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Design flood hydrographs from the relationship between flood peak and volume

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Abstract

Hydrological frequency analyses are usually focused on flood peaks. Flood volumes and durations have not been so exhaustively studied although there are many practical cases, like dam design, where the full hydrograph is of interest. A flood hydrograph may be described by a multivariate function of peak, volume and duration. Most standard bivariate and trivariate functions do not produce univariate three-parameter functions as marginal distributions, but three-parameter functions are required to fit highly skewed data as flood peak and volume series. In this paper, relationship between flood peak and hydrograph volume is analysed to overcome this problem. A Monte Carlo experiment was carried out to generate an ensemble of hydrographs that keep the statistical properties of marginal distributions of peaks, volumes and durations. This ensemble can be applied to determine the Design Flood Hydrograph (DFH) for a reservoir, which is not a unique hydrograph, but a curve in the peak-volume space. All hydrographs in that curve have the same return period, understood as the inverse of the probability to exceed a certain water level in the reservoir any given year. The procedure can also be applied to design the length of the spillway crest in terms of risk to exceed a given water level in the reservoir.

1 Introduction

Hydrological frequency analyses are usually focused on flood peaks, as culverts, bridges and river channel defences are designed with the peak flow for a given return period. There are lots of studies to estimate the flood peak frequency curve (Cunnane, 1988, 1989; GREHYS, 1996), but flood volumes have not been so exhaustively studied, despite the fact that they are needed to design some structures like dams, where the entire flood hydrograph is of interest.

A flood event may be described by a multivariate function of peak, volume and duration, as a joint distribution of their marginal distributions. Some attempts have been carried in this direction. Goel et al. (1998) employed the bivariate normal distribution of peak and volume, after a normalization of data series by two Box-Cox transformations, in order to lower the skewness coefficient to a value nearly zero and to correct the coefficient of kurtosis to a value nearly three. Other studies were based on the bivariate normal distribution (Krstanovic and Singh, 1987; Sackl and Bergman, 1987), but as flood peaks and volumes are highly skewed, prior transformations in data series are required. In the case that statistical behaviours of peak and volume data are represented by Gumbel distributions, a bivariate extreme value distribution can be used (Yue et al., 1999). A bivariate lognormal distribution was developed by Yue (2001). All these attempts assume that flood variables can be represented by the same distribution. To relax the restriction of a unique distribution function to represent peak and volume, bivariate and trivariate distributions have been derived using the Copula method. Different Copula families have been used: Favre et al. (2004) considered the Farlie-Gumbel-Morgenestern, Frank and Clayton families and no significant differences were shown among them; De Michele et al. (2005) considered an Archimedean Gumbel's 2-Copulas, simulating the dependence between peak and flood volume by the Kendall's τ rank correlation coefficient. Grimaldi and Serenaldi (2006) developed an asymmetric Archimedean Copula more flexible than symmetric Copulas. Zhang and Singh (2007) utilized the Gumbel-Hougaard Copula to simulate the trivariate distribution of peak, volume and duration.

The Design Flood Hydrograph (DFH) is the hydrograph adopted according to design standards to ensure safety of a structure (Xiao et al., 2009). Design standards for dams are based on the Probable Maximum Flood (PMF) or on a given return period. Some attempts have been carried out to estimate the return period of a hydrograph, as the inverse of its probability of occurrence, by the joint probability of a bivariate distribution. This joint probability is not explicit when the variables are correlated and the conditional return period, given a maximum value of the other variable, must be calculated (Zhang and Singh, 2006). The joint return period has a lower probability of occurrence than the inclusive probability of both events, named as primary return period, and a higher probability of occurrence than the exclusive probability of both events, named as secondary return period. This means that a structure could be under-dimensioned if it is designed with the primary return period and over-dimensioned if it is designed with the secondary return period (Salvadori and De Michele, 2004).

But the return period is the average time elapsed between two successive events exceeding a given threshold (Ponce, 1989), which must be defined in terms of acceptable risk to the structure. Hydrological risk at a bridge or a culvert is related to the maximum water level in the reach, which mainly depends on peak discharge. Therefore, the threshold can be defined as a given discharge. But hydrological risk at the dam is related to the maximum reservoir level and maximum released flow during the event, which do not only depend on the maximum inflow discharge, as there can be several floods with different combinations of volume and peak that yield the same level and release. At first, a greater peak will be worse for dams with smaller reservoir areas, and a greater volume will be worse for dams with larger reservoir areas, but the crest length of the spillway must be considered and could modify this statement. Therefore, peak and volume are each more influential in the risk depending on the reservoir area, the crest length of the spillway and whether the spillway is controlled or uncontrolled. The problem is complex and a set of hydrographs can have the same design return period. In addition, a pair of peak and volume values will have a different return period than that of their marginal distributions. Therefore, peaks and volumes cannot be utilized independently as thresholds to assess dam risk. The threshold must be defined as a given water level in the reservoir, so that the return period is the inverse of the probability to exceed that reservoir water elevation any given year.

In this paper, a methodology is presented to obtain Flood Design Hydrographs for dam design in Spanish basins. Peaks and volumes in most Spanish basins are highly skewed and are best described by the Generalized Extreme Value distribution (GEV). As a suitable bivariate distribution from three-parameter distributions has not been

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developed yet, the relationship between peak discharge and hydrograph volume has been analyzed from recorded data, in order to generate a large set of annual maxima synthetic hydrographs that keep the marginal distributions of peaks, volumes and durations. Each hydrograph is routed through the dam to compute the maximum water level in the reservoir. As the return period assigned to a flood is the inverse of the probability to exceed a water level, it is calculated as the total number of hydrographs divided by the number of hydrographs that reached a maximum water level higher than the threshold. With this procedure, the DFH for a given return period is not a unique hydrograph, but a curve in the peak-volume domain, so that there will be a set of hydrographs with the same return period and the same risk to the dam.

