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AMBIENTE**

E.T.S.I. CAMINOS, CANALES Y PUERTOS

**Incorporation of Operational Variables
in Stochastic Hydrological Dam Safety
Analysis**

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Realizando el acto de defensa y lectura de la Tesis el díade de 2020, en

Calificación:

*A mis abuelos y a mis padres,
por enseñarme la importancia del esfuerzo y la perseverancia.*

*A Irene,
por complementarme y llenarme de ilusión.*

" Lo peor no es cometer un error, sino tratar de justificarlo, en vez de aprovecharlo como aviso providencial de nuestra ligereza o ignorancia."

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Resumen

A lo largo de la historia, se han dado situaciones en las que, presas diseñadas y construidas acordes a la buena práctica ingenieril han sufrido accidentes con consecuencias catastróficas. En consecuencia, las nuevas regulaciones y normativas tienden a aumentar las exigencias de seguridad hidrológica de las Grandes Presas, influyendo tanto en el diseño como en la adaptación de las presas existentes a los nuevos criterios exigidos. Es necesario por tanto definir metodologías de evaluación que puedan aplicarse al conjunto de presas existentes con la finalidad de identificar aquellas que tengan un margen de seguridad menos holgado y requieran actuación. En España, se adopta el criterio de establecer la seguridad hidrológica en función de un periodo de retorno asociado a las Avenidas de Proyecto. Sin embargo, no existe un criterio riguroso en la definición de este periodo de retorno, dado que la misma avenida puede ser caracterizada a través de su caudal punta, volumen o duración; o por una combinación de estos factores. En la definición de la Avenida de Proyecto, a excepción de la obtención de la Tormenta de Proyecto, la mayoría de variables involucradas se toman como determinísticas, cuando en realidad tienen un carácter estocástico. Variables como el reparto temporal de la lluvia o las condiciones de humedad antecedentes a la avenida son, entre otras, fijadas de manera determinista por el proyectista a pesar del grado de incertidumbre existente sobre la determinación de éstas. Gracias a los avances en informática, la modelación distribuida presenta una alternativa para abordar estas cuestiones. Modelos distribuidos físicamente basados acoplados con generadores estocásticos de clima permiten simular los procesos hidrológicos de manera detallada; tanto temporalmente, como espacialmente.

Además de las variables expuestas, existen otros factores extrínsecos no contemplados en la definición del periodo de retorno de las avenidas asociados a la seguridad hidrológica de la presa, como son el nivel inicial en el embalse, o la operación y disponibilidad de los distintos órganos de desagüe de la presa. Por ello, se considera que las leyes de frecuencia de niveles máximos en el embalse y de caudales máximos desaguados son las variables más representativas de la seguridad hidrológica; tanto de la presa como aguas abajo de ésta. Avenidas de caudales de elevados periodos de retorno pueden resultar menos comprometedoras que otras de menor periodo de retorno si los órganos de desagüe no pueden ser operados correctamente. Por lo que se refiere al nivel

inicial en el embalse, no sólo afecta desde el punto de vista del diseño frente a solicitudes extremas, sino que mantiene una relación directa con la capacidad de regulación del embalse. En muchos de los embalses existentes se adoptan resguardos para paliar las consecuencias de las avenidas extremas, que conllevan una reducción de los recursos disponibles para el suministro de las demandas y, por consiguiente, pérdidas económicas derivadas. Sin embargo, en la literatura y en la práctica profesional, ambos aspectos son generalmente estudiados de forma independiente.

La temática expuesta es de importancia en la actualidad, dado el gran número de presas existentes que han de ser evaluadas para garantizar la seguridad de la población, así como el suministro de recursos hídricos. La presente tesis doctoral busca proporcionar una metodología que, dentro de un entorno de modelación estocástica, mejore: a) la generación estocástica de avenidas, b) la operación de los órganos de desagüe en situación de avenida, c) el análisis de la influencia de variables operacionales (nivel inicial en el embalse y probabilidad de fallo de los órganos de desagüe) y d) la definición de resguardos estacionales en presas teniendo en cuenta la operación ordinaria de la mismas.

Abstract

Throughout history, there have been situations in which dams designed and built in accordance with good engineering practice have suffered accidents with catastrophic consequences. Consequently, new regulations and standards tend to increase the hydrological safety requirements of Large Dams, influencing both the design and the adaptation of existing dams to the new criteria required. It is, therefore, necessary to define assessment methodologies that can be applied to all the existing dams in order to identify those that have a smaller safety margin and require action. In Spain, the criterion for hydrological safety is adopted according to a return period associated with the Design Flood. However, there is no rigorous criterion in the definition of this return period, given that the same flood can be characterized by its peak flow, volume or duration; or by a combination of these factors. In the definition of the Design Flood, with the exception of the Design Storm, most of the variables involved are taken as deterministic, when in fact they have a stochastic nature. Variables such as the temporal distribution of rainfall or the humidity conditions in the basin prior to the flood are, among others, determined in a deterministic manner by the designer despite the degree of uncertainty that exists regarding their calculation. Thanks to the advances in computer science, distributed modeling presents an alternative for addressing these issues. Distributed physically-based models coupled with stochastic climate generators provide a framework to simulate hydrological processes in a detailed manner; both temporally and spatially.

In addition to the aforementioned variables, there are other extrinsic factors not contemplated in the definition of the return period of the floods associated with the hydrological safety of the dam, such as the initial reservoir level, or the operation and availability of the different spillway outlets of the dam. For this reason, the frequency curves of maximum reservoir levels and of maximum released outflows are considered to be the most representative variables of the hydrological safety; both of the dam and the downstream riverbed. Floods with high return periods may be less compromising than others with a smaller return period if the spillways cannot be properly operated. With regard to the initial level in the reservoir, not only does it affect the design regarding dam safety, but it also maintains a direct relationship with the reservoir's regulation capacity. In many of the existing reservoirs, the flood control volume is increased to alleviate the consequences of extreme floods, which lead to a reduction in the water resources

available for the supply of the demands and, consequently, derive in economic losses. However, in the literature and in professional practice, both aspects are generally studied independently.

The subject matter presented is of current importance, given the large number of existing dams that must be evaluated to guarantee the safety of the population, as well as the supply of water resources. This doctoral thesis seeks to provide a methodology which, within a stochastic modeling environment, will improve: a) the stochastic generation of floods, b) the operation of the spillway and dam outlets in a flood situation, c) the analysis of the influence of operational variables (initial level in the reservoir and probability of failure of the spillway outlets) and d) the definition of seasonal conservation levels in dams taking into account the regular operation in the reservoir.

List of acronyms

A	Area of the watershed.
a, b	Dimensionless parameters of Equation (3.4). selected from the CEDEX (MARM, 2011 ; Jimenez-Alvarez et al., 2012) national study.
AL	Activation level.
ARMA	Autoregressive moving average model.
AWE-GEN	Advanced weather generator.
CDF	Cumulative distribution functions.
CEDEX	The Public Works Studies and Experimentation Centre of Spain, Ministry of Development.
$C_{g1-g2\%}$	Scenarios represented by the independent rate of failure of each gate (g1 and g2).
CN	Curve number.
COD	Crest of dam (m.a.s.l.).
$Cost_{break}$	Damage cost if the dam fails.
D	Damage function.
D_{Event}	Duration of a storm event.
DFL	Design flood level.
d_j	Duration of the maximum annual observed flood events (days).
D_{MO}	Damage function associated with Maximum Outflows.
D_{MWRL}	Damage function associated with Maximum Water Reservoir Levels.
D_o	Function of damage costs associated with the maximum outflows.

D_Z	Function of damage costs associated to the maximum reservoir levels.
EAD	Expected annual damage.
FCL	Flood control level.
GEV	Generalized Extreme Value distribution.
GSC	Gated spillway crest (m.a.s.l.).
ICOLD	International Commission on Large Dams.
IDF	Intensity-Duration-Frequency curves.
I_F	Failure risk index.
I_i	Inflow at a time i .
$I_{\max}$	Maximum intensity of a storm event.
IMF	Instantaneous maximum flow.
I_{NF}	Non failure risk index.
I-O	Inflow-Outflow rule of operation method.
IPF	Instantaneous peak-inflow (m^3/s).
IQR	Interquartile range.
I_R	Global risk index.
K	It is also mentioned as K-value. It represents a parameter used in the K-Method
M	Number of years of the observed daily inflows time series.
MCL	Maximum Conservation Level (m.a.s.l.).
MDF	Maximum annual mean daily inflow (m^3/s).
MILP	Mixed Integer Linear Programming rule of operation method.
MIT	Minimum interevent time.

MNL	Maximum Normal Level (maximum reservoir level to which water might rise under normal operation (ICOLD,1994)) (m.a.s.l).
MO	Maximum outflow (m^3/s).
$\text{MO}_{\text{TR}=500\text{y}}$	Maximum outflow that corresponds to a return period of 500 years.
MSM10	Mean soil moisture in the top 10 cm of the soil layer of the basin.
$\text{MSM10}_{\text{Event}}$	MSM10 associated to the beginning of a storm event.
MWL; MWRL	Maximum Water Level (maximum reservoir level which the dam has been designed to stand (ICOLD,1994)) (m.a.s.l).
$\text{MWRL}_{\text{TR}=1000\text{y}}$	Maximum Water Reservoir Level that corresponds to a return period of 1,000 years.
N	Number of events considered.
n	Number of time intervals until the reservoir runs out of flood control capacity.
NSE	Nash Sutcliffe Efficiency Coefficient (Nash and Sutcliffe, 1970).
O_{ALT}	Alert outflow.
O_{EMER}	Emergency outflow.
O_i	Outflow at a time i.
O_{MAX}	Maximum released outflows.
$O_{\text{max.disch.}}(S_i)$	Maximum outflow that can be discharged at the current reservoir level.
$O_{\text{max.Gr}}$	Maximum gate opening/closing gradient.
O_{WARN}	Warning outflow.
P	Objective penalty function applied in MILP.
p	Probability of exceedance.

$p(\text{break} \text{MWRL}_i)$	Probability of overtopping failure conditioned to reaching a certain level in the reservoir.
$p(\text{MWRL}_{\text{MAX}} \geq \text{COD} \text{MWRL}_i)$	Probability of reaching the reservoir level at crest of dam once a certain reservoir level has been reached during a flood event.
PC	Peacheater creek.
PF	Maximum annual hydrograph peak-flow.
PFFC	Peak flow frequency curve.
P_o	Penalty function of released outflows.
P_s	Penalty function of storage volume.
P_t	Total maximum rainfall depth (mm).
Q1	First quartile (25 % percentile).
Q2	Median.
Q3	Third quartile (75 % percentile).
Q _I , Q _{II} , Q _{III} , Q _{IV}	Quadrants of the comparative scheme: first, second, third and fourth respectively.
$Q_{\text{MAX.}}$	Maximum mean daily peak-inflows (m^3/s).
$Q_{\text{min.}}$	Minimum inflow between the two flood peaks compared (m^3/s).
Q_o	Start of the receding limb (m^3/s).
Q_p	Outflow proposed by VEM.
Q_t	Recession flow at any time (t) after the beginning of the receding limb (m^3/s).
R	Reference model.
R^2	Coefficient of determination.
RE	Relative error.
RIBS	Real-time integrated basin simulator.

RMSE	Root mean square error.
S_{AL}	Volume at the activation level.
Sc.1	Scenario in which initial reservoir level was set equal to Maximum Normal Level.
Sc.2	Scenario in which initial reservoir level was set equal to Flood Control Level.
Sc.3	Scenario in which initial reservoir level was considered variable.
SCS	Soil Conservation Service.
S_{FCL}	Volume at the flood control level.
S_i	Reservoir storage at a time i .
S_{i-1}	Reservoir storage at time $i-1$.
S_i^F	Available flood control capacity at a time i .
SPANCOLD	Spanish National Committee on Large Dams.
S_{TCP}	Volume at the top of conservation pool.
T	Tested model.
TCP	Top of control pool.
TIN	Triangulated irregular networks.
T_K	Kendall's return period.
T_r	Return period (years).
tRIBS	TIN-based real-time integrated basin simulator.
T_{RMNL}	Return period of maximum water reservoir levels or maximum outflows obtained assuming constant initial reservoir level equal to MNL (years).
T_{rVAR}	Return period of maximum water reservoir levels or maximum outflows obtained assuming variable initial reservoir level (years).

V	Maximum annual volume (hm^3).
VEM	Volumetric evaluation method (Girón, 1988).
V_{Event}	Total depth of a storm event.
VFC	Volume frequency curve.
w_o	Weight associated to the penalty function of released outflows.
w_s	Weight associated to the penalty function of storage volume.
$Y_{R=90\%}$	Yield that can be supplied with a 90% of reliability.
Z_0	Initial reservoir level (m.a.s.l).
Z_{MAX}	Maximum reservoir levels.
α, u, γ	Parameters of the GEV distribution (scale (m^3/s for Q_{MAX} and hm^3 for V), location (m^3/s for Q_{MAX} and hm^3 for V) and shape (dimensionless) respectively).
β	Recession constant expressed as the inverse units of time (t).
δ	Incremental of non-exceedance probability between two consecutive elements in the sample.
ΔO	Variation of maximum outflows of the tested method respect to the reference model.
ΔO_i	Variation of outflows at a time i.
ΔS_i	Variation of the storage in the reservoir at a time i.
Δt	Time step.
ΔZ	Variation of maximum reservoir levels of the tested method respect to the reference model.
Θ	Time difference between two flood peaks (P_1 and P_2) compared (days).

$\bar{r}$	The variability of the date of occurrence about the mean date ($\bar{\theta}$).
$\bar{\theta}$	Mean date of occurrence of the annual maximum peak-flow/volume.

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1. Introduction

This introduction shows a general overview of the relevant aspects of prior knowledge that lead to the contributions of this thesis. The key motivation of this thesis is to provide new methods and tools for improving the analysis of hydrological dam safety:

- First, in [Section 1.1](#), the motivation and objectives of the research are described, focusing on answering the triplet: what is hydrological dam safety, why its important and how this thesis proposes a new framework for its analysis.
- In [Section 1.2](#), the structure of this doctoral thesis is presented.
- In [Section 1.3](#), the publications associated to this research are listed.

1.1. Research motivation and aims

1.1.1. What is hydrological dam safety?

Historically, the concept of dam safety has been characterized by a unique relationship with absolute safety. Generally, society assumes that dams are absolutely safe, and its exigencies are not related to the execution of the dam as civil works but to the correct response to the uses for which they were designed ([Aranda Domingo, 2014](#)). However, the concept of absolute safety is not possible from a practical point of view. From the point of view of the safety of the dam, the citizen demands no risk, regardless of whether or not the different regulations are enforced. Throughout history, there have been situations in which dams designed and built in accordance with good engineering practice have suffered accidents with catastrophic consequences. Today's society needs the following message to be conveyed: "dams are safe", but, as field practitioners, this assertion cannot be made without some nuances. However, in view of the current situation, what is indispensable is the precision and transmission to society of the concept, as well as the quantification of the level of safety of the dams. All dams must be "sufficiently" safe and conceived as such by the citizen. Therefore, a specific evaluation of each of the existing dams will be needed, but the basic problem is the establishment of this criterion of sufficiency ([De Andrés Rodríguez-Trelles and Mazaira, 2003](#)).

However, this evolution in the thinking of society has not been reflected in the methods for analysing hydrological safety of dams, which have changed relatively little

in recent decades (Sordo-Ward et al., 2012; 2013). Hydrology and dams are two fields of study with a close relationship. However, even though both fields are related, hydrology is studied independently of the type of dam and its characteristics in most of the studies (ICOLD, 2018). Worldwide, the greatest cause of dam failure is overtopping, with an estimated 36% of the total registered dam breaks (ICOLD, 1995). These overtopping failures, in many cases, were not exclusively caused by extreme flood events but also by other causes such as gate operational failures (Hartford et al, 2016).

Furthermore, dams can also cause catastrophic consequences without breaking. Under flood control operation conditions, the spilled outflows may cause floods in the downstream riverbed when the capacity of the river channel is overpassed (Bianucci et al., 2013).

Therefore, hydrological dam safety can be understood as the analysis of how likely: a) a dam can be overtopped and b) the capacity of the downstream river can be overpassed.

1.1.2. Why is important to analyse the hydrological safety of dams in Spain?

Dams in Spain play an important role due to geographical, climatic and socioeconomic characteristics of the country. Spain is a Mediterranean country which presents a great irregularity both spatially and temporally regarding to water runoff resources. Due to these characteristics, the proportion of the net amount of water from natural resources that can be used does not reach 10% (Berga, 2006).

In order to increase water availability, it has been necessary to retain water in the territory through the construction of hydraulic infrastructures such as dams and reservoirs. Currently, the number of catalogued dams in Spain is 1,583 dams (MAPA, 2019). Around 60% percent of the dams in Spain have been constructed between 1950 and 2000, being therefore a key factor in Spain's economic development since mid of the last century. Dams have played a key role in Spain, protecting from floods, producing hydroelectric energy, and serving as operation points of the water resources management. Under the International Commission On Large Dams (ICOLD) guidelines, Spain accounts for a total

of 1,082 Large Dams¹. This figure situates Spain as the ninth country in terms of number of Large Dams around the globe (ICOLD, 2019).

Due to the number of Large Dams in Spain and their importance, there is a variety of different dam owners in Spain, both public and private holders. Around one third of the total Spanish Large dams are owned and operated by Spanish Ministry for the Ecological Transition through the River Basin Authorities which, in addition, hold the authority to enforce and develop integrated water resources planning and management, flood control and environmental protection, among other activities. In accordance with the Water Framework Directive, 18 River Basin Districts have been defined in Spain. River Basin Authorities are public entities with autonomy of action and their own legal personality. Two types of them can be distinguished: inter-regional basins (e.g. Júcar River Basin) and intra-regional basins (e.g. Catalonia Intra-Regional Basins) (Castillo-Rodriguez et al., 2015).

With more than 1,400 dams (including also small dams and ponds), functions and responsibilities of dam owners regarding dam safety management have evolved as a result of a changing dam safety regulation framework. The evolution of the regulations of Dams in Spain is closely related to the catastrophes that have occurred throughout history. After the breakage of the Ribadelago Dam, which occurred in 1959, the Spanish Large Dams Standards Commission was created, which in 1962 proposed “Instrucción para el Proyecto, Construcción y Explotación de Grandes Presas” (Instruction for the Design, Construction and Exploitation of Large Dams), which was finally approved by Order of the Ministry of Public Works on March 31, 1967. This law is partially in force today (MAPA, 2018), being complemented with and partially abolished by “Reglamento Técnico sobre Seguridad de Presas y Embalses” (Technical Code of Dams and Reservoirs Safety), approved on March 12, 1996.

Due to the ageing of dams in Spain, new regulations and standards tend to increase the hydrological safety requirements of dams, influencing both the design and adaptation of existing dams to the new criteria required. It is therefore necessary to define assessment methodologies that can be applied to all existing dams in order to identify those that are less safe and require action plans. This need has been reflected in the National Law Project of the Technical Safety Standards for Dams and Reservoirs (MAPA, 2018),

¹ ICOLD criteria: A Large Dam is a dam with a height of 15 metres or greater from lowest foundation to crest or a dam between 5 metres and 15 metres with a storage of more than 3 million cubic metres.

presently pending approval, which will abolish the previous legislation associated to Dam Safety, providing a new legal framework.

Therefore, as exposed, there is a need on assessing the current state of dams in Spain, in which hydrological dam safety plays a key role.

1.1.3. How we analyse hydrological dam safety? Objectives of this thesis.

The hydrological methods for analysing dam safety have evolved little in recent decades. Specifically, in Spain a methodology based on the characterization of a "Design Storm" is still being applied, which by means of hydrometeorological models turns into the "Design Flood", with which the spillways are designed or checked.

This methodology is mainly deterministic, introducing a single probabilistic component in the determination of the accumulated precipitation in 24 h associated with a specific return period that varies between 100 and 10,000 years. The rest of the variables are defined according to the designer's criteria and intervene in the calculation in a deterministic manner, despite the fact that there is a high degree of uncertainty about them. Among them, we can differentiate:

1. Parameters that characterize the generation of runoff in the basin and its transportation: the duration of the storm, the spatio-temporal simultaneity of storms, the initial state of the basin at the beginning of the storm event, among others.
2. Parameters associated to the dam-reservoir system: the manoeuvring of the different spillways and outlets, the initial reservoir level at the beginning of the flood event, or the probability of blockage or malfunctioning of discharge outlets and spillways.

When accounting for this variables probabilistically, the uncertainties of them can be assessed, and stochastic procedures can be performed to obtain results that resemble more to the reality of nature and the reality of dam-reservoir systems.

Moreover, in many cases, the probability of flood events tells little about the probability of dam overtopping, as the maximum reservoir levels and outflows can be reached due to other circumstances, such as low reservoir storage or various operational faults. Thus, it is essential to change the focus of hydrological dam and downstream safety from floods to an overall overview of the dam-reservoir system.

Therefore, within this thesis, not only the probability of overtopping due to extreme flooding is analysed but also the influence of operational variables such as the initial

reservoir level, the probability of failure of spillways or the flood control operational methods. As a consequence, the main objective of the thesis is the analysis of the influence of the variables of the operation of the dam on its hydrological safety through the development of a complete stochastic methodology for its analysis (Figure 1.1).

Figure 1.1: General scheme of the stochastic methodological approach proposed in this thesis.

To fulfil these purposes, the following specific objectives are defined:

- O.1. Development of a stochastic methodology based on hydrological modelling that allows to generate a representative set of hydrographs at the entrance to the dam for the evaluation of its hydrological safety.
- O.2. Analysis of the operation of spillways and discharge outlets under flood operation scenarios and improvement of their management strategies. To fulfil this objective, by analysing the traditional methodology used in Spanish dams, a new flood operation method has been developed that can be easily applied by the dam operators, reducing the associated risks in terms of hydrological safety, both in the operation of ordinary and extreme floods.
- O.3. Development of a stochastic methodology for the study of different operating variables related to hydrological dam safety, such as the level of the reservoir previous to a flood event. By the use of probabilistic approaches, an assessment of the influence of initial reservoir level is carried out for different case studies.
- O.4. In relation to objective O.3, this thesis develops a framework to assess the influence of the probability of malfunctioning or failure of gated-spillways. Thus,

the objective is to analyse the influence of this operational variable within hydrological dam and downstream safety.

O.5. Joint analysis of the operation of the reservoir, both in regular and flood situations. Development of a methodology that takes into account both factors in the definition of seasonal conservation levels in dams.

Within the framework of the thesis, different case studies have been selected to analyse the performance of the methods proposed in different basins and dam configurations.

1.2. Thesis structure

The results of the research are shown in this doctoral thesis, which is organized according to ten chapters. The motivation, aims and organization of the thesis have already been introduced in the present [Chapter 1](#). A review of the studies performed over the literature leading to the present research is displayed in [Chapter 2](#). The general methodology followed to accomplish the research, including the description and connection among the specific methodologies proposed for each part into which the research is divided, as well as a summary of the considered case studies are shown in [Chapter 3](#). The next chapters are focused on the results of this research, according to the objectives shown in the previous section:

- [Chapter 4](#): An analysis of the number of rainfall events to be considered for a proper hybrid event-based modelling is performed, being this the first part of the proposed hydrological modelling framework. Afterwards, the proposed hydrological Hybrid model is applied, showing its performance in one of the case studies of the thesis.
- [Chapter 5](#): Within this chapter, a new method for the operation of flood discharge structures is presented.
- [Chapter 6](#): In this chapter, an economic analysis through risk indexes is carried out, assessing the impact of accounting for the influence of initial reservoir level and the uncertainty associated to this variable.
- [Chapter 7](#): Within this chapter, the influence of initial reservoir level and gate failure in hydrological dam safety is analysed.
- [Chapter 8](#): A stochastic procedure for defining seasonal conservation levels is proposed.

Finally, conclusions and further research approaches are presented in [Chapter 9](#).

1.3. Summary of research outcomes of this thesis

The research outcomes of this thesis have been published in several journal articles and conferences. A summary of the main publications is shown:

- Papers published in peer-reviewed journals indexed in the Journal Citation Reports (first and last page of each manuscript shown in the Appendix):

1. *Gabriel-Martin, I.; Sordo-Ward, A.; Garrote, L.; Castillo, L.G.; 2017. Influence of initial reservoir level and gate failure in dam safety analysis. Stochastic approach. Journal of Hydrology, 550, 669-684. doi:10.3390/w11030461 JCR: Q1 SJR: Q1*
2. *Gabriel-Martin, I.; Sordo-Ward, A.; Garrote, L.; T. García, J. Dependence Between Extreme Rainfall Events and the Seasonality and Bivariate Properties of Floods. A Continuous Distributed Physically-Based Approach. Water 2019, 11, 1896. doi:10.3390/w11091896 JCR: Q2 SJR: Q1*
3. *Gabriel-Martin, I.; Sordo-Ward, A.; Garrote, L.; Granados, I.; 2019. Hydrological Risk Analysis of Dams: The Influence of Initial Reservoir Level Conditions. Water, 11 (3), 461. doi:10.3390/w11030461. JCR: Q2 SJR: Q1*
4. *Sordo-Ward, A.; Gabriel-Martin, I.; Bianucci, P.; Garrote, L.; 2017. A Parametric Flood Control Method for Dams with Gate-Controlled Spillways. Water, 9(4), 237. doi: 10.3390/w9040237. JCR: Q2 SJR: Q1*
5. *Gabriel-Martin, I.; Sordo-Ward, A.; Santillán, D.; Garrote, L.; 2020. Flood Control Versus Water Conservation in Reservoirs: A New Policy to Allocate Available Storage. Water, 12(4), 994. doi: 10.3390/w12040994. JCR: Q2 SJR: Q1*

- Paper published in peer-reviewed journals indexed in the Emerging Sources Citation Index:

6. *Gabriel-Martin, I.; Sordo-Ward, A.; Garrote, L.; 2018. Influencia del nivel inicial en la definición de resguardos estacionales en presas. Ingeniería del agua, 22 (4), 225-238. doi: 10.4995/ia.2018.9526. (In Spanish) JCR: - SJR: -.*

Furthermore, alongside the research outcomes previously exposed, research in fields related to this thesis has been performed during the PhD studies:

- Paper published in peer-reviewed journal indexed in the Journal Citation Reports:

7. *Bejarano M.; Sordo-Ward, A.; Gabriel-Martin I.; Garrote, L.; 2019. Tradeoff between economic and ecological effects of hydropower production under different environmental flows scenarios. Journal of Hydrology, 572, 790-804,. doi:10.1016/j.jhydrol.2019.03.048. JCR: Q1 SJR: Q1*

- Paper published in peer-reviewed journal indexed in the Scopus Database:

8. *Sordo-Ward, A.; Gabriel-Martin, I.; Perales-Momparler, S.; Garrote, L.; 2019. Influencia de la precipitación en el diseño de SUDS. Revista de Obras Públicas, 166(3607), 28-31. ISSN: 00348619 JCR: - SJR:Q4*

2. Literature review

An analysis of the developments in hydrological safety in both scientific and professional fields is carried out within this chapter, focusing on the aspects exposed in Chapter 1. This chapter is divided into the following subsections (Figure 2.1):

- **Section 2.1:** An introduction to this chapter is presented, comparing the proposed stochastic framework of this thesis with the current traditional approach. The relationship of hydrological dam and downstream safety with the maximum reservoir level and maximum outflow frequency curve is analysed.
- **Section 2.2:** An analysis of the literature regarding to the determination of hydrological forcing in hydrological dam safety is carried out.
- **Section 2.3:** An analysis of the different operative schemes for the management of controlled spillways under flood conditions is carried out.
- **Section 2.4:** The influence of accounting for the variability of initial reservoir level is reviewed, analysing how different authors have accounted for this variable.
- **Section 2.5:** A review of the different ways of accounting for spillway reliability is performed.
- **Section 2.6:** Finally, a review of the literature regarding to the definition of seasonal maximum conservation levels is performed.

Figure 2.1: Sections of the Literature review associated to the hydrological dam safety stochastic approach proposed.

2.1. Hydrological dam and downstream safety

Failure of Large Dams is a concern in many countries due to the high economic and social consequences associated to it. When designing a dam, engineers usually apply techniques to assure that the risk associated to dam failure is low, being the standards applied differently depending on the country in which the dam is located ([Rettemeier and Köngeter, 1998](#); [Ren et al, 2017](#)). Even though the risk assumed is low, the associated risk should be recalculated, as the legal regulations, climate conditions, basin and dam characteristics may vary along time ([Fluixá-Sanmartín et al, 2018](#)).

In the standard engineering approach, hydrological assessment of dam and downstream safety for a given return period is analysed with deterministic methods. A design flood hydrograph is obtained and routed through the reservoir-dam system assuming constant reservoir level prior to the arrival of the flood and equal to the maximum normal operating level. The Design Flood, characterized by its peak flow, is used for the design of the dam outlets without considering any safety coefficient as is usual in other types of structures. The Design Flood in Spain is associated with the return period of total accumulated precipitation in 24 hours, thus characterizing the risk associated with it and therefore the safety of the dam ([Sordo-Ward et al., 2012](#)).

The wide use of this methodology has been due to its easy application and few information requirements. However, scientific advances in the field of hydrology combined with an increase in computational capacity, the development of geographic information systems and spatial modelling processes have made it possible to propose stochastic models for the extreme characterization of the hydrological behaviour of basins.

Many of the involved variables in hydrology have a stochastic nature ([Carvajal et al., 2009](#); [Sordo-Ward et al., 2013](#)). In recent decades, many authors proposed probabilistic approaches accounting for the randomness associated with different variables (e.g.: [Eagleson, 1972](#); [Arnaud and Lavabre, 2002](#); [De Michele et al., 2002, 2005](#); [Carvajal et al., 2009](#); [Sordo-Ward et al., 2012; 2013](#); [Paquet et al., 2013](#); [Bianucci et al., 2013, 2015](#); [Brigode et al., 2014](#); [Flores-Montoya et al., 2015, 2016](#)). Several authors had been able to obtain accurate maximum peak-inflow frequency curves within a Monte Carlo Framework ([Loukas, 2002](#); [Rahman et al., 2002](#); [Arnaud and Lavabre, 2002](#); [Aronica and Candela, 2007](#)). However, it is a matter of importance to characterize

not only the extremal incoming reservoir floods but the hydraulic behaviour and response of dams, as their failure could have catastrophic socio-economic consequences (Serrano-Lombillo et al., 2010; 2016).

