Mathur, B. S. 2001 Derivation of a curve number and kinematic-wave based flood frequency distribution. *Hydrol. Sci. J.* **46** (4), 571–584.

Loukas, A. 2002 Flood frequency estimation by a derived distribution procedure. *J. Hydrol.* **255**, 69–89.

Nash, J. E. 1960 A unit hydrograph study with particular reference to British catchments. *Proc. Inst. Civil. Eng.* **17**, 249–282.

Rahman, A., Weinmann, P. E., Hoang, T. M. T. & Laurenson, E. M. 2002 Monte Carlo simulation of flood frequency curves from rainfall. *J. Hydrol.* **256**, 196–210.

Raines, T. H. & Valdés, J. B. 1993 Estimation of flood frequencies for ungauged catchments. *J. Hydraul. Engng.* **119** (10), 1138–1154.

Rodríguez Iturbe, I. & Valdés, J. B. 1979 The geomorphologic structure of hydrologic response. *Water Resour. Res.* **15** (6), 1409–1420.

Rodríguez Iturbe, I., González-Sanabria, M. & Bras, R. L. 1982 A geomorphoclimatic theory of the instantaneous unit hydrograph. *Water Resour. Res.* **18** (4), 877–886.

Rosso, R. 1984 Nash model relation to order ratios. *Water Resour. Res.* **20** (7), 914–920.

Seoane, R. S. & Valdés, J. B. 1993 A derived flood frequency distribution for ungauged catchments. In *Proceedings of the Annual Conference of the Water Resources Planning and Management Division of the American Society of Civil Engineering*, Seattle, US, pp. 384–387.

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