2 Case studies

Santillana, Entrepeñas and Buendia reservoirs were selected as case studies. The three reservoirs are located on the Tagus basin, Middle West Spain, and belong to the 32nd homogeneous region (Fig. 1). There are not recorded data of inflow discharges to the reservoirs, but they can be estimated from recorded mean daily water levels and releases at the 93 033, 93 001 and 93 087 reservoir stations.

The Santillana reservoir is located in the Manzanares river, near the city of Madrid. The dam is an earthfill embankment with a height of 40 m and crest length of 1355 m. Flood flows over the spillway are controlled by a 5.25 m by 12 m gate. Entrepeñas reservoir is located on the Tagus river. The dam has a concrete cross section with a height of 87.35 m and length of 383 m. Flood flows over the spillway are controlled by five 10.76 m by 5.50 m gates. Buendia reservoir is located on the Guadiela river. The concrete dam has a height of 78.73 m and length of 315 m. Flood flows are controlled by five 12.20 m by 1.50 m gates. Further details of their main characteristics are included in Table 1.

3 Marginal distributions

The marginal distributions of Annual Maximum Discharges (AMD) and Annual Maximum Volumes (AMV) were estimated from recorded data. Identification of the AMV in a year is the main problem of the marginal distribution of maximum volumes. An AMV could be obtained from a long hydrograph with a low peak discharge, but it would not imply a high risk for the dam. As the study starts from the AMD frequency curve, volumes linked to these peaks should be identified, so that the methodology is consistent.

3.1 Flood peak frequency distribution

A regional study was carried out in Spain, in order to improve local estimations of flood frequency curves, and continental Spain was divided in 30 homogeneous regions. Spanish geography shows a high climatic variability, so that regions were identified by means of geographical characteristics. Index-flood is the most common regional method (Bocchiola et al., 2003; Kjeldsen and Jones, 2007; Noto and La Loggia, 2009) and it supplies regional values of L-coefficient of skewness (L-CS) and L-coefficient of variation (L-CV) in a homogeneous region. There is an agreement about regionalization of L-CS, as its estimation uncertainty from local data is high, even for long record lengths. But regionalization of L-CV is widely discussed. Firstly, its estimation uncertainty is lower than that of L-CS and very similar to that of the mean, which cannot be regionalized. In addition, the relationship between CV and basin area seems to be very complex as it depends on the interaction between different runoff processes and it has been seen that it increases with basin area, until a threshold, and then decreases with basin area (Blöschl and Sivalapan, 1997; Iacobellis et al., 2002). As this L-CV pattern has been seen in Spanish regions, a regional shape estimation procedure was selected to relax the restriction of a regional value of L-CV. Comparison between the two methods showed that the regional shape estimation improves the estimation of quantiles in the upper tail of the frequency distribution, as it is the case of this paper (Hosking and Wallis, 1997, p. 150).

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The three reservoirs belong to the 32nd region, which has a regional L-CS value equal to 0.253. Mean daily discharges at reservoir stations were transformed into maximum instantaneous discharges by the Fuller's formula (Fill and Steiner, 2003). A GEV distribution (Eq. 1) was fitted to the AMD series with the regional value of L-CS (Table 2).

$$F(x) = \exp\left\{-\left[1-k\left(\frac{x-u}{\alpha}\right)\right]^{1/k}\right\} \quad (1)$$

where, u is the location parameter, α is the scale parameter and k is the shape parameter.

3.2 Flood volume frequency distribution

Regionalization results of AMD were extended to AMV data series. Volumes of the hydrographs linked to the AMD were identified. The start and the end of the hydrograph were assumed to be the start and the end of the surface runoff. The start was identified as an abrupt rise of the discharge higher than 20%. The end was identified as the point from which the receding limb is described by an exponential function (Eq. 2). The α coefficient was assumed to be equal to 0.0063 h^{-1} in the 32nd region. The dependence between two successive peaks was identified by the independence criteria proposed by Cunnane (1979).

$$Q = Q_0 \cdot e^{-\alpha t} \quad (2)$$

Homogeneity of AMV was tested at the homogeneous regions previously identified by heterogeneity measures based on L-Moments (Eqs. 3 and 4) (Hosking and Wallis, 1993). Homogeneity of AMV series was met as can be seen in Table 3. Volume frequency curves were fitted with a GEV distribution and a regional shape parameter

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(Table 2).

$$V_i = \left(\frac{\sum_{j=1}^{N} n_j \cdot \left(t_{i,j} - t_i^{\mathrm{R}} \right)^2}{\sum_{j=1}^{N} n_j} \right)^{1/2} \tag{3}$$

$$H_i = \frac{V_i - \mu(V_i)}{\sigma(V_i)} \tag{4}$$

where, V_i is the weighted standard deviation of the at site sample L-Moment ratio of i-th order, N is the number of stations in the region, n_j is the length of the sample at site j, $t_{i,j}$ is the L-Moment ratio of i-th order at site j, t_i^{R} is the regional value of the L-Moment ratio of i-th order, H_i is the i-th heterogeneity measure and $\mu(V_i)$ and $\sigma(V_i)$ are the mean and standard deviation of the simulated values of V_i, on a large number of simulated regions with N sites, having each site the same record length as their real-world counterparts.