Flood control has two fundamental objectives, which are to guarantee the safety of the dam, by preventing the maximum level of the reservoir from being exceeded as well as to avoid overflows to minimize the damage caused by flooding downstream of the dam. Several authors (Mediero et al., 2010; Bianucci et al., 2013; Serrano-Lombillo et al., 2012a; Aranda Domingo, 2014; Micovic et al., 2016; Michailidi and Bacchi, 2017) pointed out that hydrological dam safety and downstream safety should be assessed by analysing the return periods of the maximum reservoir water levels and maximum outflows respectively. This way, the hydrological dam safety analysis does not only depend on the hydrological forcing, but also on the dam and reservoir characteristics and operation rules.

2.2. Hydrological forcing

Firstly, a distinction should be made between the basic evaluation criteria for the selection of floods and the calculation methods for developing these criteria. The former is generally defined by the risk that a given community is willing to accept in the face of possible failure or collapse of the dam and depends not only on technical and economic considerations but also on social, political, cultural and environmental ones.

As far as flood estimation methods are concerned, there are numerous classification criteria based on: the origin of the information to be considered (based on rainfall or flow data), calculation methodology (deterministic or probabilistic), expected results (peak flows or hydrographs), temporal scope (continuous or event-based methods), among others.

In Spain, the Technical Guidelines for Dam Safety (SPANCOLD, 1997) define two large groups, the deterministic type and the probabilistic type. In these groups, the deterministic method is understood to be that which unequivocally calculates the maximum flood flow from meteorological and hydrological data. In this group, the "Probable Maximum Flood (PMF)" method is highlighted (USACE, 1991). Probabilistic methods are considered to be those that carry out a study based on available data (rainfall

and/or flow) and determine peak flows and flood hydrographs for different return periods. The latter is the one currently applied in Spain.

This methodology makes it possible to characterize hydrological risk as a function of peak flow, but there are other factors that influence the response of the dam and reservoir to floods. In fact, the entire hydrograph is of interest for the design and risk assessment of dams. The hydrograph must be abated by the reservoir through the dam to know how it transforms into a series of outflows and different level variations in the reservoir. For this reason, the univariate analysis of peak flows should be extended to a multivariate analysis that takes into account other variables such as volume and duration that allow the construction of complete hydrographs.

The national regulations of different countries establish a return period for the design of dams (in Spain from 500 to 10,000 years) ([Rettemeier and Köngeter, 1998](#); [Ren et al, 2017](#)). However, they do not specify whether the return period is associated with the peak flow, the volume of the hydrograph or the entire hydrograph. Design return periods are commonly associated to the return period of the design flood hydrograph, considering that rainfall events generate hydrographs with peak-inflows with the same return period. Many authors questioned this hypothesis (e.g.: [Adams and Howard, 1986](#); [Alfieri et al., 2008](#); [Viglione and Blösch, 2009](#); [Sordo-Ward et al., 2014](#)). It is also well-known that return periods associated to peak-inflows are not the same as those associated to hydrograph volume, existing a trend to analyse this issue by using copulas for multivariate flood frequency analysis (e.g.: [De Michele et al., 2002](#); [Shiau et al. 2006](#); [Salvadori et al., 2011](#); [Requena et al., 2013](#); [2016](#); [Aranda Domingo, 2014](#)). This question can be extended to the return period associated to the hydrological safety of dams. In addition, the risk associated with an event may be undervalued (or overvalued) if only the return period associated with one of the variables that characterize the hydrograph is analysed ([De Michele et al., 2002](#); [Salvadori et al., 2011](#)). In fact, as exposed in [Section 2.1](#), the return period should be defined in terms of the risk associated with overtopping the crest of the dam or with the downstream damage, rather than in terms of the probability of occurrence of natural events, to take into account the characteristics of the reservoir, the dam and its spillway ([Mediero et al., 2010](#); [Bianucci et al., 2013](#); [Serrano-Lombillo et al., 2012a](#); [Aranda Domingo, 2014](#); [Micovic et al., 2016](#); [Michailidi and Bacchi, 2017](#)).

To characterize the hydrological forcing, several methods can be applied and are mainly classified according to two main groups:

- Statistical flood frequency analysis: statistical methods are generally based on processing existing local and regional flow through statistical procedures such as distribution fitting ([USWRC, 1976](#); [USGS, 1982](#)), making appropriate use of historical references where available.
- Derived flood frequency simulation methods: by the use of a hydrological model, a set of hydrographs is obtained from which a flood frequency distribution can be derived.

Statistical methods need large flow records (hardly available ([Zhang and Shing, 2017](#); [Klein et al., 2010](#))), have the drawback of the uncertainty associated to the distribution fitting for large return periods ([Katz et al., 2002](#)), and provide a value of peak-flow, volume, or duration but not the hydrograph shape. Furthermore, when they are applied in ungauged basins, the physical processes that occur in the watershed are not usually considered and the uncertainty on the flood quantiles estimation increases ([Salinas et al., 2013](#)).

Due to these reasons, derived flood frequency methods are generally preferred over statistical ones ([Li et al., 2014a](#)). Derived flood frequency methods can be divided into two approaches: continuous simulation and stochastic event-based methods ([Li et al., 2014a](#)).

Using a continuous distributed hydrological model provides the user the possibility of deriving flood frequency distributions from the continuous hourly streamflow obtained at any desired point in the drainage network of the basin by forcing the distributed hydrological model with an hourly stochastic weather generator. This approach has the advantage of estimating the variables for the entire period of simulation. However, continuous models tend to be more complex than event-based models, with computational efforts that could be very intensive, even when using high performance computing and parallelization processes.

On the other hand, event-based simulations require much shorter simulation times. However, these methods are based on the assumption that the flood hydrograph has the same return period as the storm event, which is (as previously exposed) not realistic ([Adams and Howard, 1986](#); [Alfieri et al., 2008](#); [Viglione and Blösch, 2009](#)). Furthermore,

different properties of storm events could affect the derivation of flood frequency curves: rainfall temporal distribution, event duration, maximum intensity, and total storm depth, among others.

Examples of both derived flood frequency approaches can be found in the existing literature. In the case of event-based approaches, some studies have combined non-complex stochastic storm generators (i.e., [Sordo-Ward et al., 2012; 2013](#)) or complex rainfall generators (i.e., [Foufoula-Georgiou, 1989](#)); with semidistributed (i.e., [Franchini et al., 1996](#)) or distributed models (i.e., [Flores-Montoya et al., 2016](#)). In the case of continuous simulations, in order to reduce the computational cost, most authors have worked with lumped or semidistributed models (i.e., [Haberlandt et al., 2008; Brocca et al., 2013](#)), and some of them have worked with distributed models by using high performance computers ([Blazkova and Beven, 2004; Vivoni et al., 2011](#)).

To overcome the limitations in the two approaches, several authors have proposed combining event-based models with continuous models. [Paquet et al. \(2013\)](#) proposed the SCHADEX model, which consists on replacing stochastic rainfall events within a short continuous simulation with observed data and a Monte Carlo framework. [Li et al. \(2014a\)](#) developed the hybrid-CE approach, which consists of combining continuous long-term simulations of rainfall and short continuous hydrological simulations to probabilistically characterize the rainfall and initial soil moisture (respectively) to force an event-based model. Both approaches used lumped models.

2.3. Spillway operation

Flood management aims to guarantee dam safety, minimize downstream floods, and maintain the full operational capacity of reservoirs once a flood is over ([Wurbs, 2005; Wang et al., 2010; Hossain and El-Shafie, 2013; Feng and Liu, 2014; Pan et al., 2015](#)). Dams with gated spillways represent around 30% of large dams around the world ([ICOLD, 2003](#)), and provide more possibilities for water conservation and flood abatement than those with fixed-crested spillways ([Sordo-Ward et al., 2013](#)). Gate management during a flood event represents a challenge for the dam operator, who makes decisions under pressure and during uncertain conditions. In addition, the time frame for decision making is usually extremely short, the information available is generally sparse, and the predictability of the meteorological situation is limited ([Afshar and Salehi, 2011; Molina et al., 2005](#)). Real-time flood control operations at dams have been historically

approached via predefined rules obtained by simulation techniques (Li et al., 2014b), or by using simulation methods such as the Volumetric Evaluation Method (VEM) (Girón, 1988), commonly used in Spain. Flood control policies generally establish the discharge at each time step by considering the available information at a previous time, e.g., inflow, reservoir stage, stored volume, and outflow discharge downstream, among others (Loucks and Sigvaldason, 1982). Dam Master Plans include this information, which helps operators to be efficient (Wurbs, 2005). These operation rules are represented and assessed by simulation models, which are usually more flexible and easier to interpret for the dam operators than other schemes, such as optimization (Needham et al., 2000; Rahman and Chandramouli, 1996; Chang, 2008; Le Ngo et al., 2007; Oliveira and Loucks, 1997; Bianucci et al., 2013) or data-based learning models (Chang and Chang, 2006; Chang et al., 2010; Ishak et al., 2011; Khalil et al., 2005; Mediero et al., 2007; Cuevas Velasquez, 2015).

Several simulation frameworks have been developed to tackle this issue. The software HEC-ResSim (Klipsch and Hurst, 2007) permits one to obtain the discharged outflow for each time step, accounting for the operation rules and simulating the behaviour of the reservoir. Flood Control-Reservoir Operator's System software (Karbowski, 1991) supports decision making by providing important data and possible operation alternatives (as a result of the control algorithms implemented in the program). Other models integrate real-time reservoir operation with flood forecasting. They account for the uncertainty associated with the hydrologic loads, by analysing different forecasting techniques for reservoir inflows and by including deterministic and probabilistic approaches (Zhao et al., 2011). A further example is the integrated management system for flood control at reservoirs developed by Cheng and Chau (2001), which gathers real-time information. This is processed in a central database management system in order to evaluate the alternatives of flood operation.

The improvements in the field of optimization and data-based learning allow an approach to the problem which accounts for the uncertainty of the hydrologic loads in real-time forecasting (Liu et al., 2015a) and permits the development of operation policies incorporating different objectives (Choong and El-Shafie, 2015; El-Shafie et al., 2014; Hosseini-Moghari et al., 2015; Ahmadi et al., 2014). Most of the optimization methods are based on an objective function, which minimizes the outflows and the maximum reached reservoir levels (Bianucci et al., 2013). A wide variety of optimization models

have been applied to flood reservoir operations: linear programming, non-linear programming, successive linear programming, stochastic dynamic programming, genetic algorithm, genetic programming, particle swarm optimization, Honey-Bee mating, Artificial Bee Colony, and a combination of the aforementioned (Choong and El-Shafie, 2015). However, there is still a gap between the theoretical development and practical implementation of these models (Hossain and El-Shafie, 2013; Ahmed and Mays, 2013). Optimization models are usually limited as they depend on specific parameters (penalty functions and coefficients of the objective function, among others) which need the operator's expertise on mathematical programming (Bianucci et al., 2013). In addition, due to the uncertainty associated with hydrologic loads and the limitations of the flood forecasting, dam managers usually prefer simulation methods (Jood et al., 2012).

Therefore, the operation of the flood discharge structures defines how the dam–reservoir system should respond when forced by a flood event (Fluixá-Sanmartín et al., 2018). The correct operation of the dam outlets and spillways permits to maintain the safety levels, downstream and within the dam.

2.4. Initial reservoir level

The field of dam risk assessment has evolved worldwide, with the appearance of different guides and procedures in several countries (ANCOLD, 2003; USACE 2011; SPANCOLD, 2012) to support dam stake-holders in the decision-making process related to dam safety. It is also well-known that, due to the high variability of the natural processes, the assessment of dam safety introduces uncertainties that should be assessed as part of the process. However, analysing this uncertainty is a complex task (Bianucci et al., 2015; Sordo-Ward et al., 2014).

Moreover, human actions also provide additional sources of uncertainty to the analysis. One of those variables related to human actions is the variability of initial reservoir level, due to its connection to the operation of the reservoir. As reservoir levels fluctuate (when a flood arrives the reservoir may be partially full) the assumption of considering the reservoir full prior to a flood event could be conservative in hydrological safety analysis (Carvajal et al., 2009). However, this assumption is a practice commonly implemented among dam designers and national technical guidelines (Bianucci et al., 2013).

Thus, many authors considered initial reservoir level as constant (e.g.: [Hsu et al., 2011](#); [Serrano-Lombillo et al., 2012b](#); [Sordo-Ward et al., 2012](#); [2013](#); [Bianucci et al., 2013](#)), but some considered its variability:

- In 2005, a study was carried out using a probabilistic methodology on the Ceppo Morrelli dam, a vaulted dam with a fixed-crest spillway located in Italy ([De Michele et al., 2005](#)). Within the framework of their work, they used a previous level in the reservoir obtained randomly by means of an empirical distribution function of the historical level in the reservoir.
- [Kwon and Moon \(2006\)](#) analysed overtopping probability in Soyang Dam (South Korea). They assessed initial reservoir level by accounting for the reservoir levels probability distribution of the rainy season in the dam, concluding that initial reservoir level was the most sensitive variable for the estimation of dam overtopping probability.
- In 2009, a team of engineers ([Carvajal et al., 2009](#)) analysed, among other factors, the influence of the variability of the initial level in the reservoir at the time of flooding. They analysed three dams of different characteristics. They concluded, in general, that the inclusion of the variability of the initial level in the reservoir is appreciable in the case of reservoirs in which there are considerable seasonal fluctuations in the level, being of less interest in the case of dams with constant levels throughout the year.
- [Peyras et al. \(2012\)](#) studied the operation of a roller-compacted gravity dam by modelling the initial situation of the reservoir based on an empirical distribution of monitored levels in the reservoir following the methodology of [Carvajal et al. \(2009\)](#).
- In a thesis presented in the Technical University of Valencia, [Aranda Domingo \(2014\)](#) analysed the influence of this factor on a dam with a fixed-crest spillway, specifically in the Cueva Foradada reservoir. He concluded that the influence of this factor is significant, being closer to the current reality of the dam-reservoir system.
- [Micovic et al. \(2016\)](#) analysed three dams in cascade in Canada. In their modelling framework, the stochastic simulation of floods on the system of three dams shows progression from exceedance probabilities of reservoir inflow to exceedance probabilities of overtopping depending on the initial reservoir level, storage availability, reservoir operating rules and availability of discharge facilities on demand. They concluded that overtopping is more likely to be caused by a

combination of a small flood event and an operational component failure than by an extreme flood event on its own.

It is therefore important to take this factor into account for the realistic treatment of the hydrological safety of dams. All previously mentioned studies concluded that accounting for reservoir levels fluctuations in hydrological dam safety is desirable, moreover when the dam is operated in seasonal basis as is the case of reservoirs which main purpose is irrigation, regardless of the spillway and dam typology.

2.5. Spillway reliability

As aforementioned, dams are built with different purposes, such as power production, water supply, flood mitigation and other things. A dam system, comprises the body of the dam along with all the facilities to trespass the water from the reservoir, such as outlets, conduits, channels or spillways. Nowadays, dam safety assessment is currently based on analysing if the dam is able to withstand extreme loads, such as the Design Flood or its ability to withstand the design earthquake. Even though the importance of extreme loads is unquestionable, experiences have shown that many dam failures have been caused due to operational events rather than geophysical extreme events ([Hartford et al, 2016](#)). One of the dam facilities that are more relevant to hydrological dam safety are spillways. Specifically, those spillways that are composed by gates. In this type of dams, the manoeuvring of spillways is fundamental. As previously exposed, gated-spillway dams represent about 30% of the large dams around the world ([ICOLD, 2003](#)). The malfunction of gates has caused important dam failures along history in different parts of the world. Among them, some dam failures can be highlighted in which spillway malfunctioning was one of the causes of failure:

- Elko Dam in Canada in June of 2013.
- Folsom Dam in The United States of America, in July of 1995.
- Noppikoski Dam in Sweden, on September 7, 1985.
- Tous Dam in Spain, on October 20, 1982.
- Canyon Lake Dam in The United States of America, on June 9, 1972.

Therefore, accounting for gate functionality may be of importance as gate blockage may result in an increase of the risk of dam failure ([Lewin et al., 2003](#); [Patev and Putcha,](#)

2005; Escuder-Bueno et al., 2012; SPANCOLD, 2012; Micovic et al., 2016). Gates may not open when needed because of human, mechanical, electrical failures, lack of access to the manoeuvring chamber, blockage of spillway outlets, or failure of gate or valve control software (Lewin et al., 2003; Patev and Putcha, 2005; SPANCOLD, 2012; Fluixá-Sanmartín et al., 2018).

Observed data regarding gate functionality is sparse. Some studies in literature have analysed the unavailability of gates:

- In the case of the Directorate-General for Public Works and Water Management of the Netherlands, they conducted a reliability analysis of gates in a Storm Surge Barrier, estimating an unavailability of 0.1% on demand (van Iperean, 1984)
- Lagerholm (1996) performed a survey on major dam stakeholders in Sweden, obtaining a statistic of annual failure of gates of the order of 0.1 in common gate types such as radial or vertical roller gates.
- Lewin et al. (2003) estimated that the value of failure on demand associated to a single gate was 10%, while the multiple failure rate of gates due to common causes was assumed as 1% per demand.
- In other study about the Thames River defences (UKAEA, 1987) an unavailability per gate demand of 0.0155 % was estimated.
- Micovic et al. (2016) assumed a probability of 1% for single gate failure on demand, but pointed that this assumption could not be safe.

Therefore, gate malfunctioning is a field in which stats samples are scarce. One of the important issues about gate malfunctioning is its maintenance. Deterioration could derive in a useless spillway. In dam risk analysis qualitative description could be an alternative to analyse the probability of failure within the context of hydrological dam and downstream safety. To do so, the quantitative individual reliability of a gate can be associated with the description of the condition of the gates (Fluixá-Sanmartín et al., 2018). Thus, Escuder-Bueno et al. (2012) proposed the following relationship between the condition of gates and its probability of failure (Table 2.1)

Table 2.1: Reliability estimation of gates with respect to their maintenance condition.

Gate Condition	Reliability (%)
Brand-New / Non-gated	100
Very well maintained	95
Some minor problems	85
Some problems	75
Non-Reliable	50
Unreliable at all	0

This individual gate-failure rates can be complemented with more complex analysis such as the use of fault trees, indicating different manners in which the gate can fail and disaggregating the probabilities of failure (Lewin et al., 2003).

2.6. Definition of Maximum Conservation Levels

Up to this section, the focus was on hydrological dam safety. As exposed, owing to the increasingly risk-averse societies, stronger hydrological safety requirements are being imposed on dams in order to fulfil new regulations and prevent dam failures (Bocchiola and Rosso, 2014). This problem can be approached with two different kinds of technical solutions: hard solutions, which could be the refurbishment of dam spillways or elevation of dam crest; and soft solutions as the allocation of additional flood control volumes. Hard solutions increase the flood control capacity while maintaining reservoir storage available for water supply. However, their main drawback is the need to allocate resources for infrastructure works. On the other hand, the implementation of soft solutions is easier and quicker, but can reduce the reliability of the system regarding to water supply.

Allocation of flood control volume is addressed by defining a Maximum Conservation Level, also known as Flood-Limited Water Level (Li et al., 2010) or Flood Control Level. The Maximum Conservation Level is the maximum operating level that the reservoir is allowed to reach under regular operation conditions. This reservoir level is below the Maximum Normal operating Level, and can vary along the seasons of the year.

When increasing flood control volumes by soft solutions, there is a trade-off between flood control and water supply demands supplied in which the most significant parameter is the Maximum Conservation Level (Xie et al., 2018).

Traditionally, practitioners only focused on hydrological dam and downstream safety when defining Maximum Conservation Levels, not accounting for other purposes

of the reservoir, such as water supply. This may imply economic consequences related to water demand reliability (Solera-Solera et al., 2012).

Some authors have focused on accounting simultaneously for both, regular (associated to water supply purposes of the reservoir) and flood control dam operation when defining Maximum Conservation Levels. Liu et al. (2015b) proposed a methodology to define seasonal Maximum Conservation Levels accounting for both flood control and water supply and Chang et al. (2017) defined variable sub-seasonal Maximum Conservation Levels to increment hydropower benefits by using fuzzy logic techniques. Other authors have studied seasonal Maximum Conservation Levels within deterministic approaches (Chen et al., 2013; Zhou and Guo, 2014; Fu et al., 2014; Ouyang et al., 2015; Moridi and Yazdi, 2017; Xie et al., 2018).

Other relevant factor is that practitioners usually define Maximum Conservation Levels assuming that the reservoir is at the maximum level under normal operating conditions prior to flood arrival (USACE, 1972; Sordo-Ward et al., 2013). While this hypothesis, as previously mentioned, results into conservative hydrological safety assessments, it can reduce the reliability of the water supply system as it has an effect on the definition of Maximum Conservation Levels.

As exposed, accounting for all the operational variables (i.e.: reservoir level variability) in hydrological dam and downstream safety leads to more realistic results, which consequently can lead to a better definition of Maximum Conservation Levels.

3. Methodology

The general methodology proposed to carry out the present research is described in this chapter. Summarized explanations regarding the specific methodologies performed for each part forming the research, as well as the links among them, are also shown. Further explanations are developed within the following chapters, in the context of the different research studies. Besides, a summary of the case studies considered for the application of such methodologies, and an outline of the software used are included.

As exposed in [Section 1.1.3](#), the objective of this thesis is to develop a stochastic methodology to assess hydrological dam and downstream safety while accounting for operational variables such as the initial reservoir level, the availability of spill outlets or the influence of the flood operational methods.

Even though the methodology must be understood as a whole unit, the following chapters develop specifics parts of it that can be combined into the following scheme ([Figure 3.1](#)).

Figure 3.1: Stochastic proposed framework of hydrological dam and downstream safety assessment.

Thus, following the objectives shown in [Section 1.1.3](#), the methodology framework is introduced in the following subsections:

- [Section 3.1](#): Hydrological forcing.
- [Section 3.2](#): Spillway operation.
- [Section 3.3](#): Initial reservoir level.
- [Section 3.4](#): Spillway reliability.
- [Section 3.5](#): Hydrological dam and downstream safety.
- [Section 3.6](#): Economic risk analysis.
- [Section 3.7](#): Maximum Conservation Levels.

Finally, a brief introduction to the case studies and software used in the thesis is shown in [Section 3.8](#).

3.1. Hydrological forcing

Three methods have been used in this thesis to generate a representative ensemble of inflow hydrographs. Two of them are fully developed within the framework of this thesis and described in this section. The third one is an application of the method developed by [Sordo-Ward et al. \(2012, 2013\)](#) and is also introduced. The main characteristics of the methods are:

- A Hybrid stochastic event-based continuous distributed approach (herein Hybrid method). It consists on the use of a distributed physically based model, coupled to a stochastic climate generator. The recent advances in parallel computing are accounted by using a High Performance Computing system. This method can be applied to any case study, but it requires high-detailed spatio-temporal weather and flow data.
- A stochastic data-based hydrological forcing generator. It consists on a simpler approach of multivariate flood modelling using a lumped stochastic event-based model that allows the generation of hydrographs from historical floods observed at the dam entrance. This method can be applied to any dam with daily inflow data.
- A stochastic event-based hydrological forcing generator. This method, previously developed by [Sordo-Ward et al. \(2012, 2013\)](#), consists on a stochastic semi-

distributed event-based hydrometeorological model. This method has been successfully applied in a consistent set of dams within Spain.

It is important to point out that, for obtaining results representative up to 10,000 years of return period, the ensemble size should be at least ten times the maximum return period analysed (Loukas et al., 1996; Loukas, 2002; Blazkova and Beven, 2004; Sordo-Ward et al., 2012). The ensembles generated within this thesis have a simple size of 100,000 hydrographs, with the exception of those related to the development of the Hybrid method, due to computational and timing issues (a continuous simulation of 100,000 years would take years of simulation). In order to validate the methodology proposed, a continuous simulation of 5,000 years was performed. Thus, the method shown was validated only up to return periods of 500 years. However, as the Hybrid-event based method reduces the computational times from months to hours, ensembles of 100,000 years could be generated.

3.1.1. A Hybrid stochastic event-based continuous distributed approach

In this thesis, a Hybrid method to derive the flood frequency curve is presented, combining the advantages of both continuous and event-based hydrological models within a distributed physically-based approach.

Distributed physically-based hydrological models appeared in the 1960s and have been the object of critics due to their complexity and difficulty of use (Fatichi et al., 2016). Nowadays, the availability of higher resolution spatio-temporal datasets, the appearance of high performance computers, and the development of parallel computing (Blazkova and Beven, 2004; Cui et al., 2005; Kollet et al., 2010; Vivoni et al., 2011) have opened the possibility of using these models for large size basins and long-term hydrological continuous simulations. Thus, challenges such as the influence of land-use changes (Pumo et al., 2017; Arnone et al., 2018) or the impact of climate change (Liuzzo et al., 2010; Piras et al., 2014) on the involved hydrological processes can be analysed with these models.

Another use of distributed hydrological models is the estimation of extreme floods for the design of civil infrastructures, such as bridges, levees, or dams. For these structures, it is necessary to estimate hydrographs of low probability of occurrence, that is, with return periods that go from hundreds to thousands of years in the case of dams (Rettemeier and Köngeter, 1998; Ren et al., 2017). As indicated, one of the main

disadvantages of physically distributed hydrological models is the computation time. When these models simulate flood events, the computational cost is lower. However, the main disadvantage of event-based models is the uncertainty in the estimation of the humidity in the basin prior to the storm event to be simulated, which is crucial in deriving the corresponding hydrograph. Likewise, the spatio-temporal distribution of storms implicates that the rain with the greatest total depth or intensity of the year does not have to be directly associated with the hydrograph of maximum peak flow or volume of the same year.

These sources of uncertainty can be minimized by continuously simulating the response variables of the basin. However, the possibility of obtaining flows with a high temporal resolution at any point of the drainage network entails a high computational cost. This reason makes it difficult to use it for the derivation of frequency curves of peak flows and volumes for extreme return periods where continuous simulations of thousands of years would be necessary.

This thesis proposes a methodology that combines the advantages of distributed event-based and continuous models, for the derivation of frequency curves of maximum flow and maximum annual flood volume, by combining a stochastic continuous climate generator with a physically-based distributed hydrological model based on both event and continuous simulations.

For the definition of the proposed Hybrid method, the focus was on two critical aspects of event-based simulations:

- Relationship between storm events and floods.
- Initial moisture conditions in the basin prior to storm events.

3.1.1.1. Criterion for storm selection for the Hybrid method.

The focus of this first part of the Hybrid method is, by using a fully physically-based distributed hydrological model coupled with a stochastic weather generator in a continuous simulation, to provide a guideline to select the minimum necessary storm events that should be simulated in event-based approaches to obtain similar results to those obtained by continuous simulations in the derivation of flood frequency curves. [Sordo-Ward et al. \(2016\)](#) carried out a similar analysis in some watersheds in mainland Spain, by using a semi-distributed event-based model coupled with a continuous

stochastic rainfall generator. They separated all the events within the generated series and analysed the relationship between the storms and the maximum annual peak-flow in the Manzanares river basin (Spain) and two of its sub-basins. Flores-Montoya et al. (2016) also carried a similar analysis using a distributed event-based model in two basins in France. However, in both studies, the lack of a continuous simulation required the estimation of the initial soil moisture (same curve number value in Sordo-Ward et al. (2016) and probabilistically in Flores-Montoya et al. (2016)).

Within this thesis, the continuous simulation with a fully physically-based distributed hydrological model enables to account for the variations of the initial soil moisture. Thus, not only can the magnitude and recurrence of events be analysed, but also the relation between storms and the seasonality of the flood hydrographs. Furthermore, within this thesis, the relationship within storms and flood hydrographs is analysed, not only in a univariate way, but with a bivariate approach through the use of copulas for a better assessment between the relation of storms and the dependence of the maximum annual peak-flow and volume.

Therefore, the first objective of the methodology is to propose a criterion for the selection of the minimum number of storm events per year to guarantee a correct derivation of the bivariate flood frequency curve. Figure 3.2 presents a general scheme of the modelling framework and methodology proposed:

Figure 3.2: Criterion for storm selection for the Hybrid method.

- Stochastic weather generation. It consists on the generation of 5,000 years of hourly weather series using the Advanced WEather GENERator (herein AWE-GEN) developed by (Fatichi et al., 2011). In the present study, this model is used to simulate hourly point series of climate from the statistical properties of the observed series in

the case study. AWE-GEN (Fatichi et al., 2011) is a generator capable of reproducing low and high-frequency characteristics of hydro-climatic variables and essential statistical properties of these variables.

The weather generator employs both the physically-based and stochastic approaches and is a substantial evolution of the model presented by Ivanov et al. (2007). AWE-GEN is a statistical hourly stationary model capable of reproducing statistical properties of several weather variables including precipitation, cloud cover, shortwave incoming radiation, air temperature, vapour pressure, wind speed, and atmospheric pressure over a range of time scales. Since rainfall is the main weather variable involved in the framework of this work, a brief introduction of the precipitation model is reported.

The AWE-GEN uses the point Neyman–Scott rectangular pulse approach to generate the internal structure of the precipitation process, based on Cowpertwait (2004) studies. Such models are able to capture the main observed rainfall time-series statistic characteristics; for the case of the AWE-GEN model: (1) mean waiting time between adjacent storm origins (h); (2) mean waiting time for rain cell origins after the storm origin (h); (3) mean duration of the rain cell (h); (4) mean number of cells per storm (-); (5) shape parameter (-) and (6) scale parameter (mm/h) of the Gamma distribution of rainfall intensity. These six parameters are fitted minimizing an imposed objective function using the simplex method (Nelder et al., 1965) on a monthly basis, i.e., the six parameters are inferred for each month in order to account for seasonality. As exposed by Fatichi et al. (2011), to validate the generated series it is recommendable to analyse statistics different from those used in the calibration. Within this thesis, as the focus is on rainfall events and floods, an analysis on how the stochastic generated series were able to reproduce the observed rainfall extreme properties is performed. The reader is referred to Fatichi et al. (2011) and Ivanov et al. (2007) for further details on AWE-GEN.

- Distributed physically-based hydrological modelling. The basin response is modelled by using the TIN-based real-time integrated basin simulator (herein tRIBS (Ivanov et al., 2004a; Vivoni et al., 2007)). tRIBS (Ivanov et al., 2004a; Vivoni et al., 2007) is a physically based, distributed continuous hydrologic model. Its predecessor, the real-time integrated basin simulator (RIBS) of Garrote and Bras (1995) implemented an event-based model for rainfall–runoff analysis. tRIBS has inherited the functionality

of RIBS while adding the hydrology necessary for continuous operation. tRIBS uses an adaptive multiple resolution approach, described by [Vivoni et al. \(2004\)](#), based on triangulated irregular networks (TIN) to represent the terrain topography. tRIBS considers the spatial variability in precipitation fields, land-surface descriptors, and is able to solve the basin hydrologic response at an hourly temporal resolution and a very fine spatial (10–100 m) scale. tRIBS includes parameterizations of rainfall interception, evapotranspiration, infiltration with continuous soil moisture accounting, lateral moisture transfers in the unsaturated and saturated zones, and kinematic-wave runoff routing. The model computational basis, structure, and description of process parameterizations are given in full detail in ([Ivanov et al., 2004a](#); [Vivoni et al., 2007](#)). This model has been implemented in watersheds of various sizes (up to 21,000 km² ([Ko et al., 2019](#))) and in different locations characterized by different climates.

The results of the simulation are 5,000 years of continuous flow at the basin outlet, with a warming period of ten years to reduce the influence of initial conditions in the results obtained.

- Separation of storm events. Analysing the rainfall series within the stochastic weather series generated, the independent storm events are separated. There are different methodologies ([Restrepo-Posada and Eagleson, 1982](#); [Sifalda, 1963](#); [Yen and Chow, 1980](#); [Bonta and Rao, 1988](#)) to identify independent rainfall events.

Rain events are usually identified by fixing the duration of the minimum inter-event time (MIT) that follows, or precedes, a rainfall event. Two rainfall events are independent if there is no rainfall (considering a minimum rainfall threshold of 0.25 mm/h) between them in a period equal or superior to the MIT ([Figure 3.3](#)). Within this study, the exponential method ([Restrepo-Posada and Eagleson, 1982](#)) is selected to determine the value of the MIT. This method assumes that the storm arrival time and the time between two storms follows an exponential distribution. Consequently, the duration of dry periods can be approximated by an exponential distribution with a mean that is equal to the standard deviation, and therefore the coefficient of variation (CV) is equal to one. To obtain the value of the MIT, the CV for different dry-period durations is obtained applying Equation (3.1):

$$CV_k = \text{std}(dp_k) / \overline{dp}_k, \quad (3.1)$$

where k is the dry-period duration (1 h, 2 h, ...), $\overline{dp}_k$ is the average of dry-period durations greater than the duration of k , and $\text{std}(dp_k)$ its standard deviation. The value of the MIT is determined as the value of k in which CV_k approximates to one.

Once the MIT is obtained, all the storm events within the rainfall time-series are obtained. The separation is performed to the observed and generated series for: (a) The validation of AWE-GEN simulations, and (b) for the comparison of rainfall versus floods.

Figure 3.3: Example of two independent rainfall events (black and grey for events one and two, respectively). The total depth of the event (V_{Event}) is represented by the shaded area, the maximum intensity (I_{max}) is represented by the dots, and the duration (D_{Event}) by the lower line. Two dry periods are represented in the figure. The first one, has a shorter duration than the minimum inter-event time (MIT) and therefore, the two wet periods belong to the same event. In the second dry period, the duration is longer than the MIT and it represents the separation between the two events.

Ranking of storm events. Afterwards, each storm is ranked within its year of occurrence following three main criteria: total depth (V_{Event}), maximum intensity (I_{max}), and duration (D_{Event}) (Figure 3.3). The events in the 5,000 years are ranked within the same year from the highest to the lowest according to the three criteria. Therefore, for each criterion, the biggest rainfall event of each year is identified as rank one, the second one as rank two, etc.

Finally, for each criterion, the rank of the storm event (maximum intensity, total depth, and total duration) that generate the maximum peak-flow and maximum flood volume of each year within the 5,000-year time-series is identified.

- Rainfall versus flood comparison. The flood hydrograph related to each storm event is obtained. Afterwards, an analysis of the relationship between the storm rank and (a) the maximum annual peak-flow and maximum annual volume, (b) the seasonality of flood hydrographs, (c) the dependence between the peak-flow and volume, and (d) the bivariate frequency of floods through a copula-based analysis is carried out.

Thus, a minimum number of events per year to be simulated can be proposed for event-based modelling purposes and, therefore, for the application to the Hybrid method.

3.1.1.2. Initial soil moisture conditions in the basin: simulation and probability distributions

Another critical aspect for the development of the Hybrid method is the initial moisture conditions in the basin prior to storm events. It is important to point out that the Hybrid method developed is based on the hypothesis of independence between rainfall events and prior hydrological soil moisture conditions. The validity of this hypothesis, previously assumed by other authors with similar purposes ([Paquet et al., 2013](#)), will be studied in [Chapter 4](#).

1. First, a continuous simulation of 50 years is carried out forcing tRIBS with 50 years of weather forcing of the 5,000-year weather series for the study of the criterion for storm selection. From the simulation, the mean soil moisture in the top 10 cm (MSM10) of the soil layer of the basin at an hourly time step is extracted.
2. Afterwards, from the times series of hourly MSM10, the values associated to all the storm events within the 50 years of hourly weather series are extracted. Therefore, each storm event has an initial soil moisture value associated (MSM10_{Event}).
3. Finally, the probability distribution of MSM10_{Event} for each month of the year is obtained.

By the assumption of the hypothesis of independence between rainfall events and prior hydrological soil moisture conditions, not only MSM10_{Event} values associated to the larger events are accounted, but all MSM10_{Event} values within the series. Therefore, a larger set of initial soil moisture conditions can be obtained with shorter continuous simulations, thus reducing computational costs.

3.1.1.3. The Hybrid method application

Finally, the proposed Hybrid method is presented, based on the results obtained from the study of the criterion of storm selection and the initial soil moisture conditions.

Figure 3.4 presents a general scheme of the modelling framework and methodology of the Hybrid model:

Figure 3.4: General scheme of the Hybrid method.

- First, a continuous set of 5,000 years' hourly weather forcing series is generated using a stochastic weather generator. The chosen stochastic weather generator is AWE-GEN (Advanced Weather Generator (Fatichi et al., 2011)). The time-series used are the same as those generated for analysing the criterion for storm selection.
- Simultaneously, the separation of storm events is performed by applying the exponential method (Restrepo-Posada and Eagleson, 1982) to the 5,000 years climate series. As the weather time series are the same as those generated for analysing the criterion for storm selection, so are the storm events.
- According to the criterion developed in Chapter 4, the five major events of each of the 5,000 years in terms of total depth are simulated in an event-based framework in tRIBS, assigning an initial moisture state value for the basin using a Monte Carlo framework. This initial soil moisture value is obtained from the probability

distribution of initial soil moisture values of all events within that month in the continuous simulation of 50 years.

- Finally, the maximum annual hydrographs are obtained in terms of maximum peak flow and volume, and the associated frequency curves are derived.

To validate the method, the results obtained by the Hybrid method are compared to those obtained by deriving the flood frequency curves from the continuous simulation of 5,000 years, analysing (a) the maximum annual peak-flow and maximum annual volume, (b) the seasonality of flood hydrographs, (c) the dependence between the peak-flow and volume, and (d) the bivariate frequency of floods through a copula-based analysis.

3.1.2. A stochastic data-based hydrological forcing generator

This methodology permits to obtain stochastic inflow hydrographs representative of the observed daily annual floods. As previously exposed, to assure that the results obtained are representative, a set of 100,000 hydrographs is generated.

This method allows, by applying a simple Monte Carlo approach, to generate ensembles of 100,000 maximum annual floods within any location with available daily flow data, accounting for three observed variables: volume, duration and daily peak-inflow. The methodology is as follows (Figure 3.5):

1. Generation of 100,000 hydrograph durations. First step consisted on separating maximum annual flood events from the continuous observed time series (daily time step) by applying the methodology described in Bulletin 17B (USGS,1982). The start of the flood hydrograph is identified as an abrupt rise of the inflows by more than 20%. The end is considered as the point in which the receding limb follows the expression (Equation (3.2)):

$$Q_t = Q_o \cdot e^{-\beta t}, \quad (3.2)$$

where Q_t is the recession flow in m^3/s at any time t after the beginning of the receding limb, Q_o represents the start of the receding limb, and β is a recession constant expressed as the inverse units of time (t). When Equation (3.2) is represented in logarithm scale, the flows that follow the equation represent a straight line, useful for identifying the end of the hydrograph (and applied in this study). Furthermore, the dependence between two consecutive peaks is identified by applying the

independence criteria of daily peak-flows proposed on the Bulletin 17B (USGS,1982). The second peak is not part of the hydrograph if any of the following conditions is not fulfilled (Equation (3.3)):

$$\theta < 5 + \log(A) \text{ or } Q_{\min} > 0.75 \cdot \min(P_1, P_2), \quad (3.3)$$

where θ (days) is the time difference between two peaks, A (mi²) is the area of the basin, $Q_{\min}$ (m³/s) is the minimum flow between the two peaks and P_1 (m³/s) and P_2 (m³/s) are the two peaks compared. Applying this technique, the maximum annual flood (associated to the maximum daily peak-inflow of the year) duration is obtained. Afterwards, the observed probability distribution of flood durations is calculated, and a random sample of 100,000 durations (daily time step) is generated.

2. Generation of 100,000 hydrograph volumes. For the range of the durations identified above, for each duration and observed year, the maximum annual volume is calculated. It is assumed that the maximum annual daily peak-inflow and the maximum annual volume for the different durations are generated by the same flood event (hypothesis previously done by other authors, e.g.: De Michele et al. (2005), Sordo-Ward et al. (2016)). Then, a statistic distribution is fitted to the maximum annual volume for each hydrograph duration considered. Several distributions are considered (LP-III, Gumbel, GEV) being their parameters estimated using L-Moments technique (Hosking, 1990). The distribution chosen is the one that passed Kolmogorov-Smirnov test with the highest P-Value. For each duration of the 100,000 generated durations, a random value of volume is generated following the statistic distribution of the associated duration.
3. Obtainment of 100,000 maximum annual rainfall depth values. Each flood volume value (set of 100,000) is divided by the basin area in order to obtain the cumulated runoff. Then, each cumulated rainfall depth is determined applying a runoff-rainfall transformation based on Curve Number Method (USDA, 1972). The Curve Number, can be obtained from a national study performed by CEDEX (MARM, 2011; Jimenez-Alvarez et al., 2012) that provided Curve Numbers in a 500 by 500 meters grid based on different parameters (soil characteristics, land uses, infiltration data...) within Spain. For basins in other country, the Curve Number can be obtained through traditional procedures (USDA, 1972).

4. Temporal distribution of the 100,000 cumulated rainfall depth values associated to the sample of 100,000 durations. A hydrological event-based model based on [Sordo-Ward et al. \(2012\)](#) is applied. Each cumulated rainfall depth value had a duration associated. The cumulated rainfall depth values are distributed along the duration associated in an hourly time step. This is done by applying an autoregressive moving average (ARMA) (2,2) model ([Sordo-Ward et al., 2012](#)) with an hourly time step. Therefore, 100,000 hourly hyetographs are obtained.
5. Generation of 100,000 reservoir hourly inflow hydrographs. The Curve Number method ([USDA, 1972](#)) is applied to each of the 100,000 hyetographs in order to estimate the corresponding net hyetographs (using the same Curve Number as step 3 to preserve the volume). Finally, the sample of 100,000 hourly hydrographs is obtained applying the Soil Conservation Service dimensionless unit hydrograph procedure ([USDA, 1972](#)) to the set of net hyetographs. The sample of 100,000 hourly hydrographs has volume and duration values that follow the observed probability distributions of both variables. In order to calibrate the model, a local factor of intermittency (torrentiality, F_t) can be applied to the ARMA (2,2) model previously calibrated by [Sordo-Ward et al. \(2012\)](#) by the consideration of hourly rain-gauges close to the basin. The factor of intermittency marks the minimum percentage of event time in which rainfall occurred, discarding unitary hyetographs that do not fulfil this condition. Finally, the model validity is checked by comparing the simulated and observed peak flow frequency curves. In order to compare the peak flow frequency curves, instantaneous peak-inflows based on observed mean daily peak-inflows can be obtained. To do so, Fuller's formula (Equation (3.4)) proposed by CEDEX from a study performed for the entire continental Spain ([MARM, 2011](#); [Jimenez-Alvarez et al., 2012](#)) is applied. They fitted the formula for different homogeneous regions based on measured data:

$$IPF = MDF \cdot (1 + a \cdot A^b), \quad (3.4)$$

where IPF is the instantaneous peak-inflow (m^3/s), MDF is the maximum annual mean daily inflow (m^3/s) and A is the area of the basin in km^2 . The parameters a and b are selected from the mentioned study ([MARM, 2011](#); [Jimenez-Alvarez et al., 2012](#)).

Figure 3.5: Generation of synthetic reservoir inflow hydrographs

3.1.2.1. Seasonal characterization of inflow hydrographs

For the analysis of Maximum Conservation Levels, a seasonal characterization of the hydrological forcing needs to be performed. It should be noticed that this procedure is only applicable in locations with magnitude of floods homogeneous along the year.

Each of the seasons is determined by applying a graphical test to the observed daily inflows proposed by Ouarda (1993) and Ouarda et al. (2006). This test is based on a peak over threshold (POT) analysis. In order to identify the seasons, Ouarda et al. (2006) test different thresholds values while assuring independence between the selected POT. To define the range of thresholds values to be considered, the criteria recommended by Lang et al. (1999) is applied. Furthermore, the criteria proposed by the Water Resources

Council (USWRC, 1976) to assure the independence between two consecutive floods is also applied.

Afterwards, the cumulative empirical probability of POT during the year is plotted against the time of the year, for each tested threshold value. The slope changes within the plot indicate the significant seasons. Then, the probability of occurrence of floods within each season is calculated and, following a Monte Carlo framework, each hydrograph is associated to one of the seasons. Figure 3.6 shows a general scheme of the seasonal characterization of the stochastic floods.

Figure 3.6: Seasonality characterization of hydrological forcings.

3.1.3. A stochastic event-based hydrological forcing generator

A semi-distributed event-based hydrometeorological model was applied to generate the ensemble of flood hydrographs for the study of the Dam in the Segura River Basin, based on the Monte Carlo simulation framework proposed by Sordo-Ward et al. (2012).

The model stochastically generates a set of probability of exceedance values and their corresponding return periods. By applying an extremal distribution SQRT-Et max (Etoh et al., 1986; Ministerio de Fomento, 1999) function, intensity-duration-frequency curves (IDF) (Ministerio de Fomento, 2016), and a stochastic rainfall temporal distribution (Sordo-Ward et al., 2012), a set of hyetographs is generated. Afterwards, the set of storm events are transformed into hydrographs by applying the Curve Number method (USDA, 1972), the Soil Conservation Service dimensionless unit hydrograph procedure (USDA, 1972), and the Muskingum method (McCarthy, 1939).

The reader may refer to Sordo-Ward et al. (2012) for a more detailed description of the processes involved.

3.2. Spillway operation

The lamination and spillway operation main objective is to know the response of the dam-reservoir system to hydrological stresses by calculating the flow regime discharged by the dam downstream as a function of time, as well as the levels reached in the reservoir. This provides the necessary data for calculating the consequences downstream of the dam, and for knowing the probabilities of reaching certain loads (maximum level in the reservoir, possible overtopping, duration of the overtopping situation...), which will serve to quantify the probabilities of failure of the dam.

Two gated real time operational methods are presented. First, the current operational method used by most of Spanish dam practitioners is presented. Secondly, a proposed method based on the improvement of the first one is developed. These two methods are compared to an optimization method (MILP) proposed by [Bianucci et al. \(2013\)](#) and the Inflow-Outflow (I-O) method, which exchanges inflows to the reservoir by discharges.

3.2.1. Volumetric Evaluation Method (VEM)

VEM is a method proposed by [Girón \(1988\)](#) that calculates, in real-time, the outflow during a flood event in reservoirs with gated spillways. [Figure 3.7](#) shows the general scheme of the method implementation.

Figure 3.7: General scheme of the Volumetric Evaluation Method based on [Girón \(1988\)](#).

VEM is based on four principles:

1. Outflows are lower than or equal to the maximum antecedent inflows.
2. Outflows increase when inflows increase.
3. The higher the reservoir level, the higher the percentage of outflow increase.
4. If the reservoir is at maximum capacity, outflows are equal to inflows while gates are partially opened.

The VEM progressively manages the available S_i^F , increasing the outflows as the S_i^F decreases. Given the reservoir inflow at time i (I_i), and by assuming a constant inflow in the future, the number of intervals left (n) until the reservoir runs out of S_i^F are given by Equation (3.5):

$$n = \frac{S_i^F}{(I_i - O_{i-1}) \cdot \Delta t}, \quad (3.5)$$

where O_{i-1} is the outflow at time $i - 1$ and Δt represents the operation time step. In this study, a time step of one hour is adopted. Considering that outflows must be equal to inflows when $S_i^F = 0$ and that outflows increase linearly until equaling the inflows at time $\Delta t \cdot n$, and by assuming a constant inflow in the future, the increment of outflows ($\Delta O_i = O_i - O_{i-1}$) at each time step is $\frac{(I_i - O_{i-1})}{n}$, and can be expressed by Equation (3.6) (replacing n by Equation (3.5)):

$$\Delta O_i = \frac{(I_i - O_{i-1})}{n} = \frac{(I_i - O_{i-1})^2 \cdot \Delta t}{S_i^F} = \frac{\Delta S_i^2}{S_i^F \cdot \Delta t}, \quad (3.6)$$

Equation (3.6) is valid when the time intervals (n) until the reservoir runs out of flood control capacity are higher than one ($S_i^F > \Delta S_i$). When $n \leq 1$, the method aims to balance the outflows and the inflows. When $S_i^F = 0$, the gates are in operation, maintaining the reservoir level at FCL until they are fully open. Although both antecedent assumptions are not real, by selecting a short time operation step and by recalculating the decision process at each time step, possible deviations are minimized. ΔO_i can be expressed as:

$$\Delta O_i = I_i - O_{i-1}. \quad (3.7)$$

The proposed outflow (Q_p) is determined, taking into account S_i and whether the reservoir volume is increasing ($\Delta S_i = S_i - S_{i-1} \geq 0$) or decreasing ($\Delta S_i < 0$) (Figure 3.7). The different outflows are proposed as follows:

1. If $S_i \leq S_{TCP}$, the method aims to increase the reservoir level to reach the maximum normal operative level (TCP):

$$Q_p = 0. \quad (3.8)$$

2. When $S_i > S_{TCP}$, the outflow proposed depends on ΔS_i :

If $\Delta S_i \geq 0$, the method releases outflows according to the ΔO previously defined by VEM, as follows:

- If $S_i^F \leq \Delta S_i$, the time intervals (n) until the reservoir runs out of flood control capacity are equal to or lower than one, and the method aims to balance the outflows and inflows:

$$Q_p = O_{i-1} + \frac{\Delta S_i}{\Delta t}. \quad (3.9)$$

- On the other hand, if $S_i^F > \Delta S_i$, the VEM progressively manages the available S_i^F , increasing the outflows as the S_i^F decreases:

$$Q_p = O_{i-1} + \frac{\Delta S_i^2}{S_i^F \cdot \Delta t}. \quad (3.10)$$

If $\Delta S_i < 0$ (and $S_i > S_{TCP}$), the VEM aims to continue decreasing the reservoir level until reaching the TCP:

$$Q_p = O_{i-1} + 0.5 \cdot (I_i - I_{i-1}). \quad (3.11)$$

Once Q_p is obtained, it is compared to the maximum discharge capacity at the current reservoir level ($O_{\max, \text{disch.}}(S_i)$) and the maximum of the antecedent inflows ($I_1, I_2, \dots, I_i$), and the minimum of the three values is the flow selected to be released through the gates (O_i).

3.2.2. The proposed K-Method

As previously exposed in Section 3.2.1., the main advantage of the VEM is its simplicity. However, it is a fixed method, in the sense that it cannot be adapted to the specific conditions of the basin, the reservoir, or the spillway, other than the flood control

volume. The K-Method, based on the VEM (Girón, 1988), proposes the application of a corrective parameter K to the expressions that govern the dam operation rules while the reservoir level is rising. By varying parameter K, the proportion in which the outflows are increased is changed, until the FCL is reached. In addition, several improvements to the VEM are included, considering: (1) more reservoir zones to avoid abrupt changes of outflows; (2) an alert outflow (O_{ALT}) as the maximum admissible flow for avoiding downstream damage; and (3) a maximum gate opening/closing gradient ($O_{max.Gr}$). Figure 3.8 shows the general scheme of the K-Method implementation.

Figure 3.8: General scheme of the K-Method.

The method defines three characteristic levels and reservoir volumes, the aforementioned TCP and FCL, and incorporates the activation level into the scheme (AL, the corresponding volume is S_{AL}). The activation level is the reservoir level in which the rules of operation change, increasing the outflows. The aforementioned characteristic levels determine four zones: 1, 2, 3, and 4 (Figure 3.8). The proposed outflow (Q_p) is determined as a function of S_i , the corresponding zone (1, 2, 3, or 4), and whether the reservoir storage is increasing ($\Delta S_i = S_i - S_{i-1} \geq 0$) or decreasing ($\Delta S_i < 0$).

3.2.2.1. Proposed outflow in zone 1

If the reservoir level is within Zone 1, the method aims to increase the reservoir level to reach the maximum normal operative level (TCP):

$$Q_p = 0. \quad (3.12)$$

3.2.2.2. Proposed outflow in zone 2

If the reservoir level reaches Zone 2 and it is rising ($\Delta S_i \geq 0$), the method gradually starts to release flow, avoiding abrupt changes of outflows that could be dangerous downstream the dam. The proposed outflow is as follows:

1. If $S_i^F > \Delta S_i$, releases increase linearly from 0 m³/s (when the reservoir level is equal to TCP) to the proposed outflow in Zone 3 at AL:

$$Q_p = \left(O_{i-1} + K \cdot \frac{\Delta S_i^2}{S_i^F \cdot \Delta t} \right) \cdot \frac{S_i - S_{TCP}}{S_{AL} - S_{TCP}}. \quad (3.13)$$

2. If $S_i^F \leq \Delta S_i$, the method aims to balance the outflows and inflows:

$$Q_p = O_{i-1} + \frac{\Delta S_i}{\Delta t}. \quad (3.14)$$

If the reservoir level is within Zone 2 and falling ($\Delta S_i < 0$), the method aims to continue gradually decreasing the reservoir level, avoiding abrupt changes of outflows and minimizing downstream floods:

3. If $I_i < O_{ALT}$, the method releases the maximum precedent outflow limited by the alert outflow:

$$Q_p = \min(O_{ALT}, \max(O_1, O_2, \dots, O_{i-1})) \quad (3.15)$$

4. Otherwise, the method maintains the reservoir level, avoiding an increment of downstream floods:

$$Q_p = I_i. \quad (3.16)$$

3.2.2.3. Proposed outflow in zone 3

If the reservoir level is within Zone 3 and rising ($\Delta S_i \geq 0$):

1. If $S_i^F > \Delta S_i$, the method progressively manages the available S_i^F , increasing the outflows as the S_i^F decreases:

$$Q_p = O_{i-1} + K \cdot \frac{\Delta S_i^2}{S_i^F \cdot \Delta t}. \quad (3.17)$$

2. Otherwise, if $S_i^F \leq \Delta S_i$, the method aims to balance the outflows and inflows:

$$Q_p = O_{i-1} + \frac{\Delta S_i}{\Delta t}. \quad (3.18)$$

If the reservoir level is within Zone 3 and falling ($\Delta S_i < 0$), the K-Method decreases the reservoir level at a linear rate, from the maximum antecedent outflow to the outflow proposed at AL (limit of Zones 2 and 3):

3. If $I_i < O_{ALT}$, the method releases more than the maximum precedent outflow limited by the alert outflow:

$$Q_p = Q_{Eq.(3.15)} + (\max(O_1, O_2, \dots, O_{i-1}) - Q_{Eq.(3.15)}) \cdot \frac{(S_i - S_{AL})}{(\max(S_1, S_2, \dots, S_{i-1}) - S_{AL})}, \quad (3.19)$$

$Q_{Eq.(3.15)}$ is the proposed outflow in Equation (3.15).

4. Otherwise, the method decreases the reservoir level, minimizing downstream floods:

$$Q_p = I_i + (\max(O_1, O_2, \dots, O_{i-1}) - I_i) \cdot \frac{(S_i - S_{AL})}{(\max(S_1, S_2, \dots, S_{i-1}) - S_{AL})}. \quad (3.20)$$

3.2.2.4. Proposed outflow in zone 4

If the reservoir level is within Zone 4 and rising ($\Delta S_i \geq 0$), the gates are operated, maintaining the reservoir level at FCL until they are fully open:

$$Q_p = O_{i-1} + \frac{\Delta S_i}{\Delta t}, \quad (3.21)$$

Conversely, if the reservoir level is within Zone 4 and falling ($\Delta S_i < 0$), the K-Method aims to decrease the reservoir level as soon as possible. The proposed outflow is the one defined during the last interval:

$$Q_p = O_{i-1}. \quad (3.22)$$

3.2.2.5. Determination of the released outflow at each time step

Once Q_p is obtained, it is compared to the maximum discharge capacity at the current reservoir level ($O_{\max, \text{disch.}}(S_i)$), the maximum of the previous inflows ($I_1, I_2, \dots, I_i$), and the maximum gate opening/closing gradient ($O_{\max, \text{Gr.}}(S_i)$), and the minimum of the four values is selected as the flow to be released through the gates (O_i).

3.2.3. I-O and MILP

Finally, the I-O and MILP methods are presented as extreme performance cases. The I-O exchanges inflows to the reservoir by discharges, so no attenuation effect of the reservoir is considered while the gates are partially opened. I-O corresponds to the extreme case of VEM, where no control flood volume is considered ($TCP = AL = FCL$). In this case, only Zones 1 and 4 are operative.

The MILP is applied to obtain optimal operation rules in terms of the maximum outflow and maximum level reached in the reservoir for each flood event. MILP was proposed by [Bianucci et al. \(2013\)](#), based on the mixed integer linear programming theory, minimizing an objective cost (penalty) function considering the hydraulic and operational restrictions. This objective function (P) is based on the weighted sum of two penalty functions, the P_s associated with the reservoir volumes (and affecting the dam safety) and the P_o associated with the outflows (and affecting the downstream safety), as shown in Equation (3.23):

$$P = w_o \cdot P_o + w_s \cdot P_s, \quad (3.23)$$

where w_o and w_s represent the weight associated with P_o and P_s respectively. The reader may refer to [Bianucci et al. \(2013\)](#) for a detailed description of the MILP model. It should be stressed that, unlike the three other methods, the MILP method requires knowledge in advance of the entire inflow hydrographs, and therefore, it corresponds to an unrealistic situation.

3.3. Initial reservoir level analysis

The distribution of the water storage in the reservoir, and thus of the initial reservoir level, depends basically on the inflows, the demands, the reservoir management rules, and the water losses (evaporation, infiltration, etc.). Within this section, two approaches for assessing initial reservoir level are presented. One is based on the use of the historical

reservoir volumes registered in the reservoir, whereas the second one is based on the simulation of the regular operation of the reservoir.

Either by using the register of historic reservoir levels in the dam-reservoir system or by simulating the operation of the reservoir, the initial reservoir level can be analysed obtaining the distribution of empirical levels. This distribution is then used within the Monte Carlo framework of the stochastic methodology proposed, associating an initial reservoir level to each inflow hydrograph.

In [Chapter 6](#), the impact of initial reservoir level and its uncertainty is assessed through the use of historical reservoir levels data. Within [Chapter 7](#) and [Chapter 8](#), the assessment of initial reservoir level is performed by simulating the regular operation of the reservoir.

3.3.1. Use of historic reservoir levels

To obtain a relationship between probability and levels in the reservoir, the historical levels in the reservoir can be used. To do this, it is necessary to have a sufficiently long record that is representative of the current operating situation.

The data must satisfy certain conditions in order to be valid for the hydrological dam safety assessment.

Firstly, the data must be consistent and reliable, so the series must be filtered to eliminate erroneous data. To be able to use these data, it is necessary that the data correctly represent the distribution of levels in the reservoir. To this end, the length of the series of data must be sufficient to correctly represent the variability of the levels. The literature states that a length of 25 years is sufficient ([SPANCOLD, 2012](#)). In any case, it is important to study the influence of the length of the data series on the shape of the probability curve obtained and to ensure that the length taken is sufficient, since the length of the data period may influence the results ([SPANCOLD, 2012](#)).

In addition, to avoid introducing into the probability curve data which are not representative of the normal situation of the reservoir, data corresponding to exceptional situations such as the process of filling the reservoir, emptying the reservoir for repair works or other situations which are not usual in the normal operation of the reservoir.

In order to obtain a correct characterization of the current situation of the levels in the reservoir, it is also important to eliminate the data prior to the occurrence of any event

that considerably modifies the variation in levels, such as the construction of a dam upstream or a re-growth of the dam. Furthermore, it is necessary to eliminate flood situations in the record, since the level rise caused by floods is introduced independently through the flood operation simulation of the ensemble of hydrographs.

However, the data corresponding to other unusual situations in the reservoir, such as periods of drought, should be included in the analysis of the historical level records as floods can occur on these situations.

If the level records in the reservoir do not meet the aforementioned conditions or if a possible future situation (such as the definition of different maximum conservation levels, as in [Chapter 8](#)), it is necessary to perform simulations to obtain the necessary initial reservoir level data.