4 Relationship between peak flow and hydrograph volume

Dependence of the volume on the peak discharge was analyzed in order to estimate the joint distribution. A linear relationship in the log-log space was found, both in each station and at regional scale in a homogeneous region, and was represented by regression equations (Fig. 2).

At local scale, the volume for a given maximum discharge was estimated by fitting a regression equation over the observed pairs (Eq. 5).

$$V_{i,j} = 10^{a_j} \cdot Q_{i,j}^{b_j} \tag{5}$$

where, $Q_{i,j}$ and $V_{i,j}$ are the maximum observed peak and volume in the year i at station j; a_j and b_j are the coefficients of the local regression equation at station j.

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Then, the relationship was analyzed in the regional log-log space of the real values of peaks and volumes, but a problem of scale was found, because the regression equation can not distinguish greater volumes of larger basins from smaller volumes of smaller basins. It can be seen that there are not Q-V pairs of Entrepeñas and Buendía reservoirs below a peak of 1.5, while there are not Q-V pairs of Santillana reservoir above a peak of 2 (Fig. 2a). Therefore, a standardization of peaks and volumes was carried out to overcome the scale problem, dividing the peaks and volumes by their means in each station (Eqs. 6 and 7) (Fig. 2b).

$$q_{i,j} = \frac{Q_{i,j}}{\overline{Q_j}} \tag{6}$$

$$v_{i,j} = \frac{V_{i,j}}{\overline{V_j}} \tag{7}$$

where $q_{i,j}$ and $v_{i,j}$ are the standardized maximum peak discharge and volume in the year i at station j and $\overline{Q_j}$ and $\overline{V_j}$ are the mean values of maximum peak discharges and volumes over all the record length in the station j.

At regional scale, a hydrograph volume (V) is estimated from its hydrograph peak (Q) destandardizing the regression equation (Eq. 8).

$$V_{i,j} = \left(10^{a_\mathrm{r}} \cdot q_{i,j}^{b_\mathrm{r}}\right) \cdot \overline{V_j} \tag{8}$$

where a_r and b_r are the coefficients of the regional regression equation.

Regression equations were fitted at each case study and in the 32nd region, as shown in Table 4. The variability of the relationship between peaks and volumes, or estimation uncertainty of the regression equation, was estimated by the residual variance

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(σ_{reg}) (Eq. 9).

$$\sigma_{\text{reg}} = \sqrt{\frac{\sum_{i=1}^{N} \left(\log_{10}(V_i) - \log_{10}(V_i')\right)}{N-p-1}} \tag{9}$$

where, N is the number of Q-V pairs to estimate the regression equation, V_i is the i-th observed volume, V_i' is the i-th estimated volume by the regression equation, and p is the number of variables of the equation, equal to one in this case.

5 Generation of synthetic peak-volume pairs

The return period of a hydrograph is calculated as the inverse of the probability of exceedance of the maximum water level in the reservoir that was reached while routing that hydrograph. As the probability of exceedance for high return periods is very low, a large number of hydrographs is required to accurately estimate these return periods, for which dams are designed. Therefore, synthetic hydrographs must be generated to extend the observed data.

A large set of synthetic hydrographs, keeping the statistical characteristics of observed peaks, volumes and durations, was generated. The synthetic generation consists of three steps: the first is the generation of a set of synthetic peak flows, the second is the generation of a synthetic volume for each synthetic peak, comparing the local and regional relationships between peaks and volumes, and the third is the generation of a hydrograph shape for each synthetic pair of peak and volume, which implies a certain duration.

Firstly, a random sample of probabilities with a length of 100 000 cases (p_i), generated from a uniform distribution in the range (0, 1), was transformed into a set of synthetic peak flows (Q_i^s) by an inverse GEV distribution (Eq. 10), which was fitted at each station with the regional method previously discussed. Synthetic peak flows

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keep the statistical properties of the fitted GEV distribution to the observed data at the stations, as is shown in Fig. 3.

$$Q_i^s = u + \frac{\alpha}{k}\left[1-(-\ln(p_i))^k\right] \tag{10}$$

The second step is the generation of synthetic volumes. A synthetic volume could be estimated from a synthetic peak with the regression equation between them, but this would lead to a perfect linear relationship that does not simulate its real variability. Therefore, as the residuals of the regression equation are normally distributed in the log-log space of variables (Fig. 4), a normal randomization was carried out for each synthetic peak flow, with mean equal to the result of the regression equation (Eqs. 5 or 8) and standard deviation equal to the residual variance of the regression (σ_{reg}) (Eq. 9).

The two first steps of the synthetic generation methodology were applied to the observed data at the three case studies. Two sets of 100 000 synthetic volumes were generated at each site from the set of synthetic peaks, one from the local regression equation and another from the regional regression equation. Both regressions were compared to assess their capability to keep the statistical properties of the observed data (Fig. 3).

Both regressions fairly keep the statistics of the AMV. In the Entrepeñas reservoir, the regional regression thoroughly keeps the frequency curve until 2000 years of return period, but for higher return periods the synthetic volumes are smaller than the observed ones. The local regression shows greater volumes for return periods higher than 25 years. In the case of Santillana reservoir, the regional regression fairly fits the frequency curve, but the local regression shows greater differences for return periods higher than 1000 years. The local regression thoroughly fits the frequency curve in the Buendía reservoir, but the regional regression shows small volumes for higher return periods. In each reservoir, both regressions must be compared to select the best one in each case.