3.3.2. Simulation of reservoir levels

An alternative to the use of historical records to obtain the probability distribution of initial reservoir levels is to carry out a simulation of the water resources dam-reservoir system. The aim of a simulation model is to obtain the distribution functions of a variable by analysing its behaviour. In this case, the simulation reproduces the operation of the reservoir according to the current or proposed management rules of the system.

In general, the simulation of the operation of the reservoir is carried out within a more complex simulation of the planning of the complete water resource system, which may include several reservoirs. These models consist of a sequential calculation of the location and use of the water resources. For this purpose, the mass balance equation is used, taking into account the physical restrictions of the water and the rules of management of the infrastructures.

There are several programs developed for basin planning such as AQUATOOL-DMA ([Andreu et al., 1996](#)) or HEC-ResSim ([Klipsch and Hurst, 2007](#)) that can facilitate this type of calculation. To perform these simulations is necessary to have a set of observed reservoir inflows time series, monthly demands, reservoir management policies and evaporation rates and ecological flows.

Event though the amount of information and work required result into a more laborious task than using historical reservoir levels, the simulation framework presents several advantages such as:

- It allows to have a wide register of reservoir levels. This advantage is used in [Chapter 7](#), as the historical data set is increased by the simulation of reservoir levels.
- The levels are representative of the operation of the reservoir. For the optimization of Maximum Conservation Levels, as performed in [Chapter 8](#), the only way of accounting for the multiple scenarios is through simulation of the regular operation of the reservoir.
- It allows alternatives to be considered in the safety of the dam and in the planning or management of the water resources system.

3.4. Spillway reliability

The reliability of spillways is of great importance to the safety of dams, and has been a key part of many catastrophic failures as exposed in the literature review. Despite its obvious importance, traditionally the reliability of the spillway outlets has been an aspect that has been difficult to integrate into the overall safety of a dam and has been taken into account in isolation. Through the methodology proposed in this section, spillway reliability can be integrated into the hydrological dam safety analysis, being its impact on safety quantifiable as will be shown in [Chapter 7](#).

The effect on the safety of the dam of the reliability of the spillways is indirect: low reliability results into a greater probability of reaching high (or even overtopping) reservoir levels during a flood, which increases the probability of dam failure.

Therefore, the information that must be included is the probability that each outlet can be used for this purpose, i.e. that at the time of a flood each dam outlet may or may not be used. It is usual and usually sufficient to make the assumption that each dam outlet (each span of a spillway, each pipe of an outlet) may be fully functional or not functional at all. A study could also be carried out to contemplate intermediate scenarios of partial operation, although it would be necessary to assess whether the improvements in terms of the precision of the results would be relevant.

The reliability of the dam outlets should not be confused with the possibility of their sudden collapse or opening causing an artificial flood downstream. This aspect should also be analysed, but not as a component of the system loads but as a potential failure mode (out of the scope of this thesis). In summary, this section does not address the

possibility of floodgates opening when they are not wanted, but rather the possibility that they will not open when they are.

Figure 3.9: Scheme of the fault tree proposed.

To consider the possibility of gate failure, in this thesis a combination of fault trees with gate functionality is proposed. The fault tree is designed combining different rates of gate failure based on expert judgment (Figure 3.9).

Two different failure types are considered: independent cause failure affecting a single gate and common cause failure affecting a group of gates. Each node of the fault tree represented an event that could happen or not, that is, a binary event (Kuo et al, 2008).

Once the fault-tree is designed, the process consists on generating as many gate functionality cases as hydrographs are routed within the Monte Carlo framework.

3.5. Hydrological dam and downstream Safety

Once the ensemble of flood hydrographs (100,000 hydrographs within this thesis) is routed through the reservoir-dam system, an ensemble of time series of reservoir levels and outflows is obtained (Figure 3.10). From each time series of the ensemble, the maximum outflows and maximum reservoir levels are extracted, obtaining a set of 100,000 values each.

These series allow the calculation of the non-parametric frequency distribution of the annual maximums of the different variables considered (maximum outflow and maximum reservoir level) by applying the plotting position formula of Gringorten (1963)

and the relationship between the exceedance probability and the return period (Equation (3.24)):

$$\frac{1}{T_{ri}} = p_i = \frac{i-0.44}{n+0.12} \quad (3.24)$$

where n is the number of data, i is the position that the data occupies when the data is ranked from lowest to highest, p_i is the estimate of the cumulative frequency and T_{ri} is the return period of the data found in position i of the sorted data list.

Figure 3.10: Analysis of hydrological dam and downstream safety through maximum outflows and reservoir levels frequency curves

The frequency distribution of maximum reservoir levels permits to analyse the hydrological dam safety, analysing the probability of reaching certain reservoir levels under flood operation conditions. In the case of the frequency distribution of maximum outflows, the probability of reaching certain downstream flow that could overpassed the riverbed can be analysed, focusing on the downstream river safety.

3.6. Economic risk analysis

In order to transfer the concepts of hydrological dam and downstream safety into a risk analysis context, the global risk indexed proposed by [Bianucci et al. \(2013\)](#) is used in this thesis.

This method accounts for a single indicator of the global risk associated for the maximum reservoir levels and maximum outflows. To do so, it relies on the concept of expected annual damage (EAD) ([Arnell, 1989](#)). EAD is frequently used for quantifying the damage associated to floods ([Bianucci et al., 2013](#)). It represents the area under the damage-probability curve, which is calculated as follows (Equation (3.25)):

$$EAD = \sum_{i=1}^{M-1} (p_i - p_{i+1}) \cdot \frac{(D_i + D_{i+1})}{2} \quad (3.25)$$

where M represents the number of points of probability-damage considered in the curve, p represents the exceedance probability and D the damage associated.

The EAD concept is applied within this thesis to obtain two different indexes: one related to the failure of the dam due to overtopping (I_F) (Equation (3.26)), and the second one associated to the damage (without dam failure) caused by the resulting outflows from the dam flood control operation (I_{NF}) (Equation (3.27)).

$$I_F = \sum_{i=1}^{N-1} (p(MWRL_i) - p(MWRL_{i+1})) \cdot \left(D_F \left(\frac{MWRL_i + MWRL_{i+1}}{2} \right) \right), \quad (3.26)$$

$$I_{NF} = \sum_{i=1}^{N-1} (p(MO_i) - p(MO_{i+1})) \cdot \left(D_{NF} \left(\frac{MO_i + MO_{i+1}}{2} \right) \right), \quad (3.27)$$

where D_F and D_{NF} are the damage functions (expressed in monetary terms, euros within this thesis) for failure and non-failure respectively, the index i represents the position of the ranked series (ascending order up to N , which is equal to 100,000 in this thesis) of maximum reservoir levels (MWRL, also referred as Z_{MAX} in this thesis) and maximum outflows (MO, also referred as O_{MAX} in this thesis) and p represents the exceedance probability.

Once the two indexes are obtained, the global risk index (I_R) is calculated as the aggregation of both indexes (Bianucci et al., 2013) (Equation (3.28)):

$$I_R = I_F + I_{NF}, \quad (3.28)$$

where I_F , I_{NF} and I_R are expressed in euros. To apply the global risk index methodology, it is necessary to obtain the damage cost curves (D_{MWRL} and D_{MO}). To do so, the procedure is as follows:

1. Damage cost curve associated to overtopping dam failure. It is important to highlight the difference between the risk of damage associated to MOs and to MWRLs. Whereas in the case of MOs there is an actual damage associated to a determined flow, reaching a certain MWRL might not have a damage associated (Bianucci et al., 2013). Thus, reaching a certain reservoir level does not have to result into the dam

failing, but a risk of failing. Therefore, following the approach proposed by Bianucci et al. [Bianucci et al. \(2013\)](#), the damage cost curve is calculated as (Equation (3.29)):

$$D_{MWRL}(MWRL_i) = p(\text{break}|MWRL_i) \cdot \text{Cost}_{\text{break}}, \quad (3.29)$$

where $\text{Cost}_{\text{break}}$ represents the damage cost if the dam fails (expressed in euros), whereas $p(\text{break}|MWRL_i)$ is the probability of overtopping failure conditioned to reaching a certain level in the reservoir ($MWRL_i$). $\text{Cost}_{\text{break}}$ is estimated as the cost of reconstruction of the dam aggregated to the estimated downstream damage when overtopping failure occurs. In this thesis, $\text{Cost}_{\text{break}}$ is considered as a constant value, as there was only data of the estimated downstream damage associated to the flood wave caused by the overtopping failure mode accounted in the Dam Master Plans (failure when reservoir level is at dam crest level (COD)). Regarding to $p(\text{break}|MWRL_i)$, according to [Bianucci et al. \(2013\)](#) can be represented as the probability of reaching the reservoir level at COD once $MWRL_i$ has been reached during a flood event (Equation (3.30)):

$$p(\text{break}|MWRL_i) = p(MWRL_{\text{MAX}} \geq \text{COD}|MWRL_i) = \frac{n^{\circ} \text{ events } MWRL_{\text{MAX}} \geq \text{COD}}{n^{\circ} \text{ events } MWRL_{\text{MAX}} \geq MWRL_i}. \quad (3.30)$$

This expression is estimated by routing 100,000 synthetic floods with a moderate to extreme return period of peak-flow (10 to 200,000 years). By assuming that overtopping leads to dam failure, $p(\text{break}|MWRL_i)$ is calculated as the number of events in which $MWRL_{\text{MAX}} \geq \text{COD}$ over the number of events in which $MWRL_{\text{MAX}} \geq MWRL_i$.

2. Damage cost curves associated to non-failure. Regarding to D_{MO} , it is estimated by analysing the possible damages downstream. As the dams studied within this thesis are in Spain, the damage cost curve associated to non-failure was estimated based on the information included in the Spanish National GIS viewer about flood risk assessments through flood plain analysis ([MAPA, 2019](#)).

3.7. Stochastic procedure for the definition of Maximum Conservation Levels

Maximum Conservation Levels (MCLs) represent the linking variable between flood control operation and water conservation operation of the reservoir ([Figure 3.11](#)).

To obtain the optimal set of seasonal MCLs accounting for both hydrological safety (dam and downstream) and water supply with a specific reliability, a stochastic methodology is proposed, graphically summarized in Figure 3.11.

Figure 3.11: General Scheme of the methodology proposed.

3.7.1. Study set of seasonal Maximum Conservation Levels

The number of possible configurations of MCLs vary according to the number of seasons (n) defined and the possible Maximum Conservation Levels selected for a proper discretization to analyse all the outcomes accounting for computation time (k). As the position of the MCLs matters (it is not the same to have the same MCL in one season or another), the number of configurations were given by a variation with repetition ($VR_k^n = k^n$). To define the seasons, the procedure exposed in Section 3.1.2.1 is used in this thesis.

3.7.2. Reservoir operation simulations

For each configuration of seasonal MCLs of the study set, two different reservoir operation simulations were carried out: simulation of flood control operation and simulation of water conservation (regular) operation.

3.7.2.1. Simulation of flood control operation

For each configuration of MCLs, the maximum water level in the reservoir corresponding to a return period of 1,000 years ($MWRL_{TR=1000y}$) and the maximum outflow corresponding to a return period of 500 years ($MO_{TR=500y}$) were obtained, corresponding to the set of 100,000 seasonal maximum annual inflow hydrographs generated following the procedure of Section 3.1.2, and routed by applying the VEM (Section 3.2.1). The analysis was carried out in two different scenarios of the initial reservoir level prior the flood event (Z_0) (Figure 3.11).

- Scenario 1 (Sc.1): Z_o is constant and corresponds to the seasonal MCL. The reservoir is assumed to be at its maximum operating level when the reservoir gets the maximum annual flood.
- Scenario (Sc.2): Z_o is variable, and the reservoir can be at any level when the maximum annual flood occurs. Z_o is obtained (within a Monte Carlo framework) from the cumulative probability distribution of Z_o associated to the season of occurrence of the maximum annual flood event. This distribution is obtained from the simulation of the water conservation operation of the reservoir ([Section 3.3.2](#)).

3.7.2.2. Simulation of water conservation operation

The yield reliability (Y_R) was considered as the ratio of total volume supplied and the total volume demanded ([Chavez-Jimenez et al., 2013](#); [Sordo-Ward et al., 2019a, b](#)) during the period analysed. For each configuration of MCLs (k), the reservoir yield with reliability of 90% ($Y_{R=90\%}$) and the cumulative probability distribution of Z_o are obtained, by developing a monthly water balance model. The model manages the dam as an isolated element. The water balance considers time series of monthly inflows, environmental flow restrictions, evaporation rates, monthly demand distribution, storage-area-height reservoir curves and dead storage volume (data extracted from “Duero National Water Master Plan” ([CHD, 2015](#))). The main purpose of the reservoir is irrigation and therefore a required yield reliability of 90 % ($Y_{R=90\%}$) was adopted, adequate for irrigation demands in the region ([Garrote et al., 2015](#)).

To identify the maximum amount of water that can be supplied to satisfy a regular demand with a specified reliability, a bipartition method was applied: excessive values of demands were set (for example, similar to mean monthly runoff) and the simulation was carried out. The deficits were obtained, and specified yield reliability requirements were checked. If the specified reliability requirements were not fulfilled, the demand was reduced by half and simulated again. If the specified reliability requirements were satisfied, half of the difference was added and simulated again, and so on until the deficit (or gain) was smaller than a pre-set tolerance (e.g., $0.1 \text{ hm}^3/\text{year}$). In addition, the operation of the reservoir was simulated with the associated mean annual current demand. The procedure was repeated for both Sc.1 and Sc.2 scenarios.

3.7.3. Results analysis and solutions proposal

In order to propose the optimal configurations of MCLs within the case study, as exposed, the main decision variables selected were $MWRL_{TR=1000y}$, $MO_{TR=500y}$ and $Y_{R=90\%}$. It was assumed that the MCLs configuration would not fulfil the standards if $MWRL_{TR=1000y}$ was above the DFL and/or $MO_{TR=500y}$ was greater than the emergency flow ($O_{EMER.}$). $MWRL_{TR=1000y}$, $MO_{TR=500y}$ and $Y_{R=90\%}$ were compared, for all the configurations in the study set of MCLs (Figure 3.11), by identifying configurations that are non-inferior solutions (Pareto framework) in terms of maximum volume of water supplied (with a specific yield reliability of 90%) and hydrological safety.

3.7.3.1. Determination of possible MCLs by applying a Pareto analysis

The selected variables for conducting the Pareto analysis were $MO_{TR=500y}$ and $Y_{R=90\%}$ (Figure 2). In this thesis, it was assumed that higher levels in the reservoir (and above MNL) imply greater outflows. This assumption is always fulfilled either for dams with fixed crest spillways or if the Volumetric Evaluation Method is applied in gated spillways (as in this study). It should be noted that this assumption may not hold for all possible specific characteristics of dams, all rules of operation adopted, and all specific values adopted for the hydrological dam and downstream safety. Moreover, in this study, the outflow corresponding to the DFL condition is higher than that corresponding to $O_{EMER.}$, that is, the $MO_{TR=500y}$ is the most restrictive variable. Therefore, for each scenario (Sc.1 or Sc.2), there was a set of k^n MCLs configurations with k^n pairs of values $MO_{TR=500y}$ and $Y_{R=90\%}$. The purpose was to obtain the configurations of MCLs that minimize the value of $MO_{TR=500y}$ while maximizing $Y_{R=90\%}$, which implies a two-objective minimization problem (Equation (3.31)):

$$\text{Min}\{f_1(x_i), f_2(x_i)\} \quad (3.31)$$

in which $x_i = [MCL_{S1}, MCL_{S2}, \dots, MCL_{Sn}]$, $f_1(x_i) = [MO_{TR=500y_i}]$ and $f_2(x_i) = [-Y_{R=90\%_i}]$; being $i=1, 2, \dots, k^n$. Once the non-dominated solutions were identified, for Sc.1 and Sc.2, the non-dominated solutions from both scenarios those that did not fulfil the hydrological safety regulation standards (both for $MWRL_{TR=1000y}$ and/or $MO_{TR=500y}$) and those providing a $Y_{R=90\%}$ lower than the annual demand that is currently satisfied were eliminated. Afterwards, a comparison of the proposed solutions was carried out, providing dam stakeholders with a set of possible configurations depending on their main

objectives: increasing hydrological dam and downstream river safety or increasing the water supply with a specific reliability within the system.

Figure 3.12: Scheme of the procedure for solutions proposal.

3.8. Introduction to the case studies and software used

The methodology proposed for each part of the present research is applied different case studies (Figure 3.13). A total of four case studies are considered: three of them are based in reservoir-dam configurations in Spain, whereas the other one is a watershed in the United States. The watershed case study is analysed to develop the Hybrid method explained in Section 3.1.1. The different reservoir-dam configurations are selected to show the applicability of the proposed stochastic methodology regardless the dam characteristics (volume of the reservoir, purpose, etc.) or its location (climate, size of the reservoir basin, etc.). Each case study is briefly summarized below. The data used and the specific details of each case study are developed in its corresponding chapters:

(a)

(b)

Figure 3.13: Location of the case studies of the research (figure generated with Google Earth).(a) Dam case studies (b) Peacheater Creek.

- Dam in Tagus River Basin: based on a gated-spillway gravity dam located in the Tagus river basin. The dam is located in the west of Spain (province of Caceres). The dam watershed has an extension of 1,850 km². The climate within the area of study is Continental with mean annual precipitation of 1,000 mm. The main purpose of the reservoir is irrigation.

The volume of the reservoir at its maximum normal level is 911 hm³. The dam height is 73 meters.

Figure 3.14: Plans and picture of the dam-reservoir system of the Tagus river basin. a) section, b) plant c) view. Source: [MAPA, 2019](#)

- **Dam in Segura River Basin:** This case study is based on a gated-spillway dam located in the province of Albacete, in the southeast of Spain, which belongs to the Mundo river basin. The basin has an area of 766.5 km². The climate of the region is Mediterranean (mean annual precipitation of 557 mm). The main purposes of the reservoir are flood regulation, hydropower generation, and water supply for the Region of Murcia.

The volume of the reservoir at its maximum normal level is 35 hm³. The dam height is 46 meters.

Figure 3.15: Plans and picture of the dam-reservoir system of the Segura river basin. a) section, b) plant c) view. Source: [MAPA, 2019](#)

- Dam in Douro River Basin. This case study is based on a gated-spillway dam located in the headquarters of Esla river, a tributary of the river Douro. The dam is located in the northwest of Spain (province of Leon). The dam river basin has an extension of 582 km². The area has a continental climate characterized by flood events of several days of continuous rainfall of medium intensity (mainly generated by frontal systems). The mean annual precipitation within the basin is 1,650 mm. The main purpose of the reservoir is irrigation and hydropower generation.

The volume of the reservoir at its maximum normal level is 651 hm³. The dam height is 100 meters.

Figure 3.16: Plans and picture of the dam-reservoir system of the Douro river basin. a) section, b) plant view. Source: [MAPA, 2019](#)

- **Peacheater Creek:** This case study is a basin, located in the Oklahoma State, in the United States. This watershed is a subbasin of the Baron Fork basin. The Peacheater Creek basin has a drainage area of 64 km². It was selected since it was originally used to test, calibrate, and validate the tRIBS model (Ivanov et al., 2004a), model selected and explained in Section 3.1.1. Further details on the basin and the model implementation are given in Chapter 4.

All the experiments were developed under the environment of the Matlab programming platform. Its matrix calculation concept allows handling large volumes of data (both in its generation and subsequent process and analysis) with reasonable calculation time costs. It also adapts well to the execution of systematic and intensive calculation processes, and it has a powerful graphic interface that facilitates the analysis and post-processing of results (The MathWorks, 2018).

Another important aspect to take into account in this type of studies where the volumes of data generated are important, is the design of a database associated with the calculation processes, which should be simple, robust and allow the traceability of each of the cases analysed. In this sense, Matlab environment is flexible and allows for automation and correct interface between the calculation processes and the databases (both with the model input data, the data generated and the post-process data).

Furthermore, in some parts of this research, intensive repetitive calculations where necessary under the Monte-Carlo framework performed. To perform intensive tasks, such as those shown in Chapter 4, Chapter 6 and Chapter 8, the use of a High Performance Computer reduced the computational times.

The cluster used was Magerit, a High Performance Computer (HPC) owned by Universidad Politécnica de Madrid. The experiments were carried out using the Intel family processors architecture, which consists of 41 nodes with two intel processors Intel XEON E5-2670 of eight cores each, and 64 GB RAM/Core. Thus, multiple independent simulations are carried out at the same time by using multiple processors in Chapter 6 and Chapter 8.

Regarding to the distributed hydrological model (tRIBS) used in Chapter 4, the code of the model is programmed in C++ (Ivanov et al., 2004a). Matlab within this case is used as a pre-processing and post processing tool, whereas Magerit capabilities are used to

perform simulations accounting for parallel computation techniques as will be detailed in [Chapter 4](#).

The following table shows the relation of the chapters with the methodology ([Table 3.1](#)).

Table 3.1: Relationship between chapters, methodological key aspects, case studies, software used and role of the High Performance Computer (HPC).

Chapter	Key aspects	Case study	Software used	Role of HPC
4	Hybrid method	Peachewater Creek	Matlab	Parallel Computing
			tRIBS	
			AWE-GEN	Intensive experiment repetition
5	K-Method Global risk index	Segura River Dam	Matlab	Not used
6	Uncertainty associated to initial reservoir level Global risk index	Tagus River Dam	Matlab	Intensive experiment repetition
7	Initial reservoir level Spillway reliability	Douro River Dam	Matlab	Not used
8	Maximum Conservation Levels		Matlab	Intensive experiment repetition

4. Hydrological forcing: The Hybrid method

This first part of this chapter focuses on proposing the minimum number of storms necessary to derive the extreme flood hydrographs accurately through event-based modelling, by using a distributed physically-based model. To do so, the methodology exposed in [Section 3.1.1.1](#) is applied. Afterwards, initial soil moisture conditions are analysed in order to develop the Hybrid method ([Section 3.1.1.2](#)), which combines the advantages of continuous and event-based hydrological models. Finally, the Hybrid method is applied, analysing its performance ([Section 3.1.1.3](#)).

4.1. Application to Peacheater Creek basin

4.1.1. Study basin

The Hybrid method was applied to Peacheater Creek (herein, PC), using the hourly climate data of Westville, a weather station located within the basin ([Figure 4.1a](#)).

Figure 4.1: Case study and model setup. (a) Basin location, elevation, and drainage network. (b) Discretization of the basin into Voronoi polygons for triangulated irregular network real-time integrated basin simulator (tRIBS). (c) Partitioning of the basin for parallel computation using eight processors.

This sub-basin is located in Baron Fork at Eldon river basin (Oklahoma, USA) and it was selected since it was originally used to test, calibrate, and validate the tRIBS model ([Ivanov et al., 2004a; 2004b](#)). The Peacheater Creek basin has a drainage area of 64 km², with elevations ranging between 248 and 432 m.a.s.l. The sub-basin is characterized by gentle (slopes between 2% and 5%) to steep (slopes between 15% and 40%) hillslopes. The vegetation is a mixture of oak-hickory-pine forest grasslands (mainly located in the southern part of the basin) and cropland-urban areas (mainly in the northern region). The

impervious fraction of the sub-basin is estimated to be about 1.56%. The predominant soil consists of gravelly silt loams (Ivanov et al., 2004a ; 2004b).

Additional details on the climate, hydrology, and basin characteristics have been presented in (Ivanov et al., 2004a ; 2004b; Vivoni et al., 2007).

4.1.2. Setup of modelling experiments

Two different modelling experiments were carried out:

1. Stochastic weather generation: 5,000 years of punctual hourly weather were generated with AWE-GEN (Fatichi et al., 2011); calibrated with the climate data recorded from 1997 to 2016 (both years included) at the Westville weather station: Rainfall, temperature, wind speed, radiation, cloudiness, relative humidity, and atmospheric pressure. Raw data had a resolution of 5 min. The data was processed into hourly values, in order to force AWE-GEN and perform the stochastic weather generation.
2. Hydrological simulations: Numerical simulations in PC were carried out using the tRIBS model with a TIN of 6,680 nodes (Figure 4.1b) derived from a US Geological Survey (USGS) 30-m DEM using the procedure described in Vivoni et al. (2004). The simulations were based on a model calibration conducted by Ivanov et al. (2004a ; 2004b) as part of the distributed model intercomparison project. Ivanov et al. (2004a ; 2004b) obtained a correlation coefficient of 0.763 and a Nash-Sutcliffe coefficient of 0.565 (which can be considered as satisfactory following the guidelines exposed in Sirisena et al. (2018)) for the hourly simulated streamflow at the outlet of Baron Fork basin (in which PC is located), compared to the hourly observed streamflow from April of 1994 to July of 2000. To make the experiment approachable, the computational load was balanced by using parallel computing techniques. The basin was partitioned into eight different parts using a surface-flow partitioning script (Vivoni et al., 2011) with the graph-partitioning software METIS (Karypis and Kumar, 1999), which balances the number of TIN nodes across processors minimizing the dissections that occur in the channel network. Each processor calculates tRIBS variables within each part of the basin and, every time step each processor sends messages to the others to sum up all the calculations done within the basin. Thus, the computational load is balanced between the different processors (Figure 4.1c) and the experiment is approachable.

The experiment was carried out by using Magerit, a high performance computer owned by the Technical University of Madrid. The Intel family processors architecture, which consists of 41 nodes with two intel processors Intel XEON E5-2670 of eight cores each, and 64 GB RAM/Core was used. The time spent in the continuous hydrological simulations was two and a half months, whilst the weather simulations were carried out within one day of computation. Regarding to the Hybrid method simulations, they were performed within fourteen hours (due to the continuous simulation of 50 years, whilst the event-based simulations took one hour).

4.1.3. Limitations of the experiment

The applied methodology has some limitations that should be noted:

1. Rainfall was considered uniform within the whole basin, as a result of using a punctual stochastic weather generator. As pointed out by [Liuzzo et al. \(2010\)](#), due to the small size of PC, there is not a big difference between considering the spatial and punctual rainfall. However, within the same study, accounting or not for the rainfall spatially had an appreciable difference for a basin with a bigger drainage area (Baron Fork, 808 km²).
2. The study is focused on return periods up to 500 years. For higher return periods, the length of the generated weather forcing series should be analysed to ensure that it is representative enough of the return period studied.
3. The procedure did not account for climate-change effects or land-use changes, but considered stationary climate conditions. However, interannual variations of the mean annual rainfall were accounted for due to the nature of AWE-GEN.
4. The methodology is applied to one basin, which limits the extrapolation of the results obtained.

4.1.4. Validation of the method. Hybrid method Vs. Continuous simulation

In several sections within this chapter, in order to compare the values obtained at the different storm ranks and the results of the Hybrid method to the reference value (the one obtained from the continuous simulation), the relative error (RE) was obtained as follows (Equation (4.1)):

$$RE (\%) = \left(1 - \frac{\text{Compared value}}{\text{Reference value}}\right) \cdot 100, \quad (4.1)$$

4.1.4.1. Magnitude and recurrence. Univariate frequency analysis

An analysis of how the storm events were related to the maximum annual peak-flow and volume was carried out, focusing on the:

1. Probability that a storm with a determined rank generates the maximum annual value of the (a) peak-flow and (b) volume.
2. Relation between the storm rank and the return period associated to the (a) maximum annual peak-flow and (b) maximum annual volume.
3. Maximum storm rank required to obtain 95% and 99% probability of achieving the (a) maximum peak-flow and (b) maximum volume for a specific year and for different ranges of the return period.

The return periods associated to the maximum annual peak-flows and to the maximum annual volumes were obtained independently, by using the Gringorten plotting position formula ([Gringorten, 1963](#)).

4.1.4.2. Seasonality

A seasonality analysis was carried out based on circular statistics ([Berens, 2009](#)). As exposed by ([Parajka et al., 2010](#)), circular statistics are an effective method to define the basis of the timing of hydrological extreme events within a year. The date of occurrence of an event in year i can be graphed on a unitary circle to give the angle (θ_i) in polar coordinates (Equation (4.2)):

$$\theta_i = D \cdot 2\pi/L, \quad (4.2)$$

where D stands for the day of the year (one for 1 January, two for 2 January, etc.) and L the number of days within the year (365, or 366 if it is a leap year). Then, the mean seasonality can be defined as the average vector from the origin which represents the mean date of occurrence ($\bar{\theta}$) of all the maximum annual floods (peak-flow or volume) within the case study basin. The $\bar{x}$ and $\bar{y}$ coordinates of the average vector are obtained from the sample of 5,000 extreme events by (Equations (4.3) and (4.4)):

$$\bar{x} = \frac{\sum_{i=1}^{5,000} \cos(\theta_i)}{5,000}, \quad (4.3)$$

$$\bar{y} = \frac{\sum_{i=1}^{5,000} \sin(\theta_i)}{5,000}, \quad (4.4)$$

therefore (Equation (4.5)):

$$\bar{\theta} = \tan^{-1} \frac{\bar{y}}{\bar{x}}, \quad (4.5)$$

whereas the variability of the date of occurrence about the mean date is characterized by the length parameter $\bar{r}$ (Equation (4.6)):

$$\bar{r} = \sqrt{\bar{x}^2 + \bar{y}^2}, \quad (4.6)$$

which ranges from zero (uniform distribution around the year) to one (all extreme events occur on the same date of the year).

The values of $\bar{\theta}$ and $\bar{r}$ obtained from the continuous simulation to those obtained from the different storm ranks and from the Hybrid method.

4.1.4.3. Dependence

An analysis on how the different storm ranks and the Hybrid method affected the dependence between the peak-flow and volume was performed. To do so, (1) graphical techniques and (2) numerical measures were used.

1. The Chi-Plot, which is a technique that displays a measure of location of an observation regarding the whole of the observations (λ_i) against a measure of the Chi-square test statistic for independence (X_i) was used. Therefore, the bigger the distance between the points and the horizontal axis is, the larger the dependence is. The dependence is positive if the points are above the upper control limit, and negative if they are located below the lower control limit (Fisher and Switzer, 1985; 2001), which was established with a probability of 90% as shown in Genest et al. (2007).
2. Spearman's rho and Kendall's tau. Moreover, dependence measures are needed to procure a quantitative value of the dependence relation between variables. For this purpose, the Spearman's rho and Kendall's tau were adopted as rank-based non-parametric measures of dependence.