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6 Generation of hydrographs

Each Q-V pair must be transformed in a flood hydrograph to be routed through the reservoir. Hydrographs in a river can have multiple shapes as different events can produce different runoff responses. Different methods have been proposed to construct a hydrograph. The selection of a method restricts the shape of the hydrograph and homogenizes the results. Random shapes must be used to relax this restriction. Randomization can be achieved coupling a stochastic rainfall generator and a hydrological model, both calibrated in the basin (Mediero et al., 2007; Garrote et al., 2008). But, if a large and varied enough set of observed hydrographs is available, it can be utilized as a random sample.

A large set of 919 observed hydrographs is available in the 32nd homogeneous region. The variability of hydrograph shapes in this set was measured by two variables: the time of peak (H_p) and the location of the hydrograph centroid (H_c) (Eqs. 11 and 12). These variables were standardized to be dimensionless and comparable, being H_c a modification of the shape mean variable (S_m) developed by Yue et al. (2002). They show enough spreadness to use the set of observed hydrographs as a random sample to generate synthetic hydrographs (Fig. 5).

$$H_p = \frac{t_p}{D} \tag{11}$$

$$H_c = \frac{1}{V}\sum_{i=1}^{D-1}\left(\frac{x_i}{D}\cdot V_i\right) \tag{12}$$

where t_p is the time of peak, in h; D is the hydrograph duration, in h; V is the hydrograph volume, in hm^3; V_i is the hydrograph volume between t_i and t_{i+1}, in hm^3; x_i is the time distance from the beginning of the hydrograph to the centre of V_i, in h.

The third step of the generation was carried out as follows. Firstly, the ratio between peak and volume is computed for each synthetic Q-V pair and the observed hydrograph shape with the most similar ratio is selected. Then, the hydrograph is resized by

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the synthetic peak discharge. The synthetic hydrographs keep the statistical properties of hydrograph durations of the observed data for both regressions at each case study, except for the local regression in the Entrepeñas reservoir that gives much higher durations than observed (Fig. 3).

7 Design flood hydrographs

The DFH is a high magnitude flood hydrograph that ensures the dam safety to a given level and is represented by its low probability to be exceeded. In Spain, the top of the surcharge pool is fixed to not be exceeded by the flood of 1000 years of return period. In practice, the flood hydrograph for a return period of T-years is constructed with the T-year peak flow and the output volume of a hydrological model, calibrated in the basin. In the case that the volume frequency curve is known, the T-year volume is used, so that the T-year flood hydrograph has T-year peak and T-year volume. But, the probability of occurrence of that hydrograph is unknown, as it is the joint probability of the marginal probabilities of peak and volume.

The hydrograph of a T-year return period must be defined in terms of risk to the dam, as the inverse of its probability to exceed a maximum water level in the reservoir or a maximum released flow, instead of estimating its probability of occurrence. Therefore, the risk of a flood can only be known by being routed through the reservoir.

Each set of 100 000 synthetic hydrographs was routed through the corresponding reservoir. Reservoir level at the beginning of the flood was assumed to be at the top of the conservation pool, which is the traditional practice for dam design. For the sake of simplicity, an uncontrolled spillway was assumed, so that the maximum level leads to the maximum release. Then, each set of synthetic hydrographs was sorted according to the maximum water level obtained while routing the hydrograph through the reservoir. Maximum reservoir levels for different T-year return periods were calculated as reservoir levels with an exceedance probability of $1/T$ over the total number of hydrographs (Table 5).

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In the two-dimensional space Q-V, there will not be a unique hydrograph for a T-year return period, but a curve with a set of hydrographs that yield the same maximum reservoir level (Fig. 6). The dependence of the return period on each variable can be known from these curves. The milder the slope of the curve, the greater is the dependence on the volume and the steeper the slope, the greater is the dependence on the peak. For a return period of 5 years, the Buendía reservoir has the mildest curve, which shows peak value ranges from 91.5 to 708.8 m^3/s (1.5–217 years of return period in the marginal distribution) and volume ranges from 60.1 to 191.9 hm^3 (2.4–9.7 years). This means that the return period of hydrographs is mainly given by the return period of volumes. On the other hand, the Santillana reservoir has the steepest curve. The peak ranges from 60.7 to 151.5 m^3/s (1.9–12.5 years) and the volume ranges from 11.2 to 99.3 hm^3 (2–304 years). In this case, the return period of hydrographs is mainly given by the peak discharges. The Entrepeñas reservoir is an intermediate case, with peak ranging from 183.1 to 695.8 m^3/s (2.5–67 years) and volume ranging from 43.3 to 201.5 hm^3 (2.1–21.7 years). The return period of hydrographs depends on both variables.

The risk at the dam and in the downstream reach can be known from the frequency curves of water levels over the spillway crest and releases (Fig. 7). An increase of the top of the dam can be decided from the water depth frequency curve and additional river defenses could be required downstream the dam to achieve a safety level from being flooded.

In addition, the return period curves depend on the spillway length and it can be designed from the probability to exceed a given water level. (Fig. 8). This is very useful in the case that there is a maximum level to not be exceeded, for instance, to prevent a village from being flooded. The spillway length can be selected in terms of risk to exceed that threshold.

Assuming that the water level at Santillana reservoir cannot exceed an elevation equal to 894 m, the exceedance probability of this water level was calculated for different spillway lengths: 6, 9, 12 and 15 m, and these probabilities were transformed

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into return periods (Table 6). A minimum spillway length of 12 m should be selected to have a low enough probability of exceedance and risk to exceed that level, e.g. a return period higher than 1000 years or an exceedance probability lower than 0.001.

In the case that a restriction of maximum discharge downstream of the dam also exists, the spillway length can be selected from both curves, minimizing the risk to exceed a water level and the risk to exceed an outflow discharge downstream of the dam (Fig. 9).

8 Conclusions

A methodology to generate flood hydrographs that keep the statistical properties of peak, volume and duration marginal distributions has been developed. This methodology takes advantage of the regional studies of peak flows and hydrograph volumes that have been carried out recently in Spain, showing that a homogeneous region in terms of peak flow is also homogeneous in terms of hydrograph volume. Accuracy of peak and volume frequency curves was improved thanks to these regional analyses, which lead to a regional shape parameter or regional L-CS, in order to enhance the estimations for the higher return periods.

The relationship between peaks and volumes was analyzed in the log-log space at local and regional scales. A linear relationship exists between standardized peaks and volumes in a homogeneous region. These relationships were simulated by a regression equation and their variability was assessed by the residual variance of the regression.

A large set of synthetic peaks was generated from the peak frequency curve. Volumes linked to these peaks were generated by a regression equation and a normal randomization, to take into account the variability in the relationship between peaks and flows. Finally, a hydrograph shape was linked to each Q-V pair from the ratio between peak and volume. The synthetic sets thoroughly keep the statistics of the peak and duration frequency curves and fairly keep the statistics of the volume frequency curve.

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The set of synthetic hydrographs is very useful for dam design and assessment of dam safety in terms of risk. Routing the synthetic hydrographs through the reservoir, the maximum level and maximum release for each hydrograph can be known, so that the return period can be fixed in terms of the maximum water level at the reservoir. It was seen that there is not a unique hydrograph, but a curve with different combinations of peak and volume, which lead to a given risk and return period. The most influential variable can be known from the slope of these curves. The milder the slope of the curve, the greater is the dependence on the volume and the steeper the slope, the greater is the dependence on the peak.

Probability distributions of water depths over the spillway crest and releases can also be known. These distributions are very useful to assess the safety level of the dam from a hydrological point of view. Finally, the spillway length can be designed in terms of probability to exceed a water level, as risk to the dam, and probability to exceed an outflow discharge, as risk to flood a location downstream the dam.

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Table 1. Main variables of reservoirs: drainage area (A_d), volume up to the spillway crest (V), flooded area at the spillway crest height (A_f), elevation of the spillway crest (H_s).

Reservoir	A_d (km^2)	V (hm^3)	A_f (km^2)	H_s (m a.s.l.)
93 001	4060	710.1	29.56	715
93 033	3256	48.9	5.35	889
93 087	247	1519.2	77.5	710.5

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Table 2. Statistics of the AMD (m^3/s) and AMV (hm^3) series and parameters of GEV distributions fitted with a regional shape parameter.

	Local statistics of AMD				GEV parameters		
Station	Mean	L-CV	L-CS	L-CK	u	α	k
93 033	74.25	0.374	0.191	0.092	48.983	35.135	−0.127
93 001	185.29	0.451	0.356	0.184	109.194	105.808	−0.127
93 087	155.41	0.402	0.266	0.099	98.535	79.070	−0.127
	Local statistics of AMV				**GEV parameters**		
Station	Mean	L-CV	L-CS	L-CK	u	α	k
93 033	14.67	0.367	0.113	0.024	9.226	4.969	−0.348
93 001	63.80	0.537	0.417	0.209	29.150	31.631	−0.348
93 087	80.07	0.674	0.619	0.441	25.496	49.819	−0.348

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Table 3. Heterogeneity tests and regional statistics of AVD series at the 32nd region.

Heterogeneity tests			Regional statistics		
H1	H2	H3	L-CV	L-CS	L-CK
1.7317	1.0543	0.8090	0.5354	0.4138	0.2547

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Table 4. Local and regional regression equations. n is the length of the observed data, ρ is the Pearson's correlation coefficient, a and b are the parameters of the equation and σ_{reg} is the standard deviation of the residuals.

Station	n	ρ	a	b	σ_{reg}
93 033	43	0.7157	−0.3787	0.9221	0.2081
93 001	68	0.8059	−0.8707	1.2236	0.2749
93 087	60	0.5936	−1.3315	1.4335	0.2859
Regional	919	0.7004	−0.6496	1.1057	0.2525
Standardized regional	919	0.6487	−0.0855	1.1272	0.2503

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Table 5. Reservoir levels for different return periods (T) with exceedance probabilities (p).

T (y)	p	Reservoir level (m)		
		Santillana	Entrepeñas	Buendia
5	0.2	890.55	716.17	711.24
10	0.1	890.98	716.52	711.50
50	0.02	891.96	717.28	712.13
100	0.01	892.39	717.62	712.45
500	0.002	893.42	718.54	713.13
1000	0.001	893.85	718.90	713.47

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Table 6. Exceedance probabilities (p) of a reservoir water level of 894 m for different spillway crest lengths in the Santillana reservoir. Exceedance probabilities were transformed into return periods.