4.1.4.4. Magnitude and recurrence. Copula-based frequency analysis.

Finally, an analysis on how the rank of storms and the Hybrid method affected the bivariate properties of the pairs of peak-flow and volume by carrying out a copula-based analysis. To do so, the MATLAB toolbox MhAST Toolbox (Sadegh et al., 2018) was used. Within that toolbox, a total of 26 parametric models of copula families can be fitted to bivariate data. Within this study, for the sake of simplicity, the focus was established on three well-known Archimedean one-parameter copula families for their uses in hydrological sciences: Clayton, Gumbel and Frank. The procedure was as follows:

First, the three studied copulas were fitted to the pairs peak-flow and volume values obtained from the continuous simulation using MhAST. For these three copulas, MhAST uses the MATLAB built-in function, which estimates the parameter (θ_c) of each copula using the method of maximum likelihood. The selection of the copula that provided the best fit was based on the root mean square error (RMSE) (Equation (4.7)) and the Nash–Sutcliffe efficiency (NSE) (Equation (4.8)) coefficient, which measures how different the empirical observed bivariate probabilities ($\tilde{Y}$) and their modelled bivariate counterpart (Y) are:

$$\text{RMSE} = \sqrt{\frac{\sum_{i=1}^n [\tilde{y}_i - y_i(\theta_c)]^2}{n}}, \quad (4.7)$$

$$\text{NSE} = 1 - \frac{\sum_{i=1}^n [\tilde{y}_i - y_i(\theta_c)]^2}{\sum_{i=1}^n [\tilde{y}_i - \bar{\tilde{y}}]^2}, \quad (4.8)$$

RMSE values are within the range $[0, \infty)$ with zero being a perfect fit, whereas NSE ranges in the interval $(-\infty, 1]$, with one being the perfect fit. The best fit is the one that had a lower value of RMSE and a higher value of NSE.

Once the copula family was chosen, that copula was fitted to the different pairs of peak-flow and volume obtained considering different storm ranks, according to the rainfall criteria separation studied. The RE of the estimated copula parameter with respect to the copula parameter of the continuous simulation was obtained.

Finally, an graphical analysis was performed to analyse the differences between the Kendall's return period (T_K) (Salvadori and De Michele, 2011) calculated by both, continuous simulation and a) different storm ranks according to the V_{Event} and I_{max} criteria b) the Hybrid method. Kendall's return period represents the mean inter-arrival time of

critical events lying on the probability level t , and can be obtained as follows (Equation (4.9)):

$$T_K = \frac{\mu}{1-K_C(t)} = \frac{1}{1-K_C(t)}, \quad (4.9)$$

where μ is the average interarrival time of events (i.e., $\mu = 1$ in this Chapter, as annual maxima are being analysed) and $K_C(t)$ is the Kendall's distribution function, where t represents the probability level. For the sake of brevity, the reader is referred to (Salvadori and De Michele, 2011; Brunner et al., 2016) for further details. Finally, the isolines of T_K of 10, 50, and 100 years were obtained using the MhAST toolbox, estimated through continuous simulation and a) through different storm ranks for the studied criteria separation b) through the Hybrid method.

4.2. Obtained results

4.2.1. Stochastic weather generation. Rainfall extremes validation.

The ability of the weather generator (AWE-GEN) to reproduce rainfall extremes was analysed by comparing the stochastic rainfall frequency curves with those observed at different aggregation periods (Figure 4.2a–d, 1 h, 6 h, 12 h, 24 h, respectively). As shown, the behaviour of the observed rainfall frequency curves was mostly within the range of the simulated ones.

Furthermore, it was analysed how the seasonality of the storm events was retained by AWE-GEN. To carry out the analysis, it was necessary to separate the storm events. By applying the exponential method (exposed in Section 3.1.1.1) a MIT of 15 h was determined from the observed hourly rainfall time-series of Westville Station. Applying the methods shown in Section 4.1.4.2, the seasonality of two main characteristics of storm events (Figure 3.3) was compared: The maximum annual $I_{\max}$ (Figure 4.2e) and the maximum annual V_{Event} (Figure 4.2f).

Figure 4.2: Validation of the generated rainfall series. (a–d) Comparison between the observed rainfall frequency curve (blue dots) (obtained by applying the Gringorten plotting position formula ([Gringorten, 1963](#)) and synthetic rainfall frequency curves obtained by bootstrapping 1,000 series of 40 years of hourly rainfall obtained from the 5,000 years of synthetic weather series generated: (a) Maximum annual cumulated rainfall (P) in: (a) 1 h, (b) 6 h, (c) 12 h, and (d) 24 h. (e) Comparison between the seasonality of the observed maximum annual intensity (blue circles), with the predominant direction (blue line) compared to the predominant direction of the seasonality of 1,000 series of 20 years of maximum annual depth obtained by bootstrapping from the 5,000 years of synthetic weather generated (in grey lines), with the mean predominant direction represented in black. (f) The same as e, but for total depth, with observed values represented in red circles.

The seasonality is well-reproduced by AWE-GEN for the maximum annual $I_{\max}$ ([Figure 4.2e](#)). The mean occurrence ($\bar{\theta}$) (represented by the direction of the vectors) is late June (observed, in blue) and early July (simulated, in black), with a similar variability of seasonality ($\bar{r}$) (represented by the length of the vectors ([Figure 4.2e](#))), with an

observed and simulated value of 0.61 and 0.56, respectively). However, in the case of V_{Max} (Figure 4.2f), the observed seasonality is slightly more disperse than the generated (the length of the observed vector is 0.16, whereas the simulated is 0.29) and $\bar{\theta}$ is delayed by one month. Overall, the generated synthetic series are able to reproduce extreme rainfall properties.

4.2.2. Rainfall versus flood comparison

Once the weather series had been generated, they were used as weather forcing for the hydrological model tRIBS performing the simulations. Afterwards, the relationship between the storm ranks and the flood hydrographs generated at the basin outlet was obtained.

4.2.2.1. Magnitude and Recurrence. Univariate Frequency Analysis

Table 4.1 shows the probability that a storm of a specific rank generates the (a) maximum annual peak-flow and (b) the maximum annual volume. The three ranking criteria are shown: V_{Event} , I_{max} , and D_{Event} . It can be seen that the V_{Event} criteria was the most relevant; that is, a lower storm rank is required for obtaining both the maximum annual peak-flow and volume.

According to the different storm rank criteria:

1. V_{Event} . Considering the biggest storm (rank one) results in a probability of generating the maximum annual peak-flow (Table 4.1) of more than 53%, whereas considering the storms of rank one and two this probability increases to more than 75%. If the storms considered are up to rank 3, this probability exceeds 86%. When analyzing the maximum annual volume (Table 4.1), the same probabilities are greater than 68%, 86%, and 93%, respectively.
2. I_{max} . The probability of generating the maximum annual peak-flow (Table 4.1) is more than 45%, 63%, and 75% for a storm rank up to one, two, and three, respectively. In the case of maximum annual volume (Table 4.1), the same Figures are 35%, 52%, and 64%.
3. D_{Event} . The probability of achieving the maximum annual peak-flow and volume with these criteria is the lowest. Considering storms up to rank ten, the probability of achieving the maximum peak-flow is less than 40% (Table 4.1), and less than a 60% (Table 4.1) probability of resulting in the maximum annual volume.

Table 4.1: Relative and cumulated frequency that a storm of a specific rank generates the maximum annual peak-flow and the maximum annual volume.

Storm Rank	Relative/Cumulated Frequency of Coincidence with Maximum Annual Peak-Flow (%)			Relative/Cumulated Frequency of Coincidence with Maximum Annual Volume (%)		
	Total Depth	Maximum Intensity	Total Duration	Total Depth	Maximum Intensity	Total Duration
1	53.5/53.5	45.2/45.2	9/9	68.8/68.8	35.6/35.6	17.2/17.2
2	21.7/75.2	18.6/63.8	5.8/14.8	17.9/86.7	17.3/52.9	8.6/25.9
3	11.4/86.6	11.5/75.3	4.8/19.6	6.9/93.6	11.3/64.2	7/32.9
4	5.3/91.9	6.9/82.2	3.3/22.9	2.7/96.3	8/72.2	4.7/37.5
5	3.4/95.3	4.2/86.4	3.6/26.5	1.8/98.1	5.5/77.7	4.5/42
6	1.7/97	3.1/89.4	2.9/29.5	0.7/98.8	4.2/81.9	3.4/45.4
7	1.3/98.2	2.2/91.7	2.9/32.3	0.5/99.3	3.1/85	3/48.4
8	0.8/99	2.1/93.7	2.5/34.8	0.3/99.6	2.8/87.9	3/51.4
9	0.3/99.3	1.5/95.2	2.3/37.1	0.1/99.7	2.5/90.4	2.4/53.8
≥10	0.7/100	4.8/100	62.9/100	0.3/100	9.6/100	46.2/100

Therefore, for PC, the best criterion of the storm rank is V_{Event} , followed up by I_{max} when analyzing the correspondence between the storms and maximum annual peak-flow and volume. For the sake of simplicity, the following results only focused on these two criteria, excluding D_{Event} .

Afterwards, the focus was on the magnitude of peak-flows and volumes. The different storm ranks (Figure 4.3, total depth criteria (red) maximum intensity criteria (blue)) were related to the associated return period (Tr) of peak-flows (Figure 4.3a) and volumes (Figure 4.3b) obtained from the analysis of the 5,000 years flow simulated at the basin outlet. According to the two criteria selected, it can be seen that:

1. V_{Event} . The higher the Tr of the peak-flow (Figure 4.3a) and volume (Figure 4.3b) are, the lower the storm rank needed. Therefore, there is a strong correlation between the Tr and the storm rank. For the Tr of the peak-flow over 10 years, all the storm events have a rank lower than or equal to four, whereas for the Tr over 100 years all the storms are of rank one or two. For the Tr of volume over 10 years, all the storm events have a rank lower than or equal to three, whereas for the return periods over 100 years all the storms are also of rank one or two.

2. $I_{\max}$. There is more dispersion than in the V_{Event} criteria. For the Tr of the peak-flow over 10 years, storms have a rank of ten or lower, whereas for the Tr over 100 years all the storms up to rank three should be considered. In the case of volume, the Tr higher than 10 years corresponds to storms with ranks up to 23 (not shown in Figure 4.3), whereas for the Tr over 100 years, storms up to rank ten can generate the maximum annual volume.

Figure 4.3: Storm order that corresponds to every return period (Tr) of maximum annual peak-flow (a) and maximum annual volume (b) represented as red dots for the total depth criteria and as blue dots for the maximum intensity rank criteria. The numbers on the right represent the probability that a storm of a specific rank generates the maximum peak-flow (a) or volume (b) of given a year.

Finally, regarding the analysis of magnitude and recurrence, the maximum storm rank required to obtain 95% and 99% probability of achieving the maximum annual peak-flow and the maximum annual volume for a specific year and for different ranges of Tr for the V_{Event} and I_{Max} criteria was calculated (Table 4.2). Considering storms up to rank six according to the V_{Event} criteria, there is a 95% probability of achieving the maximum annual peak-flow and volume for all ranges of the Tr, which increases to 99% if all storm

events up to rank nine are considered. For I_{Max} , the ranks are 13 and 22 for the same probability values. As also shown in Figure 4.3, the storm rank required is less as the Tr increases in the two ranking criteria. It is remarkable that, in the case of V_{Event} , for return periods higher than 50 there is a probability of achieving the maximum annual peak-flow and the maximum annual volume of 99% if the three highest events are considered.

Table 4.2: Maximum storm rank required to obtain 95% and 99% probability of achieving the maximum peak-flow or volume for a specific year, depending on the ranking criteria used and the range of return periods studied.

Probability (%)	Tr (Years)	Maximum Storm Rank to be Considered		Maximum Storm Rank to be Considered	
		Peak-Flow		Volume	
		Total Depth	Maximum Intensity	Total Depth	Maximum Intensity
95/99	All range	5/8	9/16	4/7	13/22
95/99	$1 \leq Tr < 10$	6/9	10/16	4/7	13/22
95/99	$10 \leq Tr < 50$	3/3	4/7	1/2	8/14
95/99	$50 \leq Tr < 100$	2/3	2/3	1/2	6/10
95/99	$Tr \geq 100$	2/2	2/3	1/2	6/6

4.2.2.2. Seasonality

The preservation of seasonality by the storms depending on the rank considered was analysed. Figure 4.4 shows the seasonal behaviour of storms regarding the maximum peak-flow (Figure 4.4a–d and Figure 4.4i–l) and maximum volume (Figure 4.4e–f and Figure 4.4m–p). Comparisons were carried out with the seasonality of the continuous simulation (red in Figure 4.4a–f and blue in Figure 4.4i–p) and the consideration of storms up to rank one, three, five and ten for the V_{Event} (Figure 4.4a–f) and I_{Max} (Figure 4.4i–p) criteria.

According to the two rainfall criteria selected:

1. V_{Event} . The seasonality is preserved almost equally if storms are at least considered up to rank three for both the maximum peak-flow and maximum volume.
2. I_{Max} . Within this case, to preserve the seasonality obtained from the continuous simulation, storms up to rank ten should be considered in both cases, the maximum peak-flow and maximum volume.

Figure 4.4: Comparison between the seasonality of the maximum annual peak-flow (red lines in (a–d); and blue lines in (i–l) and maximum annual volume (red lines e–h; and blue lines (m–p) of the continuous simulation with respect to that obtained by applying different criteria (total depth (a–h) and maximum intensity (i–p)) and accounting for different rank orders (up to one, three, five and ten), in which the predominant direction of the seasonality is represented by a circle with a black line.

To quantify the impact of the rank of storms in seasonality, the values of $\bar{\theta}$ and $\bar{r}$ were obtained from the continuous simulation and compared to the ones obtained from the different storm ranks, obtaining the RE (Equation (4.1)).

Table 4.3 and Table 4.4 show the different values of $\bar{\theta}$ and $\bar{r}$ depending on the criteria and the RE, for the maximum peak-flow (Table 4.3) and for maximum volume (Table 4.4). According to the two rank criteria:

1. V_{Event} . With storms up to rank five, there is less than 1% absolute RE in the estimation of the mean direction of seasonality for both the peak-flow (Table 4.3) and volume (Table 4.4). Regarding the radius, which measures the dispersion of the seasonality, considering storms up to rank five have an absolute RE lower than 4%.
2. I_{Max} . With this criterion, the smaller absolute values of RE are in the estimation of the mean direction of the maximum peak-flow seasonality (2% if storms up to rank five are considered). When analysing the mean direction of volume seasonality, RE is over 6% for storms up to rank ten. When it comes to the radio, all RE values are over 10%.

Therefore, the V_{Event} sorting criterion is the best for preserving the seasonality of floods in this case study.

Table 4.3: Seasonality of the maximum annual peak-flow represented by the angle of its mean direction ($\bar{\theta}$) and its radio ($\bar{r}$), for different storm ranks and for the two different criteria studied. The relative error (RE) is also shown.

Maximum Storm Rank Considered	Total Depth		Maximum Intensity		Total Depth		Maximum Intensity	
	$\bar{\theta}$	$\bar{r}$	$\bar{\theta}$	$\bar{r}$	$RE_{\bar{\theta}}$	$RE_{\bar{r}}$	$RE_{\bar{\theta}}$	$RE_{\bar{r}}$
Continuous Simulation	169.7	0.29	169.7	0.29	—	—	—	—
Rank 1	146.7	0.20	187.2	0.59	13.5	29.8	-10.3	-101
Rank 3	168.1	0.26	176.5	0.47	0.9	10.1	-4.0	-61.5
Rank 5	169.7	0.28	173.0	0.41	0.0	3.8	-2.0	-40.4
Rank 10	169.9	0.29	170.9	0.33	-0.1	0.0	-0.7	413.1

Table 4.4: Seasonality of the maximum annual volume represented by the angle of its mean direction ($\bar{\theta}$) and its radio ($\bar{r}$), for different storm ranks and for the two different criteria studied. The relative error (RE) is also shown.

Maximum Storm Rank Considered	Total Depth		Maximum Intensity		Total Depth		Maximum Intensity	
	$\bar{\theta}$	$\bar{r}$	$\bar{\theta}$	$\bar{r}$	$RE_{\bar{\theta}}$	$RE_{\bar{r}}$	$RE_{\bar{\theta}}$	$RE_{\bar{r}}$
Continuous Simulation	142.9	0.25	142.9	0.25	—	—	—	—
Rank 1	146.7	0.20	187.2	0.59	-2.7	18.9	-31.0	-133
Rank 3	145.4	0.25	172.7	0.44	-1.7	-0.3	-20.8	-76.1
Rank 5	143.7	0.25	162.7	0.38	-0.6	-0.6	-13.8	-51.6
Rank 10	143.2	0.25	151.8	0.30	-0.2	-0.6	-6.2	-20.0

4.2.2.3. Dependence

The preservation of the dependence between the peak-flow and volume of the continuous simulation by the storm ranks was analysed. To do so, as exposed in the methodology, the chi-plots comparing the peak-flow volume dependence from the continuous simulation (red (Figure 4.5a–d) and blue (Figure 4.5e–h) dots) to the different

storm ranks according to the V_{Event} and I_{Max} criteria (grey dots in Figure 4.5) were graphed.

Figure 4.5: Chi-plots of the maximum annual peak-flow and volume. Grey dots represent the values obtained considering specific storm orders up to different ranks (a,e) rank one (b,f) rank three, (c,g) rank five, (d,h) rank ten). Color dots represent the values obtained from the continuous simulation (red in (a–d), blue (e–h)). Storm rank criteria are total depth in (a–d) and maximum intensity in (e–h).

For all cases (Figure 4.5), there is a positive dependence between the peak-flow and volume. Furthermore, when focusing on the upper tail dependence (first quadrant within all the plots in Figure 4.5), the points are located far from the zero value of the y axis, which is the independence hypothesis. Depending on the criteria:

1. V_{Event} . The dependence is well preserved, and both scatterplots are visually almost identical when the storm ranks up to three or more are considered. If the biggest storm of the year is considered, the dispersion is bigger.
2. I_{max} . Dependence is also preserved for this criterion, but a higher dispersion is shown in rank periods up to one and three. In the case of considering only the biggest storm (rank one), a big dispersion is found in the upper tail.

To numerically compare the degree of dependence of the peak-flow and volume of the continuous simulation with respect to different storm ranks, values of Spearman's rho and Kendall's tau were computed (Table 4.5), and their RE (Equation (4.1)). In the case of the V_{Event} criterion, all the absolute values of RE obtained from the different ranks are

lower than 1%, whereas in the case of $I_{\max}$, the absolute RE values are over 2% for storms up to rank five, reaching absolute RE values of less than 1% if only storms up to rank ten are considered.

Table 4.5: Rank-based non-parametric measures of dependence Spearman's rho (ρ) and Kendall's tau (τ) for the pair of values of maximum annual peak-flows and maximum annual volumes regarding the maximum storm rank considered and to the criteria used: Total depth or maximum intensity.

Maximum Storm Rank Considered	Total Depth		Maximum Intensity		Total Depth		Maximum Intensity	
	ρ	τ	ρ	T	RE $_{\rho}$	RE $_{\tau}$	RE $_{\rho}$	RE $_{\tau}$
Continuous Simulation	0.884	0.709	0.884	0.709	—	—	—	—
Rank 1	0.883	0.708	0.955	0.826	0.07	0.01	-8.05	-16.56
Rank 3	0.882	0.706	0.922	0.767	0.17	0.31	-4.26	-8.18
Rank 5	0.883	0.708	0.903	0.739	0.05	0.05	-2.16	-4.27
Rank 10	0.884	0.709	0.888	0.716	0.00	-0.01	-0.41	-1.01

Therefore, to preserve the dependence between maximum annual peak-flows and volumes, V_{Event} is also the best criteria for the case study.

4.2.2.4. Magnitude and Recurrence. Copula-based Frequency Analysis

The three Archimedean copulas exposed in the methodology were fitted to the 5,000 pairs of peak-flows and volumes using the MhAST Toolbox (Sadegh et al., 2018). Table 4.6 shows the parameters estimated for each of the copulas, obtained by the method of maximum likelihood, with the value of RMSE and the NSE for each of the copula fits. For the studied copulas, the best fit corresponds to the Frank copula, followed closely by the Gumbel copula. As both Gumbel and Frank provided a good fit, the Gumbel copula was chosen for further analysis as it is the only one of the three studied copulas able to model upper tail dependence (Poulin et al., 2007).

Table 4.6: Parameters of the Archimedean copula estimation from the pair of values of maximum annual peak-flows and maximum annual volumes of the continuous simulation, with the root mean square error (RMSE) and Nash–Sutcliffe error (NSE).

Copula	Parameter	RMSE	NSE
Clayton	3.10	1.68	0.992
Frank	11.5	0.58	0.999
Gumbel	2.98	0.70	0.998

Table 4.7 shows the parameter fit of the Gumbel copula for different storm ranks according to the two criteria studied. In the case of the V_{Event} criterion, the absolute RE is lower than 2% considering the storms up to any rank. Only in the case of considering storms up to rank ten with the total V_{Event} criterion, the absolute RE is lower than 0.5%. In the case of the $I_{\max}$ criterion, storm ranks up to ten should be considered for absolute values of RE lower than 5%.

Table 4.7: Parameters of the Gumbel copula estimation from the pair of values of maximum annual peak-flows and maximum annual volumes regarding the maximum storm rank considered and the criteria used: Total depth or maximum intensity, and relative error (RE) respect to the continuous simulation.

Maximum Storm Rank Considered	Total Depth		Maximum Intensity	
	Gumbel Parameter	RE	Gumbel Parameter	RE
Continuous Simulation	2.98	—	2.98	—
Rank 1	2.94	1.37	4.84	-62.34
Rank 3	3.00	-0.79	3.82	-28.32
Rank 5	3.02	-1.18	3.40	-13.91
Rank 10	2.99	-0.25	3.08	-3.26

The results of Table 4.7 have a reflection when T_K are estimated. Figure 4.6 shows the T_K of 10 (dashed), 50 (dashed-dotted) and 100 (continuous) years of the continuous simulation (red (Figure 4.6a–d) and blue (Figure 4.6e–h) lines) compared to the different storm ranks according to V_{Event} and I_{max} criteria (grey lines in Figure 4.6).

Figure 4.6: Kendall return periods of 10 (dashed), 50 (dashed-dotted) and 100 (continuous) years. Grey dots represent the values obtained considering specific storm orders up to different ranks (a,e) rank one (b,f) rank three, (c,g) rank five, (d,h) rank ten). Color dots represent the values obtained from the continuous simulation (red in (a–d), blue (e–h)). Storm rank criteria is total depth in (a–d) and maximum intensity in (e–h).

According to each ranking criterion:

1. V_{Event} . Visually, there is a good agreement between the T_K estimated with the V_{Event} criterion and the continuous simulation (Figure 4.6a–d). Worse agreement is shown for high return periods (100 years). In the case of considering storms with ranks up to ten (Figure 4.6d), the T_K are almost coincident.

2. $I_{\max}$. There is not a good agreement between the T_K estimated with this criterion compared to the continuous simulation for ranks up to three. If the biggest storm is considered, the T_K s are underestimated. In the case of considering storm ranks up to three, the return periods of 50 and 100 years are highly overestimated. T_K s are closer with storms up to rank five (still overestimated for 50 and 100 years T_K s), and almost a perfect agreement is achieved if rank periods up to 10 are considered.

4.2.2.5. Analysis of previous works

As exposed, some studies in the literature have previously analysed the influence of storms on the derivation of the flood frequency curve through event-based approaches. [Table 4.8](#) shows the main characteristics of the research on this topic. [Flores-Montoya et al. \(2016\)](#) (extended analysis in [Flores-Montoya \(2018\)](#)) carried out a similar experiment in two French basins. They extracted the storms from the synthetic time series of spatial-punctual rainfall generated with the Rainsim V3 ([Burton et al., 2008](#)), and forced the event-based distributed model RIBS ([Garrote and Bras, 1995](#)) (predecessor of tRIBS) with them. As they accounted for spatial variability, they separated the storms based on the mean area rainfall, and ranked them by total depth, mean intensity (instead of maximum intensity), and total duration. They carried out the analysis only for the five highest storms of each year, and performed the experiment simultaneously for Gnrarges and Corbs. [Sordo-Ward et al. \(2015; 2016\)](#) performed a similar analysis in four Spanish basins, by also coupling the RainSim V3 with a simpler semi distributed model ([Sordo-Ward et al. 2012; 2013](#)). Thus, they were able to analyse all the events each year as the experiment resulted less computationally expensive.

When comparing to the present study, [Sordo-Ward et al. \(2015; 2016\)](#) and [Flores-Montoya et al. \(2016\)](#) both used event-based approaches instead of a continuous approach. As a consequence, they needed to characterize the soil-moisture in the basin prior to the storm events. In the case of [Sordo-Ward et al. \(2015; 2016\)](#), they considered a constant value of the curve number in each of the sub-basins for all the storms. Regarding [Flores-Montoya et al. \(2016\)](#), they determined the initial soil moisture in a probabilistic way. Within this study, as the simulations were performed with tRIBS, all the conditions prior to the storm events were given by the continuous simulation.

Table 4.8: Comparison between similar case studies and the methodology used in each of them. For comparison purposes, the basin area, weather/rainfall generator, hydrological model and the way of accounting soil moisture are shown.

Case Study	Area (km ²)	Weather/Rainfall Generator	Spatial Resolution	Hydrological Model	Type	Initial Soil Moisture
Peacheater Creek	64	AWE-GEN ⁵	Punctual	TRIBS ⁷	Fully distributed-Continuous	Continuous assessment
Générargues ^{1,2}	245	RainSim V3 ⁶	Spatial-Punctual	RIBS ⁸	Fully distributed-Event based	Probabilistic
Corbès ^{1,2}	262.3	RainSim V3 ⁶	Spatial-Punctual	RIBS ⁸	Semidistributed-Event based	Probabilistic
Navacerrada ³	20	RainSim V3 ⁶	Spatial-Punctual	Stochastic HEC-HMS ⁹	Semidistributed-Event based	Deterministic
Santillana ⁴	211	RainSim V3 ⁶	Spatial-Punctual	Stochastic HEC-HMS ¹⁰	Semidistributed-Event based	Deterministic
El Pardo ⁴	495	RainSim V3 ⁶	Spatial-Punctual	Stochastic HEC-HMS ¹¹	Semidistributed-Event based	Deterministic
Manzanares ⁴	1294	RainSim V3 ⁶	Spatial-Punctual	Stochastic HEC-HMS ¹²	Semidistributed-Event based	Deterministic

¹Flores-Montoya et al. (2016) ²Flores-Montoya (2018) ³Sordo-Ward et al. (2015) ⁴Sordo-Ward et al. (2016) ⁵Fatichi et al. (2011) ⁶Burton et al. (2008) ⁷Ivanov et al. (2004a;b) ⁸Garrote and Bras. (1995) ⁹Sordo-Ward et al. (2012; 2013)

Table 4.9 shows a summary of the results obtained within this Chapter and the comparison of the results obtained in other studies (some aspects have not been studied by the other compared studies and are therefore not shown). Several aspects can be remarked when analyzing Table 4.9:

Table 4.9: Results obtained for the different case studies considered in Table 4.8. PF represents the maximum annual peak-flow, V represents the maximum annual volume, and RE represents the relative error.

Case Study	Best Criteria	1 st Rank PF Coincidence (%)	1 st rank V Coincidence (%)	Rank 95% PF	Rank 95% V	Rank Seasonality PF and V (<5% RE)	Rank Copula Parameter (<5% RE)
Peacheater Creek	Total depth	53	69	5	4	3	1
Générargues ^{1,2}	Total depth	70	88	4	2	—	—
Corbès ^{1,2}	Total depth	70	88	4	2	—	—
Santillana ⁴	Total depth	66	—	4	—	—	—
El Pardo ⁴	Total depth	67	—	4	—	—	—
Manzanares ⁴	Total depth	55	—	6	—	—	—
Navacerrada ³	Total depth	63	—	6	—	—	—

¹Flores-Montoya et al. (2016) ²Flores-Montoya (2018) ³Sordo-Ward et al. (2015) ⁴Sordo-Ward et al. (2016)

1. In all the case studies, among the three criteria of storm event ranking, the best was total depth. This suggests that the choice between one criterion or another has a low dependence on the hydrological model used or how the initial moisture is accounted. Accounting or not for the spatial variability of rainfall does not seem to influence either, but this might be due to the small size of Peacheater Creek.
2. Regarding the probability of achieving the maximum annual peak-flow, the lowest values (55% and 53%) correspond to the Manzanares and Peacheater Creek, respectively. The first is the biggest basin (1294 km²), and the reduction of probability can be due to the importance of propagation processes and the temporal patterns of the storm events. However, Peacheater Creek is the second smallest basin in [Table 4.9](#). The reduction of the coincidence with the maximum annual peak-flow might be due to how antecedent basin conditions prior to the storm event are considered. As the model is continuous, some lower rank events may concur with wetness states of the basin, incurring higher peak-flows. This phenomenon cannot occur in the [Sordo-Ward et al. \(2015; 2016\)](#) study basins, as the initial soil moisture was considered constant and equal for all the storm events.
3. When the coincidence of the first rank total depth storm with the maximum annual volume is analysed (Peacheater Creek and [Flores-Montoya et al. \(2016\)](#)), it can be seen that there is a higher probability of coincidence than that of the peak-flow. This suggests that total depth has a better correlation with the maximum annual volume than with the maximum annual peak-flow. This can also be seen when the minimum rank of storm required is analysed. Smaller ranks are required to achieve a 95% probability of obtaining the maximum annual volume than for obtaining the maximum annual peak-flow with the same probability.
4. In all the case studies shown, the maximum peak-flow can be obtained with a 95% probability considering all the storms up to rank four, five or six. In the case of total volume, the maximum rank required is reduced to two or four.
5. Within this section, it has been shown that seasonality and bivariate properties of extreme floods can be well-preserved if storms up to rank three and one are considered, respectively.

Therefore, to sum up, as a general recommendation, considering the five biggest storms per year classified by their total depth can result in an accurate derivation of the

flood frequency curve when using event-based models, close to that obtained from continuous simulations. Based on the results obtained, the five major events of each year in terms of total depth were selected from all the storm events within the 5,000 years to perform the Hybrid method simulations.

4.2.1. Initial soil moisture: simulation and probability distributions

A continuous simulation of 50 years was performed, forcing tRIBS with 50 years of continuous hourly weather. As previously mentioned, to develop the Hybrid method it was assumed that there is independence between rainfall events and prior hydrological soil moisture conditions.

The relationship between $MSM10_{Event}$ and V_{Event} for all the storm events in the 50 years of simulation was analysed (Figure 4.7). If all the storm events (3,797 events in the 50 years) are considered, the value of Kendall's tau for the pair of values $MSM10_{Event}$ - V_{Event} is 0.034, whereas if considering only the five largest values (2500 events) the value is 0.063. Thus, the storm events can be considered as independent (Kendall's tau values tend to zero) of the prior initial soil moisture conditions of the basin, as previously stated by Paquet et al. (2013).

It should be pointed out that this hypothesis, in the case of modelling frameworks that couple hydrological with atmospheric models (i.e.: WRF-Hydro (Gochis et al., 2015)), might not be valid due to the relationship between terrestrial hydrology and weather.

Figure 4.7: Comparison of the initial soil moisture value associated to each storm event ($MSM10_{Event}$) and the total depth associated to the storm event (V_{Event}) in the 50 years of continuous simulation for: all the storm events (blue dots) and for the 5 largest total depth events per year (red dots).

Afterwards, the cumulated frequency distributions of $MSM10_{Event}$ values of the 50 years' continuous simulation for each month of the year were obtained (Figure 4.8).

Figure 4.8: Cumulated Frequency distributions of $MSM10_{Event}$ values within the 50 years of simulation for each month of the year.

These frequency distributions are used as input for the Hybrid method. Each storm event is associated with the frequency distribution corresponding to the month of occurrence of the storm.

Afterwards, within a Monte Carlo framework, a value of $MSM10_{Event}$ is associated to each storm event. Once $MSM10_{Event}$ is obtained, a synthetic spatio-temporal basin state is generated, with a $MSM10$ value in the basin equal to the $MSM10_{Event}$. Figure 4.9 shows as an example three synthetic initial basin states for different $MSM10_{Event}$ values.

Figure 4.9: Examples of synthetic initial basin states with a $MSM10$ value of (a) 0.5 (b) 0.6 and (c) 0.7. Colorbar represents the values of soil moisture in the different voronoi polygons.

4.2.2. Hybrid method performance

Once the initial basin states were obtained, they were coupled to each of the 25,000 storm events (5 x 5,000 years), serving both as forcing for the hydrological model tRIBS

simulations. Thus, 5 flood hydrographs per year were obtained, amounting 25,000 flood hydrographs which were compared to those obtained from the 5,000 years of continuous simulation.

4.2.2.1. Seasonality

Figure 4.10 shows the seasonal behaviour of the Hybrid method regarding the maximum peak-flow (Figure 4.10a) and maximum volume (Figure 4.10b). Comparisons were carried out with the seasonality of the continuous simulation (red lines). It can be seen the mean of the date of occurrence is well preserved for both maximum annual peak-flows and maximum annual volumes, whereas the variability of the date of occurrence about the mean date is lower in the Hybrid method in both cases.

Figure 4.10: Comparison between the seasonality of the continuous (red) maximum annual peak-flow (a); and maximum annual volume (b) with respect to that obtained by applying the Hybrid method (black), in which the predominant direction of the seasonality is represented by a circle with a line.

To quantify the seasonal performance of the Hybrid method, the values of $\bar{\theta}$ and $\bar{r}$ were obtained from the continuous simulation and compared to the ones obtained from the Hybrid method, obtaining the RE (Equation (4.1)).

Table 4.10 shows the different values of $\bar{\theta}$ and $\bar{r}$ depending on the criteria and the RE, for the maximum peak-flow and for maximum volume (Table 4.10):

3. Maximum peak-flow. There is less than 8% absolute RE in the estimation of the mean direction of seasonality. Regarding the radius, which measures the dispersion of the seasonality, the Hybrid method has an absolute RE of almost 52 %.
4. Maximum volume. In this case, the Hybrid method is more accurate than in the maximum annual peak-flow. The mean direction of seasonality is estimated with an

absolute value of RE lower than 6 %, whereas the dispersion of the seasonality is estimated with an RE of 32 %.

Table 4.10: Seasonality of the maximum annual peak-flow represented by the angle of its mean direction ($\bar{\theta}$) and its radius ($\bar{r}$), for different storm ranks and for the two different criteria studied. The relative error (RE) is also shown.

Method	Maximum Peak Flow		Maximum Volume		Maximum Peak Flow		Maximum Volume	
	$\bar{\theta}$	$\bar{r}$	$\bar{\theta}$	$\bar{r}$	$RE_{\bar{\theta}}$	$RE_{\bar{r}}$	$RE_{\bar{\theta}}$	$RE_{\bar{r}}$
Continuous Simulation	169.7	0.29	142.9	0.25	—	—	—	—
Hybrid	157.7	0.14	151.0	0.17	7.1	51.7	-5.7	32.0

4.2.2.2. Dependence

An analysis of how the Hybrid method is able to preserve the dependence between the peak-flow and volume of the continuous simulation is performed. To do so, the chi-plot was graphed comparing the peak-flow volume dependence from the continuous simulation (red dots in Figure 4.11) to the Hybrid event-based model (black dots in Figure 4.11).

There is a positive dependence between the peak-flow and volume. Furthermore, when focusing on the upper tail dependence (first quadrant in Figure 4.11), the points are located far from the zero value of the y axis, which is the independence hypothesis.

Figure 4.11: Chi-plot of the maximum annual peak-flow and volume. Red dots represent the values the values obtained from the continuous simulation, whereas black dots represent the values obtained from the Hybrid-based method.

To numerically compare the degree of dependence of the peak-flow and volume of the continuous simulation with respect to Hybrid event-based model, values of Spearman's rho and Kendall's tau were computed (Table 4.11), and their RE (Equation (4.1)). Spearman's rho is obtained with a RE lower than 7 %, whereas Kendall's tau differs around 13 % in terms of RE values among the continuous simulation and the Hybrid model.

Table 4.11: Rank-based non-parametric measures of dependence Spearman's rho (ρ) and Kendall's tau (τ) for the pair of values of maximum annual peak-flows and maximum annual volumes of the continuous and Hybrid-based simulation.

Maximum Storm Rank Considered	Total Depth		Total Depth	
	ρ	τ	RE_ρ	RE_τ
Continuous Simulation	0.884	0.709	–	–
Event Based Simulation	0.941	0.803	-6.44	-13.3

4.2.2.3. Magnitude and recurrence. Copula-based frequency Analysis

The Archimedean Gumbel copula was fitted, as it is the only one of the three studied copulas able to model upper tail dependence (Poulin et al., 2007) to both the 5,000 pairs of peak-flows and volumes of the continuous and Hybrid-based model respectively using the MhAST Toolbox (Sadegh et al., 2018). Table 4.12 shows the parameters estimated for each of the copulas, obtained by the method of maximum likelihood for each of the copula fits. The absolute RE is lower than 13%.

Table 4.12: Parameters of the Archimedean Gumbel copula estimation from the pair of values of maximum annual peak-flows and maximum annual volumes of the continuous simulation and Hybrid simulation, and relative error (RE) respect to the continuous simulation.

Copula	Parameter Continuous	Parameter Hybrid	RE
Gumbel	2.98	3.36	-12.75

Furthermore, a comparison of the Kendall's return performed. Figure 4.12 shows the T_K of 10 (dashed), 50 (dashed-dotted) and 100 (continuous) years of the continuous simulation (red lines) and the Hybrid-based method (grey lines).

Figure 4.12: Kendall return periods of 10 (dashed), 50 (dashed-dotted) and 100 (continuous) years. Grey dots represent the values obtained from the Hybrid-based method, whereas red dots represent the values obtained from the continuous simulation.

Visually, there is a good agreement between the T_K estimated with for T_K of 10 years (Figure 4.12). However, for T_K of 50 and 100 years, the Hybrid method overestimates T_K in terms of volume, having a good agreement with peak-flows.

This can be explained by the analysis of the univariate distributions of maximum annual peak-flows (Figure 4.13a) and volumes (Figure 4.13b).

Figure 4.13: Univariate flood frequency distributions of maximum annual peak-flows (a) and volumes (b). Black dots represent the results obtained by the Hybrid-based method, whereas red dots represent the values obtained from the continuous simulation.

It can be seen that, in the case of peak-flows, there is an almost perfect agreement between the frequency curves of both the continuous and the Hybrid-based method (Figure 4.13a). When analysing the maximum annual volume frequency curve, it can be seen how the Hybrid-method underestimates the maximum annual volumes for return periods over 100 years (Figure 4.13b). In the case of the peak flow frequency curve, the Nash-Sutcliffe coefficient (Nash and Sutcliffe, 1970) has a value of 0.98, comparing the Hybrid-based values to those obtained by the continuous peak-flow; while in the case of the maximum annual volume frequency curve, the Nash-Sutcliffe coefficient is 0.97.

4.3. The Hybrid method: Summary and conclusions

4.3.1. Criterion for storm selection for the Hybrid method.

This Chapter approaches a focus on providing guidance on selecting the minimum number of storm events for deriving the flood frequency curve through event-based approaches. To do so, the methodology applied compares the resulting flow of coupling

a stochastic weather generator with a fully distributed physically-based model with the flow resulting from selecting only a limited number of storms per year. The methodology has only been applied to one basin, and therefore the results and conclusions are restricted to this watershed. Despite this, the study shows how the number of storms selected has an influence on the derivation of extreme floods, and its characteristics: Univariate magnitude and recurrence of maximum annual peak-flows and volumes, their seasonality, their dependence, and their statistic bivariate properties. Specifically, for Peacheater Creek:

1. When analysing the correspondence between storms and maximum annual peak-flow and volume, the best ranking criterion is total depth, followed by maximum intensity. Considering storms up to rank five sorted by the total depth criterion, resulted in a probability of generating the maximum annual peak-flow of 94% and the maximum annual volume of 99%.
2. In the case of the total depth criterion, the higher the return period of peak-flow or volume is, the lower the storm rank needed. For return periods of peak-flow or volume over ten years, all the storms have a rank equal or lower than four. For return periods higher than 50 years, there is a probability of achieving the maximum annual peak-flow and the maximum annual volume of 99% if the three highest events are considered.
3. For preserving the seasonality of maximum annual peak-flows and volumes, total depth was also the best criterion. With storms up to rank five, there is less than 1% absolute relative error in the estimation of the mean direction of seasonality for both the peak-flow and volume. Regarding the dispersion of the seasonality, considering storms up to rank five resulted in an absolute relative value lower than 4%.
4. Total depth was also the best criterion for preserving the dependence and bivariate properties of the floods derived from the continuous simulation. Even by considering only the biggest storm of each year, the absolute relative errors when estimating Spearman's rho and Kendall's tau are less than 1%. When estimating the parameter of the copula family selected (Gumbel), the relative error was less than 2% if storms up to rank five were considered, and was less than 0.5% if storms up to rank ten were considered. Regarding Kendall's return period, considering the same number of storms produces Kendall's return periods of 10 and 50 years very similar to those obtained by continuous simulation, and the 100 years return period is slightly

underestimated. If the ten biggest storms are considered, all the studied Kendall's return periods are almost equal.

Therefore, to sum up, as a general recommendation, considering the five biggest storms per year classified by their total depth can result in an accurate derivation of the flood frequency curve when using event-based models, close to that obtained from continuous simulations. Based on the results obtained, the five major events of each year in terms of total depth were selected from all the storm events within the 5,000 years to perform the Hybrid method simulations.

4.3.2. Initial soil moisture conditions and performance of the Hybrid Method

A Hybrid methodology has been presented. Through the use of an hourly climate stochastic generator and a physically based distributed hydrological model, the Hybrid method represents the frequency curves of peak flows and maximum annual volumes and its characteristics: Univariate magnitude and recurrence of maximum annual peak-flows and volumes, their seasonality, their dependence, and their statistic bivariate properties. The model represents these results in a precise way, reducing the computation times on the order from months to hours. This methodology has been applied to the Peacheater Creek case study, concluding that:

1. Independence between rainfall events and prior hydrological soil moisture conditions has been proved within the framework of the Hybrid method developed.
2. The Hybrid method preserves the seasonality of maximum annual peak-flows and volumes. The method provides less than 8% absolute relative error in the estimation of the mean direction of seasonality of both maximum annual peak-flows and volumes compared to the continuous modelling results. However, regarding the dispersion of seasonality, the Hybrid method provides less dispersion around the mean direction of seasonality, providing a relative error up to 52 % and 32 % in maximum annual peak-flows and volumes respectively.
3. The Hybrid method is able to preserve the dependence and bivariate properties of the floods derived from the continuous simulation. The correlation between maximum annual peak-flows and volumes is preserved by the Hybrid-based method similarly to the continuous modelling approach. Spearman's rho is obtained with a relative error lower than 7 %, whereas Kendall's tau differs around 13 % respect to the values

obtained through continuous modelling. When estimating the parameter of the Gumbel copula family selected, the relative error was less than 13%. Regarding Kendall's return period, Kendall's return period of 10 is very similar to those obtained by continuous simulation. However, for Kendall's return period of 50 and 100 years, the Hybrid method overestimates volume values, having a good agreement with peak-flows.

4. The proposed Hybrid method is capable of reproducing the univariate flood frequency curves with a good agreement to those obtained by the continuous simulation. The maximum annual peak-flow frequency curve is obtained with a Nash-Sutcliffe coefficient of 0.98, whereas the maximum annual volume frequency curve is obtained with a Nash-Sutcliffe value of 0.97.
5. The proposed Hybrid method reduces the time computation of continuous simulations from two and half months to fourteen hours of computation for Peacheater Creek basin (due to the continuous simulation of 50 years). Regarding to the event-based simulations, they were performed within one hour of computation.

5. Spillway operation: The K-Method

This Chapter presents a method which can be used to define real-time operation rules for gated spillways (named the K-Method). The K-Method is defined to improve the performance of the Volumetric Evaluation Method (VEM), by adapting it to the particular conditions of the basin, the reservoir, or the spillway. The VEM was proposed by the Spanish engineer Fernando Girón in 1988 (Girón, 1988) and is largely used for the specification of dam management rules during floods in Spain. This method states that outflows are lower than or equal to antecedent inflows, outflows increase when inflows increase, and the higher the reservoir level, the higher the percentage of outflow increase. The K-Method was developed by modifying the VEM and by including a K parameter which affects the released flows. A Monte Carlo environment was developed to evaluate the method under a wide range of inflow conditions (100,000 hydrographs) and with return periods up to 10,000 years.

5.1. Application to the Dam in the Segura river basin

The proposed methodology was applied to a dam located in the Segura river basin, the Talave reservoir. This dam is located in the province of Albacete, in the southeast of Spain, and belongs to the Mundo river basin. The Talave basin has an area of 766.5 km². The climate of the region is Mediterranean (mean annual precipitation of 557 mm). The main purposes of the reservoir are flood regulation, hydropower generation, and water supply for the Region of Murcia.

The characteristics of the basin and dam-reservoir system configuration are shown in Table 5.1 and Figure 5.1.

Table 5.1: Characteristic reservoir levels and outflows of the dam configuration.

Reservoir Levels (m.a.s.l)		Maximum Outflow Capacity at Design Flood Level (DFL) (m ³ /s)		Characteristic Outflows (m ³ /s)	
Top of control pool (TCP)	508.9	Gated-spillway	2×142.5	Alert outflow (O_{ALT})	100
Activation level (AL)	509.3				
Flood control level (FCL)	509.9	Bottom outlet	1×99.5	Warning outflow (O_{WARN})	150
Design flood level (DFL)	511.3	Dam body	2×9.0	Emergency outflow	300
Crest of dam (COD)	512.4	water intakes		(O_{EMER})	

Two vertical wagon gates, each one being six meters wide by four meters high, compose the main spillway. The other operative discharge structures are one bottom outlet and two water intakes in the dam body.

(a)

(b)

Figure 5.1: Case study scheme. (a) Location of the Talave dam. (b) Scheme of the discharge structures and the elevation of their axis (left). Details of gated spillways (right). Abbreviations are defined in Table 5.1.

Figure 5.2 shows a general scheme of the probabilistic approach.

Figure 5.2: Scheme of the proposed probabilistic approach.

A probabilistic approach was implemented, combining a Monte Carlo framework and deterministic flood control operation rules for dams with gated spillways. The main components of the process are:

- Rainfall generation and a hydrometeorological model. A Monte Carlo environment was produced to generate a set of storms and its corresponding hydrographs were used

as the input to the reservoir. An ensemble of 100,000 inflow hydrographs was generated by applying the methodology developed by [Sordo-Ward et al. \(2012\)](#) ([Section 3.1.3](#))

- Flood routing in the reservoir and dam operation by applying VEM, the K-Method, I-O, and MILP. The proposed K-method was compared with VEM, in order to assess the performance improvement. As exposed in [Section 3.2](#), two other methods were used for references as extreme cases: the Inflow-Outflow Method (I-O) and the Mixed Integer Linear Programming method (MILP) proposed by [Bianucci et al. \(2013\)](#). The I-O is a very simple scheme that tries to maintain the reservoir level by equating reservoir releases to reservoir inflows while the gates are partially opened, so no flood attenuation effect is achieved during the controlled phase of dam operation. On the other end, the MILP method is a full optimization method that achieves the best possible management of an inflow hydrograph, according to a given objective function. It represents perfect management under the unrealistic assumption of full knowledge of the inflow hydrograph, and thus, is taken as the reference for maximum improvement.

5.2. Performance assessment of the K-Method

Once the different gate controlled operation rules were applied, the maximum level reached in the reservoir (Z_{MAX}) and the maximum outflow downstream (O_{MAX}) were selected as representative variables for a comparison of the methods' performance.

A comparative analysis by means of two different approaches was implemented. On the one hand, a comparative scheme organized in quadrants was applied, which showed the increasing (or decreasing) of the maximum reservoir level and maximum outflow compared to the VEM. The points located in the upper right quadrant (Q_I) represent events where the maximum level reached in the reservoir and the maximum outflow were higher than by applying the VEM. The points located in the upper left quadrant (Q_{II}) show an intermediate situation, with lower maximum levels but higher outflows. The lower left quadrant (Q_{III}) represents cases for which both the maximum level and maximum outflow were lower than by applying the VEM (i.e., the best situation). The lower right quadrant (Q_{IV}) represents intermediate situations with higher maximum levels and lower maximum outflows. On the other hand, the global risk-index presented in [Section 3.6](#) was used as a comparative assessment tool.

Results comparing a representative range of K-values, I-O, and MILP with the VEM, are reported in Figure 5.3, Figure 5.4 and Figure 5.5. For K-values lower than one, no improvement with respect to the VEM was achieved for the Talave dam. The K-Method is equal to the VEM when the K-value is equal to one. For K-values higher than one, all analysed cases were located in Q_{II} and Q_{III} . Following this, the analysis of the K-Method focused on K-values higher or equal to one. When analysing the variations of the maximum reservoir level compared to VEM (ΔZ), a higher K-value resulted in a lower Z_{MAX} , regardless of the range of inflow Trs studied, reaching up to -0.15 m for $K = 2$ and -0.64 m for $K = 50$ (Figure 5.3). However, when analysing the variations of the maximum outflows compared to VEM (ΔO), different behaviours were found, depending on the Trs analysed. Figure 5.3 shows, for different ranges of Trs, the percentage of cases located in Q_{III} (the best situation), Figure 5.5 (a1–a6) shows the median and quartiles associated with the maximum reservoir levels (Z_{MAX}), and Figure 5.5 (b1–b6) shows the variations of the median and quartiles associated with the maximum outflows (O_{MAX}).

Figure 5.3: Comparison of the K-Method (for different K-values), I-O and MILP with respect to the VEM. The horizontal axis shows the increments of the maximum reservoir level in m (ΔZ). The vertical axis shows the increments of the maximum outflow in m^3/s (ΔO). The red points correspond to events with Tr ranging from one to 10 years, blue points ranging from 10 to 25, green points from 25 to 50, black points from 50 to 100, cyan points from 100 to 500, and magenta points from 500 to 10,000 years.

For Tr values lower than 25 years, all events were located in Q_{II} , regardless of the analysed K-value (Figure 5.4 (a1,a2)), decreasing the maximum levels but increasing the maximum outflows when compared with VEM (Figure 5.3 and Figure 5.5 (a1,a2,b1,b2)). Even though the maximum outflows increased, the O_{MAX} values did not jeopardize the

downstream safety for K-values lower than ten, because O_{ALT} was not exceeded ($O_{MAX} = 97.2 \text{ m}^3/\text{s}$ for $K = 10$).

Figure 5.4: Percentage of cases in Q_{III} from $K = 1.25$ to $K = 50$, for different ranges of Tr of the inflow hydrographs. The red colour corresponds to events with Tr ranging from one to 10 years (a1), blue from 10 to 25 (a2), green from 25 to 50 (a3), black from 50 to 100 (a4), cyan from 100 to 500 (a5), and magenta from 500 to 10,000 years (a6).

For Tr values ranging from 25 to 50 years, lower K-values included more cases in Q_{III} , decreasing from 73% for $K = 1.25$, to 4% for $K = 10$. For K-values higher than 20, all events were located in Q_{II} (Figure 5.4 (a3)). Higher K-values achieved lower Z_{MAX} values, but the opposite occurred for O_{MAX} . For example, the Z_{MAX} median decreased from 509.8 to 509.5 m and the O_{MAX} median increased from 111 to 118 m^3/s for $K = 1$ (VEM) and $K = 10$, respectively (Figure 5.5 (a3–b3)). For K-values higher than ten, the median of Z_{MAX} decreased to 509.3 m for $K = 50$, while the O_{MAX} median increased to 124 m^3/s for $K = 50$.

For Tr values ranging from 50 to 100 years, more than 99% of the analysed cases were included in Q_{III} for K-values from 1.25 to 6.5, more than 92% for K-values lower than ten, and this decreased down to 17% for $K = 50$ (Figure 5.4 (a4)). The median of Z_{MAX} decreased from 509.9 m for $K = 1$ (VEM), to 509.5 m for $K = 10$ and 509.3 m for $K = 50$. (Figure 5.5 (a4)). The median of O_{MAX} decreased from 177 m^3/s to 170 m^3/s with K ranging from one (VEM) to three, but increased for K-values higher than three, up to 180 m^3/s for $K = 50$ (Figure 5.5 (b4)).

For Tr values within the range of 100 to 500 years, all events were located in Q_{III} for K-values ranging from one to ten (Figure 5.3 and Figure 5.4 (a5)) and more than 93% for K-values higher than ten (Figure 5.4 (a5) and Figure 5.5 (a5,b5)). The Z_{MAX} and O_{MAX} median values decreased from 510.0 m and 270 m^3/s for $K = 1$ (VEM), to 509.6 m and 252 m^3/s for $K = 10$, respectively. For K-values higher than ten, the Z_{MAX} and O_{MAX} median values remained almost constant.

For Tr values higher than 500 years, regardless of the K-value (except for those lower than one), all events were located in Q_{III} (Figure 5.3 and Figure 5.4 (a6)). The Z_{MAX} and O_{MAX} median values ranged from 511.0 m and 375 m³/s for K = 1 (VEM), to 510.8 m and 360 m³/s for K = 10, respectively (Figure 5.5 (a6,b6)). For K-values higher than ten, the Z_{MAX} and O_{MAX} median values remained almost constant. I-O and MILP were also compared with VEM. As expected, the I-O behaviour was similar to the K-Method for higher K-values (Figure 5.3). By comparing MILP and VEM, the results showed that all events with a Tr higher than 25 years were located in Q_{III}, and those with a Tr lower than 25 years located in Q_{II} presented outflows that did not jeopardize the downstream safety (Figure 5.3).

Figure 5.5: Comparison of the K-Method (for different K-values) with respect to the VEM (K = 1). The continuous line shows the median and the shaded areas show the range between the 25th and 75th percentile of the maximum reservoir levels (Z_{MAX}) (a) and the maximum outflows (O_{MAX}) (b), from K = 1 (VEM) to K = 50. The horizontal axis shows the different values of K. The red colour corresponds to events with Tr ranging from one to 10 years (a1,b1), blue from 10 to 25 (a2,b2), green from 25 to 50 (a3,b3), black from 50 to 100 (a4,b4), cyan from 100 to 500 (a5,b5), and magenta from 500 to 10,000 years (a6,b6). For specific K-values (K = 1, 2, 4, 6, 8, 10, 20, 50), box plots are presented with their corresponding outliers (when existing) represented by red dots.

An analysis of the frequency curves of Z_{MAX} and O_{MAX} for the different methods (Figure 5.6) was also carried out. For the K-values analysed (and for K-values greater than one), they showed an intermediate behaviour between VEM and I-O. For the Z_{MAX} frequency curves, the higher the K-value, the lower Z_{MAX} obtained for the same return period (Figure 5.6a). However, when analysing the O_{MAX} frequency curves, they

intersected the VEM maximum outflow frequency curve (Figure 5.6b). For Tr values higher than one, and until the intersection of the O_{MAX} frequency curves, the greater the K -value, the greater the O_{MAX} was for the same Tr . Moreover, the intersection with VEM occurred at higher Tr s, corresponding to a Tr of 31, 50, 86, and 102 years for $K = 2, 10, 50$, and I-O, respectively. Once the O_{MAX} frequency curves intersected, the behaviour was the opposite.

Therefore, the choice of the best operation rule was not univocal for the entire range of analysed Tr and depended on the priorities selected by the dam damager (MILP was not accounted for as it needs to know the whole inflow hydrograph in advance). If an ordinary reservoir operation and the management of floods with a Tr between one and 25 years were prioritized, the best rule operation was the K-Method for the highest K -value that minimized Z_{MAX} and avoided damages associated with the released outflows. As mentioned, for this case study, it was $K = 10$.

Figure 5.6: Frequency curves. (a) Maximum reservoir level frequency curves for VEM (blue continuous line), different values of K (black lines of different types, see legend), I-O (dashed-dotted blue line), and MILP (dashed red line). (b) Maximum outflows frequency curves for VEM (blue continuous line), different values of K (black lines of different types, see legend), I-O (dashed-dotted blue line), and MILP (dashed red line).

If the management of floods with a Tr ranging from 25 to 500 years was prioritized, the best operation rule depended on whether the number of events included in Q_{III} or the entity of the decrease of Z_{MAX} and O_{MAX} was prioritized. In the case study, $K = 1.25$ maximized the number of events improved for $25 < Tr < 50$ years and $K = 10$ for $50 < Tr < 500$ years. The reservoir levels reached Zones 3 (in all events with Tr values higher than 50 years) and 4. The influence of the K -value on Equation (3.13) implied an increase of the released outflows at the beginning of the operation (Zone 2), increasing the available flood control capacity with respect to VEM for the next time steps. Therefore, Zones 3 and 4 were reached later, for which the O_{MAX} was reduced (Figure 3.8).

If the dam safety and management of extreme floods ($Tr > 500$ years) were prioritized, I-O simultaneously minimized the maximum levels and maximum outflows. However, as the Tr of the inflows increased, ΔO and ΔZ tended to be zero. In these cases, Q_p was higher than the maximum discharge capacity when the gates were fully opened and the outflow responded to an un-controlled fixed-crested spillway (Figure 3.8). This explains why Z_{MAX} and O_{MAX} tended to be constant when analysing K-values higher than ten for Tr values higher than 500 years (Figure 5.5 (b6) and Figure 5.6b). This also explains the shape of the half loop of the comparative scheme between the K-Method and VEM (Figure 5.3).

In a practical case, even if the skill required to forecast the inflow hydrograph is limited, there may be ways to identify the entity of the flood that is being managed in the reservoir. According to the general meteorological situation, the dam manager may be able to estimate the return period of the event that is occurring, and therefore, the conclusions of the above discussion could be applied to decide on the best K-value to adopt.

Finally, Figure 5.6b shows changes in the slope for both the Z_{MAX} and O_{MAX} frequency curves, and regardless of the analysed operation rule. For small and medium floods, the outflows are governed by the operation of the partially opened gates. Once the FCL is exceeded and gates become fully open, the spillways work as un-controlled fixed-crested spillways, justifying the behavior of the frequency curves.

Figure 5.7 shows the evolution of the ratio between the maximum outflow and the maximum inflow as a function of the return period of each event for different dam operation methods. Although MILP showed the most peak attenuation, as previously stated, it represents perfect management under the unrealistic assumption of full knowledge of the inflow hydrograph; thus, it was taken only as the reference for maximum improvement. Regarding the behaviors of the real-time methods, three main phases were distinguished. For a lower Tr , when all methods were active (gates partially opened), the VEM was the real-time method which showed the highest peak attenuation, because the other real-time methods specify greater outflows than those of VEM. In addition, the other methods reached the fully opened gate situation earlier than VEM, because they released greater outflows and the spillways started to work earlier as un-controlled fixed-crested spillways. For medium and high Tr values, the mentioned change in the release mechanism justifies an increase of peak attenuation and the differences

among the methods decrease. Finally, for the biggest floods where the crest of the dam is overtopped and the entire length of the dam works as an un-controlled fixed-crested spillway, peak attenuation decreased, regardless of the method.

Figure 5.7: Peak flow attenuation for different return periods and methods of operation of dams. Y axes represent the ratio between the maximum outflow and the maximum inflow for each event with their corresponding Tr (X axes). Blue dots represent the events routed by the dam by applying the VEM, dark grey dots by applying the K-Method with a K -value = 2, grey dots with a K -value = 10, light grey dots with a K -value = 50, cyan dots by applying the I-O method, and red dots by applying the MILP.