Length (m)	p	Return period (y)
6	0.0055	182
9	0.0021	485
12	0.0008	1266
15	0.0003	2857

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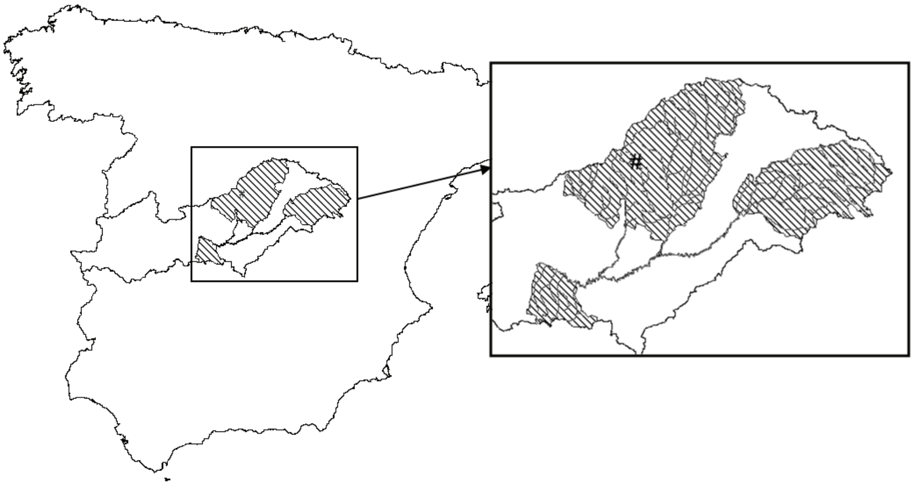

Fig. 1. Location of case studies. Lined area is the 32nd homogeneous region in the Tagus basin.

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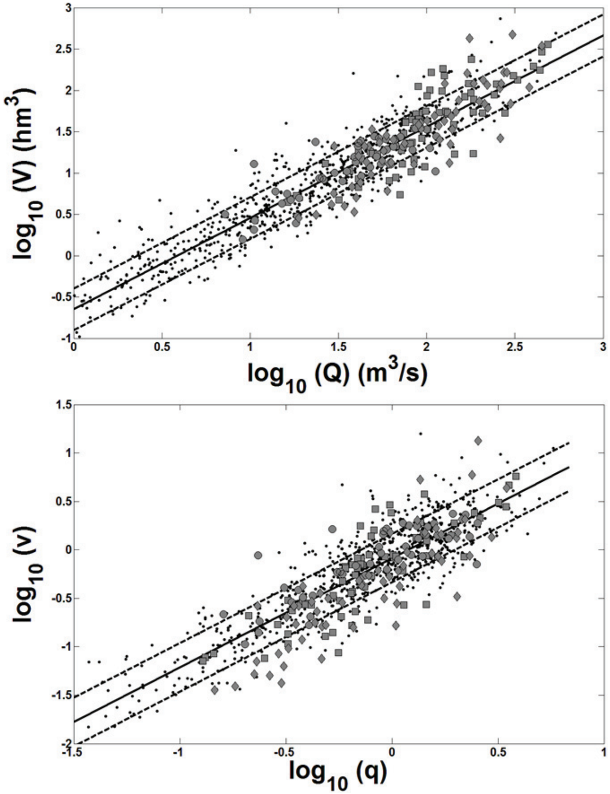

Fig. 2. Relationship between hydrograph volumes and peak flows. Solid line is the regression equation and dotted line shows the confidence interval for a confidence level of 33%. Solid points are the pairs in the whole region, squares are the pairs in the 93 001 station, circles in the 93 033 station and diamonds in the 93 087 station. **(a)** Observed volumes (V) against observed peak flows (Q). **(b)** Standardized volumes (v) against standardized peak flows (q).

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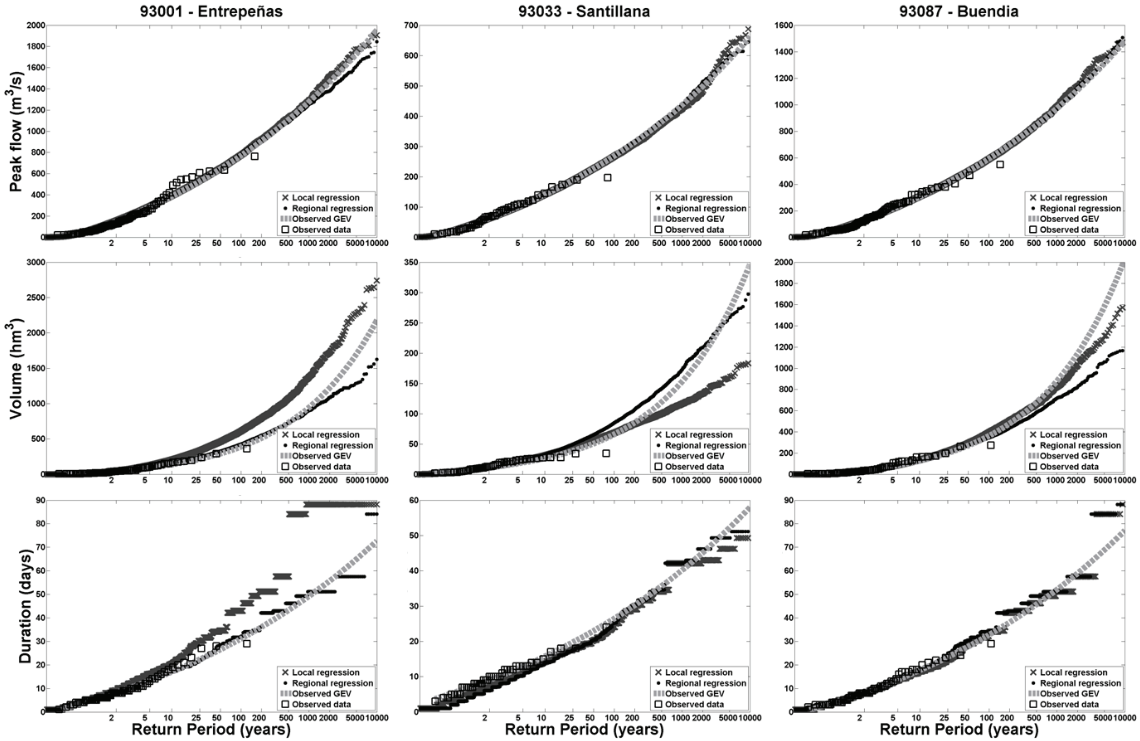

Fig. 3. Comparison between observed and synthetic peaks, volumes and durations from local and regional regressions in the three case studies.