Figure 5.8 shows two examples of application of the different proposed dam operation methods. Although MILP showed the most attenuation of peak flows for both cases and good behaviour regarding the maximum reservoir level, it represents perfect management under the unrealistic assumption of full knowledge of the inflow hydrograph. Therefore, it was only taken as the reference for maximum improvement. For a moderate flood, Figure 11a shows considerable differences of maximum peak flows and maximum reservoir levels, and the VEM was the real-time method which presented the highest peak attenuation, but the highest maximum reservoir level. In the analysis of a large flood ($Tr = 1,000$ years, Figure 5.8b), it can be seen that the gates became fully open (around 20 h) for all methods. From this time on, the spillways start to work as un-controlled fixed-crested spillways. The peak attenuation effect is mainly due to the un-controlled fixed-crested spillway mechanism and the differences among the methods decrease.

Figure 5.8: Comparison of outflow hydrographs (top) and reservoir levels (bottom) resulting from the application of different dam operation methods. (a) Hydrograph corresponding to $Tr = 20$ years and (b) corresponding to $Tr = 1,000$ years. Colored lines represent the application of the different methods: VEM (blue continuous line), different values of K (red lines of different types, see legend), I-O (dashed-dotted blue line), and MILP (magenta continuous line). Coloured shading (bottom) represents the different Zones of application of the K-Method (Zone 1 in green, Zone 2 in yellow, Zone 3 in orange, and Zone 4 in red).

In order to better quantify the influence of the K -value, the global risk index was applied. Using the damage curves (Figure 5.9a,b), the failure risk index (I_F), the non-failure risk index (I_{NF}), and the global risk index (I_R) for the Talave dam were obtained (Figure 5.9c).

Figure 5.9 shows that K -values larger than one reduce the I_R value for the Talave dam with respect to the VEM. There is also a range of K -values for which the I_R is even lower than for the I-O, which is a very conservative flood management strategy. This global reduction is due to the effect of management strategies on the I_{NF} . If I_{NF} is analysed, I-O is found to be the worst strategy, since it implies no peak attenuation during the controlled phase of the flood. The K-Method outperforms the VEM for K -values smaller than 20. For the case of the I_F , the minimum values are obtained for the I-O strategy, which emphasizes controlling the reservoir level. This strategy is even better than MILP optimization. In the case of the K-Method, it always outperforms VEM, and I_F tends to be the value obtained for I-O for large values of K . MILP represented the minimum expected value for I_R .

Figure 5.9: Risk-based approach. (a) Expected Talave Dam damage cost vs. released maximum reservoir level. (b) Expected damage cost downstream Talave Dam vs. released maximum outflows. (c) Risk indexes for VEM (dashed lines), I-O (dotted lines), MILP (dashed-dotted lines), and different values of the K-Method (continuous lines). Blue lines show the failure risk index (I_F), red lines show the non-failure risk index (I_{NF}), and black lines show the global risk index (I_R). Horizontal axis shows the different values of K. The optimum K for I_R is 5.25.

For this case study, the K-value that optimized I_R is 5.25, reducing the annual expected damage by 8.4%, when compared to VEM. The reduction represented 17.3% of the maximum possible reduction determined by MILP. Even though this may seem like a small reduction, it should be taken into account that the application of the K-Method has no associated construction costs and the improvement would be applied annually, during the whole dam life (according to the Dam Master Plan, Talave Dam's expected life is 167 years).

5.3. The K-Method: Summary and conclusions

The proposed K-Method is a decision-making tool for dam managers that allows analyzing the effects of applying different reservoir operation rules for both dam safety and downstream safety. In contrast with other fixed methods, the K-Method allows, for each analysed dam, the identification of the K-value (conducting a calibration process) that best adapts the management strategy to the specific conditions of the basin, the reservoir, the spillway, and to the objectives of flood management. It should be noted that the proposed method has been applied to only one basin and dam configuration, which

limits the generalization of the results and conclusions obtained. Although the results are promising, further research should be developed to ensure that the K-method improves the operation of flood control rules through a wide range of gated-spillway dams.

For the case study under analysis, the K-Method improved the results obtained with VEM, by reducing the maximum reservoir levels reached in the dam (the higher the K-value, the lower the maximum reservoir level), while the corresponding increase of outflows did not endanger downstream safety. In addition, by carrying out a dam risk analysis, a K-value of 5.25 lowered the global risk index, I_R , by 8.4% compared to VEM, and represented 17.3% of the maximum possible reduction determined by MILP.

Different behaviors were identified, depending on the Tr s analysed. For events with a Tr ranging from one to 25 years, K-value = 10 resulted in an important decrease in the maximum reservoir levels, while the corresponding increase of outflows did not endanger downstream safety.

For events with a Tr ranging from 25 to 100 years, higher K-values reduced the number of events that simultaneously achieved lower maximum levels and outflows. For events with a Tr higher than 500 years, I-O simultaneously achieved the lowest maximum levels and outflows. In a practical case, even if the skill required to forecast the inflow hydrograph is limited, according to the general meteorological situation, the dam manager may be able to estimate the return period of the event that is occurring and therefore the conclusions obtained could be applied to decide on the best K-value to adopt.

6. Operational variables I: The Influence of the initial reservoir level.

This Chapter presents a method to assess the influence of the initial reservoir level in hydrological dam safety and risk analysis. Traditionally, in professional practice, the procedures applied are basically deterministic. Several physical processes are defined deterministically according to the criteria of the designer (usually in the conservative side), although there is a high degree of uncertainty regarding to these processes. A relevant variable is the reservoir level considered at the beginning of flood events. Hydrological dam safety assessment methods traditionally assume that the reservoir is initially full when it receives the design flood, thus staying in the conservative side when designing a new dam. However, the distribution of reservoir levels at the beginning of flood episodes takes more importance for evaluating the real risk associated to dams in operation.

In this chapter, an analysis of three different scenarios is carried out: initial reservoir level equal to maximum normal level, equal to a maximum conservation level and following the probability distribution from the historical records. To do so, a methodology is proposed to analyse economically the affection of initial reservoir level in hydrological dam and downstream safety by the application of the global risk index (Bianucci et al., 2013). Furthermore, the uncertainty associated to initial variable level is studied in a novel approach for the case study.

6.1. Application to the Dam in the Tagus river basin

A probabilistic approach was implemented (Figure 6.1), based on a Monte Carlo approach (dotted points in Figure 6.1) to analyse a Dam in the Tagus river basin. The process was as follows:

1. Generation of synthetic inflow hydrographs. An ensemble of 100,000 synthetic inflow hydrographs representative of the observed historical annual maximum floods was generated, applying the methodology exposed in Section 3.1.2.
2. Stochastic initial reservoir level assignation. Depending on the scenario studied (a total of three, as will be explained in Section 6.1.1), an initial reservoir level was

assigned to each inflow hydrograph. Furthermore, an uncertainty analysis of the variable initial reservoir level (Scenario 3) was carried out.

3. Reservoir-Dam flood operation. For each analysed scenario (total of three), the ensemble of hydrographs was routed through the reservoir, obtaining a set of maximum reservoir levels and maximum outflows. The methodology applied was the VEM (Section 3.2.1).
4. Risk-Index analysis. By using the global risk index analysis (Bianucci et al., 2013) explained in Section 3.6, all the scenarios studied were compared.

Figure 6.1: General scheme of the methodological approach. Sc.1, Sc.2 and Sc.3 stand for the three different scenarios of initial reservoir level (Z_o) studied.

The proposed methodology was applied in a case study based on a gated-spillway gravity dam located in the Tagus river basin. The dam is located in the west of Spain (province of Caceres). The dam watershed has an extension of 1,850 km². The climate within the area of study is Continental with mean annual precipitation of 1,000 mm and a mean annual runoff value of 1,020 hm³. The main purpose of the reservoir is irrigation.

Table 6.1 shows the main characteristics of the dam and its reservoir. The dam main spillway consists of five Tainter gates, which are ten meters wide by 5.75 meters high each. The other operative discharge structure considered is a bottom outlet in the dam body.

To carry out the study, a gauge station located at the dam location with available data of daily inflows and reservoir volumes from 1958/59 to 2012/13 (hydrological years,

from October 1st to September 30th) was used. With these data, it was possible to generate the synthetic inflow hydrographs and obtain the statistic distributions of initial reservoir levels for Sc.3.

Table 6.1: Characteristic reservoir levels and spillway of the dam configuration studied.

Reservoir Levels (m.a.s.l)		Maximum Outflow Capacity at Maximum Normal Level (MNL) (m ³ /s)	
Gated spillway crest	380.25	Gated-spillway	2,200
Maximum Normal Level (MNL)	386		
Design flood level (DFL)	387	Bottom outlet	57
Crest of dam (COD)	388		

6.1.1. Generation of synthetic reservoir inflow hydrographs

A set of 100,000 reservoir inflow hydrographs was generated stochastically using the available 55 years of daily reservoir inflow data (1958/59–2012/13) applying the methodology of [Section 3.1.2](#). First, the empirical probability distribution of maximum annual flood durations was obtained. The durations were within the range of three to nine days, being five and six days the most frequent values (comprising 47.3 % of the observed values). With a Monte Carlo procedure, a set of 100,000 hydrograph durations was generated ([Figure 6.2a](#)).

Afterwards, the volume frequency curves (VFCs) were calculated, each one associated to one duration (from three to nine days). They were fitted to a GEV distribution with estimated parameters using L-Moments technique ([Hosking, 1990](#)). All fitted statistic distributions passed the Kolmogorov-Smirnov goodness of fit test. [Figure 6.2b](#) shows the fits (P-Values were within the range from 0.997 to 0.999 for all durations). Afterwards, the 100,000 hydrographs were generated as exposed in [Section 3.1.2](#). To assure the validity of the sample generated, the synthetic peak flow frequency curve (PFFC) was obtained in order to compare it with the synthetic generated one (at an hourly time step). The instantaneous observed peak flows were estimated using the equation recommended in literature for the Tagus river basin area ([MARM, 2011](#)) (Equation (6.1)):

$$\text{IMF} = \text{MDF} \cdot (1 + 5.01 \cdot A^{-0.38}), \quad (6.1)$$

Where IMF and MDF are the instantaneous maximum flow and the maximum annual mean daily flow respectively (m³/s) and A is the watershed area in km² ([MARM, 2011](#)). The pseudo-observed instantaneous PFFC was also fitted to a GEV distribution using L-Moments technique ([Hosking, 1990](#)), which also passed Kolmogorov-Smirnov

goodness of fit test with a P-Value of 0.997. Afterwards, by using Gringorten plotting position formula (Gringorten, 1963), the 100,000 peak flows (synthetic PFFC) of the sample generated were compared with the pseudo-observed PFFC (Figure 6.2c). Finally, Figure 6.2d shows the peak flow volume relationship between the observed and generated sample, showing how the synthetic events properly represent the main hydrograph characteristics.

Figure 6.2: Synthetic inflow hydrograph generation. (a) Frequency distribution of observed (blue) and simulated (red) reservoir inflow hydrograph duration (b) GEV fits using L-Moments (L-MOM) technique of maximum annual hydrograph volume for the different observed durations (blue lines) and simulated volumes (red dots). (c) GEV fits using L-Moments (L-MOM) technique of the maximum annual instantaneous peak flows. Blue circles represent the maximum annual instantaneous peak flow with return periods obtained by applying Gringorten plotting position formula, blue continuous line represents GEV L-MOM fit, blue dashed lines represent the 99 % confidence bounds whereas the red dots represent the maximum peak flows generated with a return period obtained by applying Gringorten plotting position formula. (d) Sample of 100,000 simulated floods (red dots) plotted with the observed floods (blue-edged circles).

6.1.2. Initial reservoir level. Scenarios and uncertainty analysis

In order to assess initial reservoir level, a previous analysis of the observed data was carried out:

1. First, the months in which annual maximum mean daily flow occurred (from the hydrological year of 1958/59 to 2012/13) were identified. Once they were identified, the probability distribution of occurrence of monthly annual floods was obtained (Figure 6.3a).
2. Afterwards, an analysis of the daily reservoir volumes was performed to obtain the initial reservoir volume frequency distributions. Reservoir data from 1958/59 to 1965/1966 was discarded, as those years the reservoir was filling-up and did not represent normal operation years. It was also checked that no significant changes in the reservoir operation strategy happened during the analysed period (1966/67 to 2012/13). To do so, the cumulative monthly supply, cumulative monthly storage and cumulative monthly ratio of supply over storage were calculated (Figure 6.3b, from left to right). It is shown that the relations are almost linear, being the small variations due to the natural variability of inflows.
3. Finally, the cumulative frequency distribution of exceedance of the monthly daily reservoir levels using the reservoir level time series from 1966/67 to 2012/13 was obtained (Figure 6.3c).

Afterwards, three different scenarios were studied:

- Scenario 1 (Sc.1): For all the 100,000 hydrographs, initial reservoir level was set equal to Maximum Normal Level (MNL). MNL is a constant level, and refers to the maximum acceptable level in the reservoir under ordinary operation. This reservoir level is set in the design phase of the dam. For all the 100,000 hydrographs, initial reservoir level was set at MNL (386 m.a.s.l.) and corresponds to a volume in the reservoir of 924 hm^3 (vertical black continuous line, in Figure 6.3c).
- Scenario 2 (Sc.2): For all the 100,000 hydrographs, initial reservoir level was set equal to the Flood Control Level (FCL). FCL is the level which cannot be overpassed under ordinary operation of the reservoir (prior to a flood event), assuring that the design flood does not compromise the hydrological dam safety. This level is defined in this thesis as the initial reservoir level that, after routing the set of 100,000 synthetic flood events through the reservoir, results into a maximum water reservoir level of return period 1,000 years ($\text{MWRL}_{\text{TR}=1000\text{y}}$) which is equal to the Design Flood Level (DFL), fulfilling the regulation standards. FCL is always lower or equal to the MNL, as it is usually defined to fulfil with regulation standards that are set in the years after the construction of the dam. For all the 100,000 hydrographs, initial reservoir level was

set at FCL (384 m.a.s.l.) and corresponds to a volume in the reservoir of 841.9 hm³ (black vertical dashed line in [Figure 6.3c](#)).

- Scenario 3 (Sc.3): First, each one of the 100,000 hydrographs was associated to a month of occurrence using the empirical distribution of months of occurrence of historical annual maximum floods. Afterwards, for each one of the 100,000 hydrographs, a variable initial reservoir level was assigned. To do so, first an analysis of daily reservoir levels measurements in the reservoir was carried out. Afterwards, the period of filling-up of the reservoir was removed as it does not represent the normal operation of the reservoir ([SPANCOLD, 2012](#)). Afterwards, the empirical monthly distribution of historical reservoir levels was obtained. Using this distribution, a set of 100,000 initial reservoir levels was generated, each one associated to each month in the aforementioned series of 100,000 months, and, consequently, to each one of the 100,000 inflow hydrographs. For each one of the 100,000 hydrographs, a variable initial reservoir level was assigned within a Monte Carlo framework and a set of 100,000 initial reservoir levels were generated.

Finally, in order to assess the uncertainty associated to initial reservoir level, a set of 1,000 simulations of Sc. 3 was carried out within a Monte Carlo framework ([Figure 6.1](#)), obtaining 1,000 frequency distributions of monthly initial reservoir volumes (in red in [Figure 6.3c](#)). This way, it is evaluated how the variability of this variable affects hydrological dam and downstream safety.

When analysing [Figure 6.3c](#) it can be seen how the annual fluctuation of the reservoir reflects in the cumulative probability frequency curves. The reservoir presents the lowest levels in October (discarding from June to September, months in which no floods were registered), due to the end of irrigation season. This operation policy might be of importance when analysing the influence of initial reservoir level, as pointed in by ([Carvajal et al., 2009](#)).

Figure 6.3: Initial reservoir level assessment. (a) Occurrence probability of monthly maximum annual flood. (b) Cumulative monthly supply, storage and supply over storage values. The whole period is represented in grey (1958/59 to 2012/13), whereas the selected period is represented in red (1966/67 to 2012/13) (c) Empirical monthly reservoir level distributions associated to each month in (a). Different color lines (red, green, blue and cyan) represented the observed (Obs.) reservoir level distribution. Grey lines represent the 1,000 simulations of monthly initial reservoir levels distributions for Scenario 3. Scenario 1 is represented in a black continuous line, whereas Scenario 2 is represented in a dashed black line.

6.1.3. Reservoir-Dam flood operation

For each generated hydrograph (100,000 incoming floods) and scenario, the operation of the dam gates applying the Volumetric Evaluation Method (VEM, [Section 3.2.1](#)) was simulated. The initial reservoir level depended on the scenario studied. After applying the VEM to all scenarios, the following results were obtained:

1. Scenario 1 and Scenario 2: For each scenario a set of 100,000 reservoir level and 100,000 outflow flood control time series is obtained, from which derivation of the Maximum Water Reservoir Levels (MWRL) and Maximum Outflows (MO) frequency curves can be performed.
2. Scenario 3: In this scenario, due to the uncertainty assessment, a set of 1,000 MWRL and MO frequency curves is obtained, derived from 1,000 sets of 100,000 reservoir level and outflow time series.

Therefore, 100,000 MWRLs and 100,000 MOs are obtained for Sc.1 (red in [Figure 6.4](#)) and Sc. 2 (green lines in [Figure 6.4](#)), whereas 1,000 sets (to analyse uncertainty related to initial variable reservoir level) of 100,000 MWRLs and 100,000 MOs are obtained for Sc. 3 (each one of the 1,000 sets represented in grey within [Figure 6.4](#), whereas the mean of the 100,000 simulations is represented in a blue continuous line and the 2.5 % and 97.5 % percentiles are represented in blue dashed lines).

Figure 6.4: Maximum water reservoir level (MWRL) and maximum outflow (MO) frequency curves. (a) Red line represents the MWRL frequency curve assuming initial level equal to MNL (Scenario 1). Green line represents the MWRL frequency curve assuming initial reservoir level equal to FCL (Scenario 2). Grey lines represent the 1,000 MWRL frequency curves assuming variable initial level (Scenario 3), being the median of these simulations represented by a blue continuous line and the 2.5 % and 97.5 % percentiles with blue dashed-lines. Dotted grey line represents MNL, dashed grey line represents the DFL, continuous grey line represents COD. (b) Same as figure (a) but applied to Maximum Outflow.

The most adverse situation regarding to hydrological dam and downstream safety is Sc.1 ([Figure 6.4](#)). DFL is reached at a return period of 751 years, with an initial

reservoir level of 386 m.a.s.l.. Setting FCL at 384 m.a.s.l. (Sc. 2), the DFL is reached at a return period of 1,000 years. Regarding to Sc. 3, the DFL is reached by the median MWRL frequency curve at 5,647 years, whereas the 1,000 frequency curves reach DFL between 3,710 and 9,470 years. Overtopping occurs at 4,427 and 5,840 years in Scenario 1 and 2 respectively, whereas no overtopping occurs up to 10,000 years of return period in Scenario 3.

When analysing MO frequency curves, the maximum capacity at MNL ($2,428 \text{ m}^3/\text{s}$) is reached at a return period of 87 years in Sc.1, 111 years in Sc.2 and 831 years for the median of the 1,000 frequency curves. The 1,000 frequency curves reach $2,428 \text{ m}^3/\text{s}$ between 709 and 1,092 years.

In order to evaluate how the initial reservoir level affects to hydrological dam and downstream safety in terms of return period, an indicator is proposed to analyse how initial variable level affects to hydrological dam and downstream safety. This indicator represents the return period which corresponds to the same MWRL or MO that has a return period of 1,000 years if MNL is assumed as initial reservoir level (Sc. 1), represented as Tr_{VAR} . In the case study, for the median MWRL frequency curve Tr_{VAR} was equal to 7,063 years whereas for the MO frequency curve Tr_{VAR} was equal to 7,064 years. This behaviour is similar to the one obtained in the Dam of the Douro river basin of [Chapter 7](#) and to the ones obtained in previous works ([Carvajal et al., 2009](#); [Aranda Domingo, 2014](#)), in which this indicator went from 3,200 to 9,000 years for reservoirs which main purpose is irrigation.

It should be highlighted that, if uncertainty due to initial reservoir level is evaluated, it can be seen how this indicator varies. Tr_{VAR} ranges from a minimum of 4,902 years to a maximum of 12,320 years, being the same minimum and maximum value in the case study in both MWRLs and MOs. This procedure is helpful to have an order of magnitude of how initial reservoir level can influence on hydrological dam and downstream safety, but uncertainty should be accounted for more detailed results. The value of this indicator compared to other studies in literature will be discussed in the following chapter, coupled with the results obtained in the analysis of initial reservoir level in the Dam of the Douro river basin.

To analyse the uncertainty associated to initial reservoir level an analysis similar the one proposed by [Flores-Montoya et al. \(2016\)](#) was carried out. The different values associated with a given return period of MWRL were analysed ([Table 6.2](#) represented as

the height water above the Flood Control Level (m) for the sake of clarity) and MO (Table 6.3) obtaining the 25 % percentile (Q1), the 50 % percentile (Q2 or median) and the 75 % percentile (Q3) quartiles. Moreover, in order to quantify the uncertainty of the estimations, the interquartile range (IQR) is calculated. The IQR is the difference between the upper and lower quartiles and permits to measure statistical dispersion.

Table 6.2: Values of the first (Q1), second (Q2), third (Q3) quartiles, the interquartile range IQR, and the relative value of the IQR against Q2 for different return periods (Tr) of maximum water reservoir levels. Values are represented as the water height above the Flood Control Level.

Tr (years)	Q1 (m)	Q2 (m)	Q3 (m)	IQR (m)	IQR/Q2 (%)
100	1.748	1.753	1.757	0.009	0.5
1,000	2.050	2.075	2.093	0.043	2.1
5,000	2.865	2.925	2.989	0.123	4.2
10,000	3.222	3.340	3.424	0.202	6.0

Table 6.3: Values of the first (Q1), second (Q2), third (Q3) quartiles, the interquartile range IQR, and the relative value of the IQR against Q2 for different return periods (Tr) of maximum outflows.

Tr (years)	Q1 (m ³ /s)	Q2 (m ³ /s)	Q3 (m ³ /s)	IQR (m ³ /s)	IQR/Q2 (%)
100	1,200.3	1,212.8	1,222.8	22.5	1.9
1,000	2,455.8	2,472.5	2,483.3	27.5	1.1
5,000	2,949.5	2,986.4	3,025.4	76.0	2.5
10,000	3,174.6	3,250.1	3,303.6	129	4.0

It can be seen how the uncertainty increases considerably after the return period of 1,000 years (Figure 6.4, Table 6.2 and Table 6.3). This behaviour can be related to the configuration of spillway studied. Gated-spillway dams are designed with a maximum outflow capacity. When the outflow capacity corresponding to MNL is reached, VEM is not able to maintain the target reservoir level by operating the gates, gates open and floods cannot be controlled, which results into more variability. Furthermore, this variability is possibly related to the size of the sample of synthetic inflow hydrographs used (100,000), as shown by other authors when studying stochastic approaches for deriving flood frequency curves (Loukas et al., 1996; Loukas, 2002; Blazkova and Beven, 2004). The affection of initial reservoir level for MWRLs and MOs frequency curves have a small uncertainty ($IQR/Q2 < 2.1$ %) for return periods below 1,000 years, by using 100,000 inflow hydrographs and 100,000 initial reservoir levels.

6.1.4. Risk Index analysis

Finally, the influence of initial reservoir variability in dam and downstream safety was quantified by applying the global risk index procedure (Bianucci et al. (2013)). To do so, the damage cost curves D_{MWRL} (Figure 6.5a) and D_{MO} (Figure 6.5b) were obtained. By applying the procedure explained Section 3.6, the I_F , I_{NF} and I_R values for the different scenarios were calculated. Figure 6.5c presents the values I_F , I_{NF} and I_R associated to Sc.1 and Sc.2, and the median values of the 1,000 simulations of Sc.3. Regarding to the different risk index values obtained:

- Failure risk index (I_F): In the case of Sc.1, I_F had a value of 526.4×10^3 euros, whereas in Sc.2 I_F reduced to 242.6×10^3 euros. Regarding to Sc. 3, the median value of I_F in the 1,000 simulations was 85.2×10^3 euros, with values ranging from 77.6×10^3 euros to 99.0×10^3 euros (minimum and maximum value respectively). Therefore, I_F reduces its value up to 83.8% from Sc. 1 to Sc. 3, and 64.8 % from Sc.2 to Sc.1.
- Non failure risk index (I_{NF}): In the case of Sc.1, I_{NF} had a value of $1,442.6 \times 10^3$ euros, whereas in Sc.2 I_{NF} reduced to 978.1×10^3 euros. Regarding to Sc. 3, the median value of I_{NF} in the 1,000 simulations was 89.8×10^3 euros, with values ranging from 83.6×10^3 euros to 94.0×10^3 euros (minimum and maximum value respectively). Therefore, I_{NF} reduces its value up to 94 % from Sc. 1 to Sc. 3, and 91 % from Sc.2 to Sc.1.
- Global risk index (I_R): In the case of Sc.1 I_R had a value of $1,968.9 \times 10^3$ euros, whereas in Sc.2 I_R reduced to $1,220.7 \times 10^3$ euros. Regarding to Sc. 3, the median value of I_R in the 1,000 simulations was 175.1×10^3 euros, with values ranging from 163.7×10^3 euros to 191.6×10^3 euros (minimum and maximum value respectively). Therefore, I_R reduces its value up to 91 % from Sc. 1 to Sc. 3, and 86 % from Sc.2 to Sc.1.

The uncertainty related to I_F , I_{NF} and I_R was calculated with a similar procedure as the one exposed in Section 6.1.3, shown in Table 6.4. There is an uncertainty of 6.6 %, 3.2 % and 4.2 % associated to I_F , I_{NF} and I_R respectively, regarding to initial reservoir level. The value of uncertainty of I_F is higher than the uncertainty associated to I_{NF} . When evaluating I_F , the MWRLs values that have more importance for the risk index are those related to extreme return periods (as can be seen in Figure 6.5a, there is no risk of damage below MNL). Therefore, the part of the MWRL frequency curve (Figure 6.4a) which is affecting the index in Sc.3 is the one related to return periods over 1,000 years, in which

uncertainty increases (Table 6.2). However, when evaluating I_{NF} , it can be seen that damage (Figure 6.4b) starts for values which correspond to medium return periods in Sc.3 (from 40 years of return period (Figure 6.4b)). As these values have less uncertainty (Table 6.3) and a higher value of exceedance probability (Figure 6.4b) than those affecting to I_F (Figure 6.4a and Table 6.2), uncertainty results higher in I_F than I_{NF} , being the uncertainty of I_R between both values (consequence of being the result of summing both I_F and I_{NF}).

Figure 6.5: Damage cost curves. (a) Damage cost curves associated with maximum water reservoir levels. (b) Damage cost curves associated with maximum outflows. (c) Values of dam failure (I_F , red bars), non-failure (I_{NF} , blue bars) and global (I_R , black bars) risk index for each of the three scenarios. Colored lines are added for a better comparison between scenarios and risk indexes.

Table 6.4: Values of the minimum, maximum, first (Q1), second (Q2), third (Q3) quartiles, the interquartile range IQR, and the relative value of the IQR against Q2 for the dam failure (I_F), non-failure (I_{NF}) and global (I_R) risk index.

Risk Index	Minimum (10 ³ €)	Maximum (10 ³ €)	Q1 (10 ³ €)	Q2 (10 ³ €)	Q3 (10 ³ €)	IQR (10 ³ €)	IQR/Q2 (%)
I_F	77.6	99.0	82.5	85.2	88.2	5.6	6.6
I_{NF}	83.6	94.0	88.3	89.8	91.1	2.8	3.2
I_R	163.7	191.6	171.2	175.1	178.6	7.4	4.2

It is important to point out that other aspects that may have impact in the risk index analysis were not considered, such as the probability of failure due to blockage or malfunction of gates. This would have resulted in higher values of MWRLs for lower return periods, thus resulting into higher values of I_{NF} for all the scenarios. Furthermore, this analysis could be complemented with studies of human loss life with societal risk indexes (Fluixá-Sanmartín et al (2018), SPANCOLD (2012)) for a full comprehensive risk analysis.

6.2. Influence of initial reservoir level: partial conclusions

The proposed methodology permits to assess through a stochastic procedure the influence of initial reservoir level, the uncertainty related to this variable and its influence in economic risk indexes. The results obtained showed:

1. For the case study, considering the fluctuation of initial reservoir level provided a more realistic assessment of hydrological dam and downstream safety. When a conventional approach was used (initial reservoir level equal to Maximum Normal Level), the Design Flood Level was reached at a return period of 751 years, which does not fulfil therefore with the regulation standards (1,000 years). However, when initial reservoir level was accounted, the results showed that the Design Flood Level was not reached in any of the set of 100,000 simulations carried out. When analysing a portfolio of different reservoirs, a dam that may seem safer than other at first, may not be if this variable is included in the analysis (for example, in an extreme case, comparing a hydroelectric dam, which is usually full, with a flood control reservoir, which is usually empty). Thus, this methodology can help stakeholders when carrying out decisions about prioritizing their investments.
2. The influence of initial variable reservoir level in maximum reservoir water level and maximum outflows frequency curves in the case study was within the range of other case studies like in Chapter 7 and in previous research works (Carvajal et al., 2009;

Aranda Domingo, 2014). Within the case study, the return period considering variable initial reservoir for the value corresponding to 1,000 years assuming initial reservoir level equal to MNL was 7,063 and 7,064 years for the maximum reservoir water level and maximum outflow frequency curve respectively.

3. For the case study, the affection of initial reservoir level for the maximum reservoir water level and maximum outflow frequency curve has a small uncertainty (less than 2.1 %) for return periods below 1,000 years, by using 100,000 inflow hydrographs and 100,000 initial reservoir levels.
4. Within the case study, the global risk index reduced its value up to 91 % if variable initial reservoir level is accounted, from $1,968.9 \times 10^3$ (initial reservoir level equal to maximum normal level) to a median value of 175.1×10^3 euros (variable initial reservoir level).
5. The uncertainty associated to initial reservoir level fluctuation when calculating the global risk index was 4.2 % (values ranging from 163.7 to 191.6 thousands of euros).