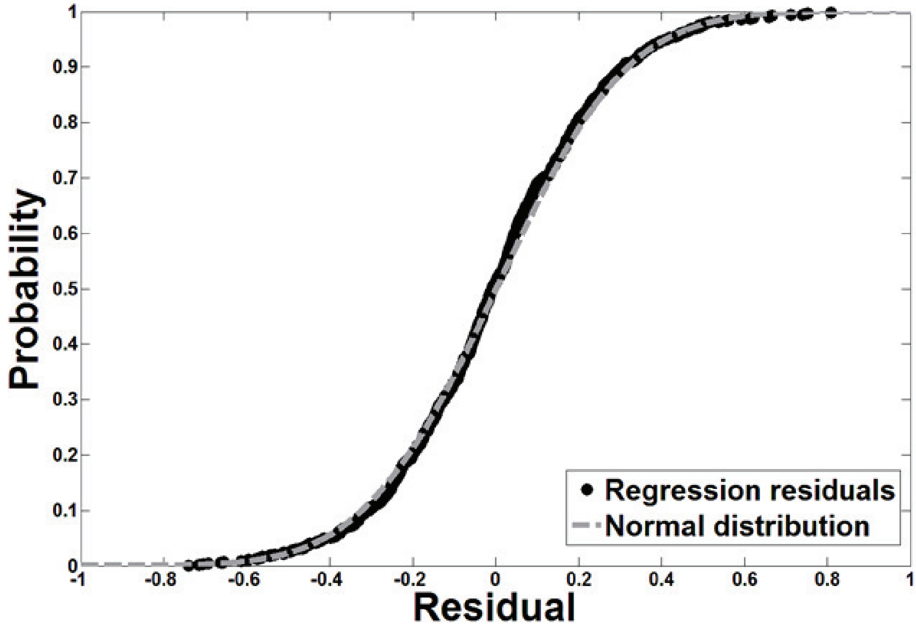

Fig. 4. Normality test of the residuals of the regional regression equation between standardized volumes (v) and standardized peaks (q).

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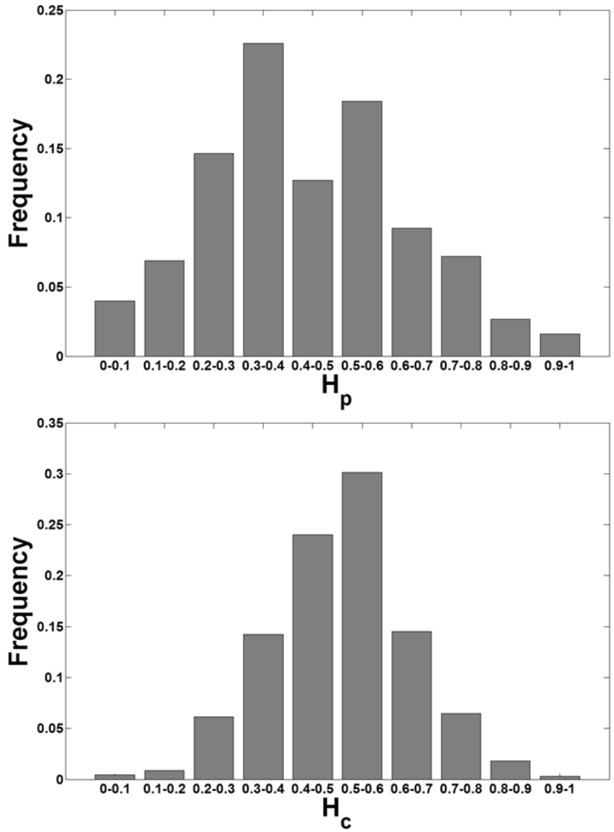

Fig. 5. Histograms of shape hydrograph variables, **(a)** H_p; **(b)** H_c.

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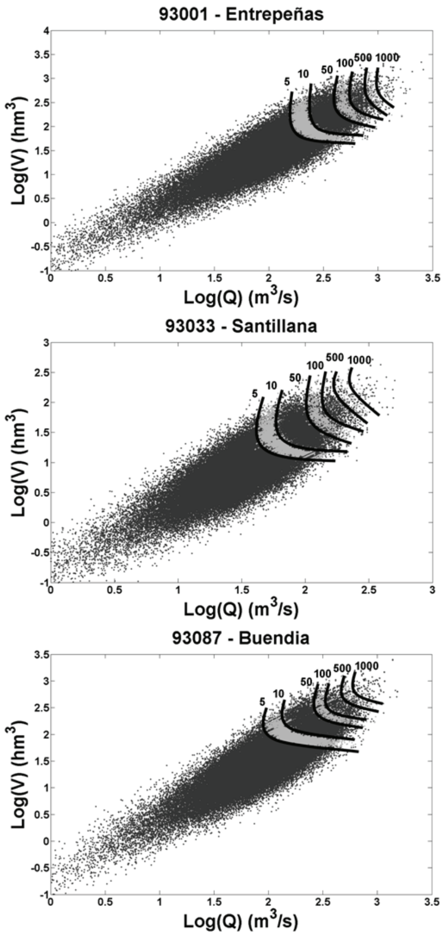

Fig. 6. Return period curves from the maximum reservoir level reached during the routing process.

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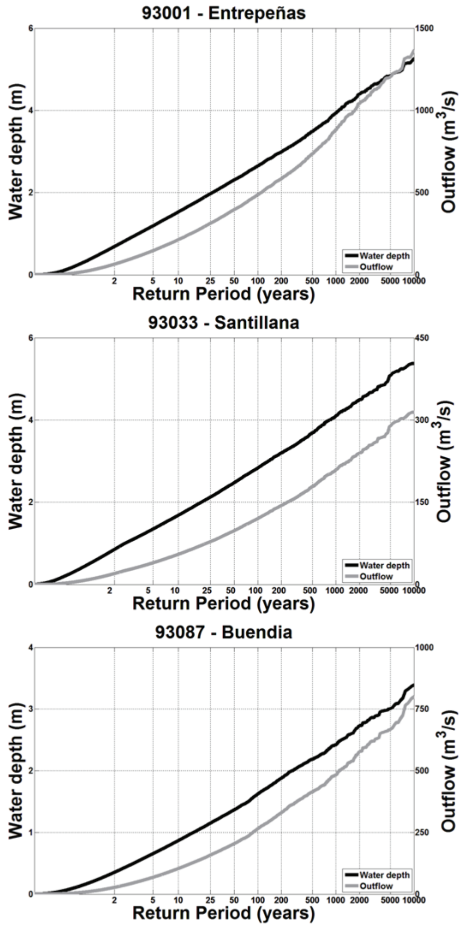

Fig. 7. Frequency curves of water levels over the spillway crest and releases.

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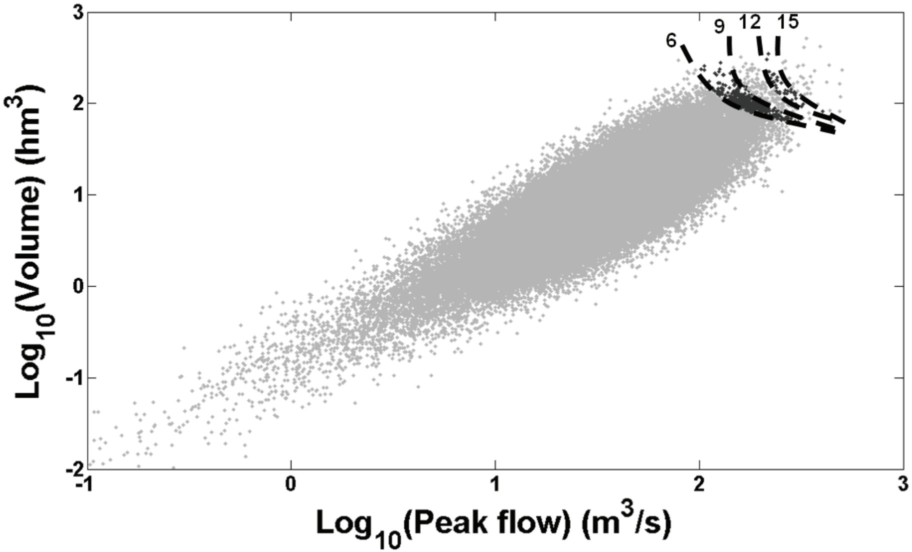

Fig. 8. Curves of floods that lead to a maximum reservoir level of 894 m for different spillway crest lengths: 6, 9, 12 and 15 m.

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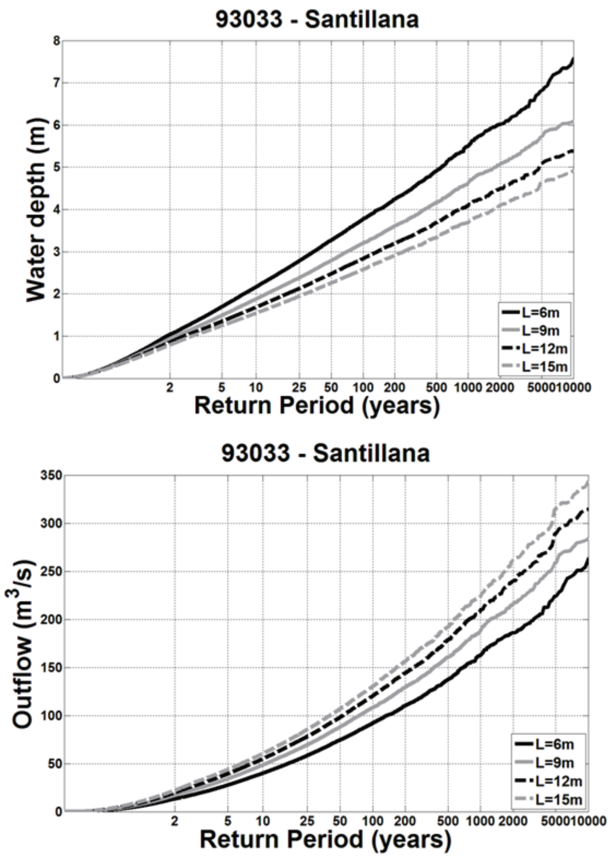

Fig. 9. Frequency curves of water depths over the spillway crest and releases for different lengths of the spillway crest in the Santillana reservoir.

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