The proposed methodology was applied to one case study and dam configuration. Thus, extrapolation of results requires further work and applications to different case studies.

7. Operational variables II: The influence of gate failure combined with initial reservoir level variability

This chapter focuses on proposing a stochastic methodology to assess the influence of considering variable reservoir levels prior to the arrival of floods in hydrological dam safety; introducing probability associated to gate failure scenarios.

Four scenarios with different gate failure rates are studied. Furthermore, a comparison is carried out regarding assuming initial reservoir level equal to the maximum level allowed in the reservoir under normal operation conditions with those considering variable initial reservoir levels.

7.1. Application to the Dam in the Douro river basin

A stochastic methodology within a Monte Carlo framework is proposed and applied to the Dam in the Douro river basin. The process is as follows (Figure 7.1):

Figure 7.1: General scheme of the stochastic methodology

- Stochastic initial reservoir level series generation. The probability of the levels reached in the reservoir for the maximum annual flood event is obtained.
- Synthetic reservoir inflow hydrographs generation. An ensemble of hydrologic loads according to available data, considering the following observed hydrograph characteristics: duration, volume and daily peak-inflow is generated, applying the methodology exposed in Section 3.1.2.

- Dam operation. The dam operation is simulated applying the Volumetric Evaluation Method (VEM) (Section 3.2.1). Furthermore, gate reliability was considered based on a fault tree analysis (Lewin et al., 2003; Escuder-Bueno et al., 2012; Serrano-Lombillo, 2011; SPANCOLD, 2012), applying the concepts exposed in Section 3.4.

The methodology was applied to Riaño arch dam; a reservoir which main purpose is irrigation. It is located on the headwaters of Esla River, a tributary of the Duero River. Riaño region has a continental climate characterized by flood events of several days of continuous rainfall of medium intensity (mainly generated by frontal systems). The dam has two bottom and two intermediate outlets, which were considered closed during the flood. Two Tainter gates and a complementary fixed-crest spillway compose the spillway. The width of each gate is eight meters, while its height is seven meters. Main characteristics of Riaño Dam and its basin are summarized in Figure 7.2 and Table 7.1.

Figure 7.2: Location of Riaño basin and schemes of Riaño Dam spillways. COD: Crest of Dam. MWL: Maximum Water Level. MNL: Maximum Normal Level. GSC: Gated Spillway Crest.

A gauge station located at the reservoir location with data of daily flows from 1954 to 1984 was used. Also, a set of monthly reservoir inflows from 1940 to 2013 was considered. In order to identify the month when the maximum annual flood occurred, the daily flows data set was used. The maximum annual floods were also identified using the daily data and obtaining the necessary information about the hydrograph variables (volume, duration and daily peak-inflow) to generate the set of 100,000 inflow reservoir hydrographs. In order to estimate reservoir levels capturing the inter-decade climate variability of the region, the observed monthly reservoir inflow time series from 1940 to

2013 were used. Based on the available observed monthly reservoir storage series (1988 to 2013), the water balance model was calibrated (from 1988 to 2004) and validated (from 2005 to 2013). Afterwards, the whole set of monthly inflow time series (1940-2013) was used to estimate the distribution of reservoir levels.

Table 7.1: Main characteristics of Riaño basin and dam. MNL: Maximum Normal Level. (maximum reservoir level to which water might rise under normal operation (ICOLD,1994)). MWL: Maximum Water Level (maximum reservoir level which the dam has been designed to stand (ICOLD,1994))

Characteristic	Value
Area (km ²)	582
Concentration time (h)	11
Mean annual runoff (mm)	1167
Mean annual precipitation (mm)	1650
10-year maximum daily precipitation (mm)	95
Reservoir volume (hm ³)	651 (MNL)
Capacity Gate 1 (m ³ /s)	310.5 (MNL)
Capacity Gate 2 (m ³ /s)	310.5 (MNL)
Capacity Fixed-Crest spillway (m ³ /s)	98.7 (MWL)

7.1.1. Generation of series of stochastic initial reservoir levels

Randomness of natural events and dam operation cause variability in reservoir levels (Carvajal et al., 2009). These two aspects were considered simultaneously for the generation of a stochastic set of variable initial reservoir levels, as follows (Figure 7.3):

Figure 7.3: Generation of series of stochastic initial reservoir levels

1. Generation of a sample of 100,000 months in which maximum annual flood event occurred. First, for each year of the available reservoir inflow time series the month when the maximum annual flood occurred was identified. The maximum annual flood corresponds to maximum annual reservoir daily peak-inflow. Second, the empirical cumulative frequency distribution of occurrence of each month was calculated. Finally, a random sample of 100,000 months following the mentioned distribution was generated.
2. Generation of a sample of 100,000 variable initial reservoir levels. In order to estimate the evolution of the reservoir levels a monthly water balance model of reservoir operation was developed. The observed monthly reservoir inflows time series, monthly demands, reservoir management policies and monthly evaporation rates were used for this purpose. The aim of simulating long reservoir level time-series was to capture the current operation practice of the dam while accounting for the inter-decade climate cycles variability. The current monthly demands were used during the whole simulation period. By analysing the simulated and observed reservoir storage, the rules of operation were defined in order to represent the dam historic water supplied to demands. Once the water balance model was calibrated and validated, the monthly reservoir level series for the observed monthly inflows period was determined. For each month of the year the cumulative frequency distribution of monthly reservoir levels was calculated. A random initial reservoir level was obtained sampling from the probability distribution associated to each month in the aforementioned series of 100,000 months. Therefore, a set of 100,000 initial reservoir levels associated to 100,000 maximum-flood months was generated.

Applying the aforementioned procedure to Riaño, the cumulative frequency distribution of the months in which the maximum annual observed floods occurred (from 1954 to 1984) was obtained. 100,000 months of occurrence of maximum annual flood following the observed distribution were generated within a Monte Carlo framework. [Figure 5.3a](#) shows the probability of occurrence of the different observed months in which the maximum annual flood occurred and the corresponding probabilities for the stochastically generated series (100,000 months). As expected both distributions were similar.

Figure 7.4: Results from the simulation of initial reservoir levels. a) Probability associated to the months in which the maximum annual flood occurred. Grey blocks: observed data; black blocks: sample of 100,000 months generated. b) Calibration (left) and validation (right) of Riaño water balance model. Grey circles show the observed reservoir monthly storage (Obs.) compared to the simulated series (Sim.) from 1988 to 2004 for the calibration process and from 2005 to 2013 for the validation process; grey line represents the linear regression and dotted lines the confidence bounds of 95 % (Conf.). c) Empirical reservoir level distributions associated to each month of the year obtained from the simulation of 73 years (1940 to 2013)

Afterwards, in order to estimate the historic behaviour of the reservoir levels a monthly water balance model of the reservoir was developed. The model was calibrated from 1988 to 2004 and validated from 2005 to 2013 (Figure 5.3b), defining the rules of operation as exposed in the methodology. Comparing the observed and simulated monthly series, a coefficient of determination (R^2) of 0.822 and a Nash Sutcliffe Efficiency (NSE) coefficient (Nash and Sutcliffe, 1970) of 0.802 was obtained. For validation R^2 was 0.905 and NSE 0.901. Once calibrated and validated, the cumulative frequency distribution of exceedance of the monthly reservoir level using the inflow series from 1940 to 2013 was obtained (Figure 5.3c). Finally, the set of 100,000 initial reservoir levels following the stated methodology was generated.

7.1.2. Generation of synthetic reservoir inflow hydrographs

A stochastic sample of 100,000 reservoir inflow hydrographs using the 30 year-long series of daily inflow data (1954-1984) was generated applying the methodology of [Section 3.1.2](#).

First, the probability distribution of observed maximum annual flood duration was obtained. The durations ranged from four to nine days being five days the most common value (30.0%). A set of 100,000 hydrograph durations through a Monte Carlo procedure ([Figure 7.5a](#)) was generated. The Volume Frequency Curves (VFCs) associated to the range of observed durations from four to nine days were obtained, fitting them to different statistical distributions (Gumbel, GEV and LP-III with parameters estimated using L-Moments technique ([Hosking,1990](#))). All fits passed the Kolmogorov-Smirnov goodness of fit test, finally choosing GEV as it was the one with the highest P-Value in all cases. Results of the estimation of parameters are shown in [Table 7.2](#), while the fits are shown in [Figure 7.5b](#) (P-Values were within the range from 0.936 to 0.998 for all durations). Through the hydrological event-based model (calibrated with the parameters shown in [Table 7.2](#)), 100,000 hydrographs were generated.

To check the validity of the procedure, the synthetic peak-inflow frequency curve (PFFC) was obtained to compare it with the simulated hourly PFFC. Instantaneous observed peak-inflows were estimated with Equation (3.4) using $a=1.78$ and $b=-0.29$, parameters previously obtained for the Case Study region by CEDEX ([MARM, 2011](#); [Jimenez-Alvarez et al., 2012](#)). The observed instantaneous PFFC was fitted to the same statistic distributions aforementioned, being their parameters estimated using L-Moments technique. All fits passed the Kolmogorov-Smirnov goodness of fit test, being GEV chosen because it was the fit with the highest P-Value (P-Value equalled to 0.999). Estimated parameters are shown in [Table 7.2](#).

Figure 7.5: Generation of synthetic reservoir inflow hydrographs. a) Cumulative frequency distribution of observed and simulated reservoir inflow hydrograph durations. b) Fits of reservoir inflow hydrograph volume for the different durations and simulated volumes c) Sample of 100,000 simulated floods (asterisks) and its comparison to the observed floods (circles). Horizontal axis shows values of peak-inflows while vertical axis shows flood volumes.

Table 7.2: Estimated values of GEV parameters using L-Moments technique and hydrological event-based model parameters. V stands for the maximum mean daily observed volumes of the year of different durations (defined by its subscript), Q_{MAX} stands for the maximum mean daily peak-inflows, M represents the length of observed data, α is the scale parameter (m^3/s for Q_{MAX} and hm^3 for V), u is the location parameter (m^3/s for Q_{MAX} and hm^3 for V) and γ is the shape parameter (dimensionless). AR(1), AR(2), MA(1) and MA(2) are the parameters of the ARMA(2,2) model used, F_t stands for the factor of intermittency used as calibration and CN is the curve number used

Parameters of the GEV distributions fitted to the observed data							
Parameters	Q_{MAX}	V_{4days}	V_{5days}	V_{6days}	V_{7days}	V_{8days}	V_{9days}
M	30	30	30	30	30	30	30
α	148.6	21.6	24.2	25.6	26.3	26.4	26.1
u	301.1	54.4	62.6	69.9	75.9	81.5	87.3
γ	0.002	0.110	0.125	0.113	0.068	0.041	0.014
Hydrological event-based model parameters							
ARMA model Parameters (Sordo-Ward et al., 2012)				Calibration		SCS Parameters (MARM, 2011; Jimenez-Alvarez et al., 2012)	
AR(1)	AR(2)	MA(1)	MA(2)	F_t (%)		CN	
0.9	0.1	0.2	-0.1	81		66	

To compare the 100,000 peak-inflows (synthetic PFFC) of the sample generated with the observed PFFC, Gringorten plotting position formula (Gringorten, 1963) was used. NSE value of the observed and synthetic PFFCs was 0.767. Table 7.3 shows the

values associated to different return periods and their relative error. Finally, the sample of 100,000 hydrographs generated with the observed values was compared. As can be seen in Figure 7.5c, synthetic events provided a correct reproduction of the main hydrograph variables.

Table 7.3: Comparison between quantiles of PFFCs of simulated and observed IPF (estimated) for different Tr

Tr (years)	Observed (m ³ /s)	Simulated (m ³ /s)	Relative Error (%)
50	879	954	8
100	982	1,047	7
500	1,219	1,248	2
1,000	1,321	1,318	0
10,000	1,651	1,587	4

7.1.3. Reservoir-Dam system routing

The set of 100,000 hourly hydrographs was routed through the reservoir accounting the set of 100,000 initial reservoir levels. In addition, the possibility of gate failure was considered applying the methodology shown in Section 3.4. This way, 100,000 maximum outflows and 100,000 maximum reservoir water levels were obtained, as follows (Figure 7.6):

Figure 7.6: Scheme for Reservoir-Dam system routing

1. Fault tree with assigned gate failure rates. A fault tree combining different rates of gate failure was designed based on expert judgment (Lewin et al., 2003; Patev and Putcha, 2005; Escuder-Bueno et al., 2012; SPANCOLD, 2012; Serrano-Lombillo, 2011; Figure 3.9). Two failure types were differentiated: independent cause failure affecting a single gate and common cause failure affecting a group of gates. Each

node of the fault tree represented an event that could happen or not, that is, a binary event (Kuo et al, 2008; SPANCOLD, 2012). It should be pointed out that rates of failure were estimated through expert judgment because the historic sample was not big enough. However, the situation when designers and operators do not have enough data about gate functionality is common (SPANCOLD, 2012). It was also considered that gate failure was independent of the season, intensity or dynamics of the flood event. In order to analyse different situations of flood management, the four scenarios shown in Table 7.4 were studied.

Table 7.4: Failure scenarios of gates functionality. Independent gate failure of 5% and 15 % per demand was considered for the assumption of a well-maintained gate and gates with minor operation issues, respectively (Escuder-Bueno et al., 2012). Failure of both gates due to common causes was considered as 1% per demand (Lewin et al., 2003). Fixed-crest spillway was fully working for all scenarios

Scenario	Failure on demand (%)		
	Common cause failure	Independent failure of gate one	Independent failure of gate two
C _{0-0%}	0	0	0
C _{5-5%}	1	5	5
C _{5-15%}	1	5	15
C _{15-15%}	1	15	15

2. Generation of 100,000 cases of gate functionality. The cumulative relative frequency distribution associated to the functionality of the different gates of the spillway was obtained. Afterwards, a sample of 100,000 cases that followed the aforementioned gate functionality distribution was generated.
3. The set of 100,000 hydrographs accounting the set of 100,000 initial reservoir levels was routed through the reservoir. To do so, the volumetric evaluation method (VEM) was applied obtaining a set of 100,000 outflow hydrographs. A detailed description of the method can be found in Girón (1988) and in Section 3.2.1. Following this procedure, the maximum reservoir water level and the maximum outflow frequency curves are obtained.

Therefore, for the case study, two sets of 100,000 MWRLs and 100,000 MOs resulted in each one of the four scenarios. In the first set, Z_0 was considered constant and equal to MNL while in the second one the sample of 100,000 stochastic initial levels was used. The MWRL frequency curves for the four scenarios and the two sets of MWRLs (Figure 7.7) were calculated.

Figure 7.7: Maximum water reservoir level frequency curves. Black markers represent MWRL frequency curves assuming initial level equal to MNL (Z_{MNL}). Grey markers represent MWRL frequency curves assuming variable initial level ($Z_{VAR.}$). Tr stands for return period, while m.a.s.l stands for meters above sea level. MNL refers to Maximum Normal Level. MWL refers to Maximum Water Level while COD stands for Crest Of Dam. Scenarios ($C_{0-0\%}$ (top-left), $C_{5-5\%}$ (top-right), $C_{5-15\%}$ (bottom-left), $C_{15-15\%}$ (bottom-right)) are represented by the independent rate of failure of each gate

Assuming constant Z_0 , for the maximum water level (MWL), the return period reduced as gate failure probability increased, ranging from 504 years in $C_{0-0\%}$ to 19 years in $C_{15-15\%}$. While in $C_{0-0\%}$ there was not overtopping in the range of 10,000 years, the return period for overtopping was 468 years in $C_{15-15\%}$. Therefore, when variable Z_0 was assumed, the probability of overtopping decreased (Table 7.5).

Table 7.5: Different return periods obtained for MWL (Maximum Water Level) and COD (Crest Of Dam) levels for return periods ranging from one to 10,000 years

Reservoir Level (m.a.s.l.)	Tr (years) $C_{0-0\%}$		Tr (years) $C_{5-5\%}$		Tr (years) $C_{5-15\%}$		Tr (years) $C_{15-15\%}$	
	$Z_{MNL.}$	$Z_{VAR.}$	$Z_{MNL.}$	$Z_{VAR.}$	$Z_{MNL.}$	$Z_{VAR.}$	$Z_{MNL.}$	$Z_{VAR.}$
MWL	504	1681	45	138	29	88	19	56
COD	-	-	1273	4864	858	2735	468	1708

Table 7.6 shows in its first row, for each maximum level reached in the reservoir (total of 100,000), the ratio of the corresponding return period assuming variable Z_0 ($Tr_{VAR.}$) and constant Z_0 ($Tr_{MNL.}$) for each of the scenarios studied ($C_{0-0\%}$, $C_{5-5\%}$, $C_{5-15\%}$, $C_{15-15\%}$). The ratios ranged from 2.0 to 4.1.

Table 7.6: For each Maximum Reservoir Water Level (MWRL) and Maximum Outflow (MO), range of variations of the ratio between the return period obtained assuming variable initial reservoir level ($Tr_{VAR.}$) and assuming MNL ($Tr_{MNL.}$) for the different scenarios. Scenarios ($C_{0-0\%}$, $C_{5-5\%}$, $C_{5-15\%}$, $C_{15-15\%}$) are represented by the independent rate of failure of each gate, being each subscript associated to one of the two gates.

Variable analysed	$Tr_{VAR.} / Tr_{MNL.}$ $C_{0-0\%}$	$Tr_{VAR.} / Tr_{MNL.}$ $C_{5-5\%}$	$Tr_{VAR.} / Tr_{MNL.}$ $C_{5-15\%}$	$Tr_{VAR.} / Tr_{MNL.}$ $C_{15-15\%}$
MWRL	(2.1-4.1)	(2.1-4.0)	(2.1-3.9)	(2.0-3.9)
MO	(2.5-6.0)	(2.6-6.1)	(2.5-6.1)	(2.5-6.1)

On the other hand, the maximum outflow frequency curves obtained considering Z_0 equal to MNL and considering Z_0 as variable were analysed. Figure 7.8 shows that variable Z_0 provided higher return periods for the same MO. For example, in $C_{0-0\%}$, a value of $870 \text{ m}^3/\text{s}$ of MO had a return period of 500 years if Z_0 was assumed as constant and 1669 years if variable Z_0 was considered. Regarding to gate failure, MO frequency curves calculated for both constant and variable Z_0 showed that for the same MO value the return period obtained was higher when gate failure rate increased; however, differences were not as remarkable as in MWRL frequency curves. For example, for the return period of 500 years, MOs ranged from 870 to $856 \text{ m}^3/\text{s}$ with Z_0 equal to MNL and from $800 \text{ m}^3/\text{s}$ to $785 \text{ m}^3/\text{s}$ assuming variable Z_0 .

Figure 7.8: Maximum outflow frequency curves. Black markers represent MO frequency curves assuming initial level equal to MNL ($Z_{MNL.}$). Grey markers represent MWRL frequency curves assuming variable initial level ($Z_{VAR.}$). Tr stands for return period. Scenarios ($C_{0-0\%}$ (top-left), $C_{5-5\%}$ (top-right), $C_{5-15\%}$ (bottom-left), $C_{15-15\%}$ (bottom-right)) are represented by the independent rate of failure of each gate

Table 7.6 shows in its second row, for each MO reached, the comparison between the return period of the maximum outflow obtained assuming Z_0 as MNL (Tr_{MNL}) and the return period obtained assuming variable Z_0 (Tr_{VAR}). Tr_{VAR} ranged from 2.5 to 6.1 times Tr_{MNL} .

The results obtained were also compared among the different gate failure scenarios. For the same MWRL, the return period of $C_{0-0\%}$ (no gate failure was assumed) was compared to the other three scenarios in which failure was considered ($C_{5-5\%}$, $C_{5-15\%}$, $C_{15-15\%}$). Considering Z_0 equal to MNL, the ratio of the return periods obtained in the scenario $C_{0-0\%}$ and the scenarios considering failure of gates ranged from one to 240, 160 and 93 in $C_{15-15\%}$, $C_{5-15\%}$ and $C_{5-5\%}$ respectively. This ratio was reduced to a range from one to 110, 80 and 41 in $C_{15-15\%}$, $C_{5-15\%}$ and $C_{5-5\%}$ respectively when Z_0 was considered as variable (Figure 7.9).

Figure 7.9: Comparison between return periods obtained assuming failure of gates ($C_{5-5\%}$, $C_{5-15\%}$, $C_{15-15\%}$) and those obtained with no failure associated ($C_{0-0\%}$) for the same MRWL. Results are shown for MNL (left) and variable initial reservoir level (right). Scenarios are represented by the independent rate of failure of gate one (g1) and gate two (g2)

It is remarkable that the higher the return period was, the higher was the difference between accounting or not for failure of gates ($C_{5-5\%}$, $C_{5-15\%}$, $C_{15-15\%}$). The same analysis was performed considering maximum outflows reached (not shown). Considering initial level equal to MNL, for each maximum outflow the ratio of the return period obtained in the scenarios with failure of gates ($C_{5-5\%}$, $C_{5-15\%}$, $C_{15-15\%}$) and $C_{0-0\%}$ ranged from 0.8 to

1.0. When variable Z_0 was assumed, the ratio ranged from 0.7 to 1.1. Both ratios remained similar for return periods ranging from one to 10,000 years.

In order to explain the different behaviors of the ratio between the Tr of the non-gate failure scenario and the Tr corresponding to the gate failure scenario, an analysis of how MWRLs and MOs were affected by gate failure events and their implications on their associated frequency curves was performed. Figure 7.10 shows this analysis for $C_{0-0\%}$ and $C_{15-15\%}$ with Z_0 equal to MNL. Events with low MWRL in $C_{0-0\%}$ corresponded to higher MWRL in $C_{15-15\%}$ (Figure 7.10a). This was reflected in the decomposition of the MWRL frequency curve of $C_{15-15\%}$ (Figure 7.10b), in which events associated to gate failure (middle and lower plots in Figure 7.10b) were related to high Tr s of the MWRL frequency curve, while the non-failure events (upper plot in Figure 7.10b) represented the lower Tr s of the curve. When comparing both frequency curves (Figure 7.10c), MWRLs which corresponded to Tr s up to 10,000 years in $C_{0-0\%}$ corresponded to less than 100 years Tr s in $C_{15-15\%}$, being the rest of curve represented by events affected by gate-failure, and therefore increasing the difference between $C_{0-0\%}$ and $C_{15-15\%}$ as the Tr increased. When analyzing MOs, gate failure implied a reduction of MO respect to $C_{0-0\%}$ (Figure 7.10d). As the events associated to gate failure represented lower MOs, they corresponded to low return periods of the MO frequency curve, being the events in which no gate failure occurred representative of the whole range of Tr s analysed (Figure 7.10e). This resulted into small differences between $C_{0-0\%}$ and $C_{15-15\%}$ MO frequency curves (Figure 7.10f), being the differences located in return periods lower than ten. The results were similar with variable initial Z_0 , adding a mitigation of the effect of gate failure for both MO and MWRL.

Finally, the relationship among inflow hydrograph volume, peak-inflow hydrograph and maximum reservoir water level was analysed. Figure 7.11 shows the results for the scenario $C_{0-0\%}$. When Z_0 was considered as a constant, there were two different relationships between peak-inflow and MWRL (Figure 7.11a). The first was a linear relation with values of maximum levels similar to MNL (84.2 % of the 100,000 events); the second was a more scattered relation (15.8 % of the 100,000 events). There were also two trends when hydrograph volume was compared to MWRL (Figure 7.11b). When Z_0 was assumed as variable, more than half of the 100,000 MWRLs (58.5 %) were below MNL, and the aforementioned relations between the variables scattered (Figure 7.11c, d).

Figure 7.10: Analysis of gate failure affection to maximum water reservoir levels (a, b, c) and maximum outflows (d, e, f) for $C_{15-15\%}$ (failure scenario). Comparison between maximum reservoir levels (a) and maximum outflows (d) for $C_{15-15\%}$ and $C_{0-0\%}$ (non-failure scenario). Decomposition of Maximum water reservoir levels (b) and maximum outflows (e) frequency curves according to events in which no failure occurred (upper), events in which one gate fails (intermediate) and events in which both gate fail (lower). Comparison between $C_{15-15\%}$ and $C_{0-0\%}$ maximum reservoir water levels (c) and maximum outflows (f) frequency curves. Dashed dotted lines represents $C_{0-0\%}$ frequency curves, different grey color dots represented $C_{15-15\%}$ values (See legends).

Figure 7.11: Comparison of peak-inflow, volume hydrograph and maximum reservoir level reached for the 100,000 hydrographs generated and for $C_{0-0\%}$ scenario. Black dots consider initial level equal to MNL. Grey dots consider variable initial level. Right Y-Axis shows the percentage of the sample in each interval of reservoir levels.

For gate failure scenarios $C_{5-5\%}$, $C_{5-15\%}$ and $C_{15-15\%}$, Table 7.7 shows the percentage of the sample of 100,000 MWRL that belongs to each interval of reservoir levels defined.

Overtopping occurred in the gate failure scenarios in 96% to 98% of the cases for return periods of inflow hydrographs lower than 1,000 years for both MNL and variable Z_0 .

Table 7.7: Percentage of the sample of 100,000 MWRL that belongs to each interval of reservoir levels defined considering the different gate failure scenarios ($C_{0-0\%}$, $C_{5-5\%}$, $C_{5-15\%}$ and $C_{15-15\%}$) and initial reservoir level conditions (constant and variable)

MRWL range (m.a.sl.)	$C_{0-0\%}$ % of Sample		$C_{5-5\%}$ % of Sample		$C_{5-15\%}$ % of Sample		$C_{15-15\%}$ % of Sample	
	$Z_{MNL.}$	$Z_{VAR.}$	$Z_{MNL.}$	$Z_{VAR.}$	$Z_{MNL.}$	$Z_{VAR.}$	$Z_{MNL.}$	$Z_{VAR.}$
MRWL<MNL	0.0	58.5	0.0	58.5	0.0	58.5	0.0	58.5
MNL<MRWL<MWL	99.8	41.4	97.8	40.8	96.7	40.4	94.8	39.7
MWL<MRWL<COD	0.2	0.1	2.1	0.7	3.2	1.1	5.0	1.7
MRWL>COD	0.0	0.0	0.1	0.0	0.1	0.0	0.2	0.1

7.1.4. Results analysis

Using a stochastic approach, assessment of the effect of initial level and gate failure in hydrological safety of the dam associated to the case study has been performed. First, MWRL frequency curves and MO frequency curves were analysed. For the MWRL frequency curve (Figure 7.7), there was a displacement of the curve to the right when accounting for variable Z_0 in all the scenarios ($C_{0-0\%}$, $C_{5-5\%}$, $C_{5-15\%}$ and $C_{15-15\%}$). The reason is the increase of flood storage capacity in the reservoir for considering a variable Z_0 . For the case of MO, variable Z_0 also caused a displacement of the frequency curve (Figure 7.8). For all scenarios, 58.5 % of the 100,000 events did not generate outflows, being the whole incoming floods stored in the reservoir (Table 7.7). Therefore, accounting for variable Z_0 implied a more realistic way of hydrological safety assessment, leading to a higher level of estimated dam and downstream river safety, because of the reduction of maximum reservoir levels (Figure 7.7 and Table 7.5) and outflows (Figure 7.8). In contrast, when comparing all the scenarios for the constant initial reservoir level hypothesis, the increase of gate failure implied less conservative results (only $C_{0-0\%}$ satisfied the Spanish National Safety Regulations in force when the dam was built (MOP, 1967) regarding overtopping (Figure 7.7). The reason is the decrease of the discharge capacity of the dam. Even though the analysed dam had a complementary fixed-crest spillway, its discharge capacity (98,7 m³/s) was very low compared to the discharge associated to the gated spillway (2 gates of 310.5 m³/s each); not being able to counteract the effect of gate failure. When it comes to MO frequency curves (Figure 7.8), variable Z_0 provided lower outflows than accounting for Z_0 as MNL. However, the MOs were

similar for all the considered scenarios ($C_{0-0\%}$, $C_{5-5\%}$, $C_{5-15\%}$ and $C_{15-15\%}$). An increase in gate failure rate was associated to an increase in overtopping cases, but the maximum outflows kept similar (Figure 7.8, Figure 7.10f). When overtopping occurred, maximum outflows increased neutralizing the decrease of discharge capacity associated to gate failure.

The analysis done in the preceding paragraph was extended in Table 7.6 and Figure 7.9. In Table 7.6, for the same maximum reservoir water level, the return period Tr_{MNL} (return period obtained assuming Z_0 as MNL) was compared to Tr_{VAR} (return period obtained assuming Z_0 as variable) in each of the scenarios ($C_{0-0\%}$, $C_{5-5\%}$, $C_{5-15\%}$ and $C_{15-15\%}$). Tr_{VAR} ranged from 2.0 to 4.1 times Tr_{MNL} for all the scenarios. This result highlighted the influence of variable Z_0 with respect to the assumption of MNL as Z_0 , pointing that the variability of the initial level accounted in the reservoir provided higher flood storage volume. However, the range between Tr_{VAR} and Tr_{MNL} for each one of the scenarios was similar. Therefore, variations in discharge capacity did not necessary imply variations in the influence of variable Z_0 . In the case of MO (Table 7.6), the same behaviour was identified according to variable Z_0 and gate failure, being the ratio Tr_{VAR}/Tr_{MNL} ranged from 2.5 to 6.1 times. Therefore, variable Z_0 increased both hydrological safety of the dam (MWRLs) and downstream safety for all gate failure scenarios. However, it seemed that gate failure did not make a difference on the influence of variable Z_0 , as the ratios for both MWRLs and MOs remained similar for all the scenarios.

In Figure 7.9, the difference between the return periods obtained was shown for the same MWRLs in $C_{5-5\%}$, $C_{5-15\%}$ and $C_{15-15\%}$ compared to $C_{0-0\%}$ for both assumptions of Z_0 (MNL and variable). As stated, the increase of failure rate implied a decrease of the hydrological safety of the dam (MWRLs) for both assumptions of Z_0 . The larger the return period was in $C_{0-0\%}$, the larger was the affection of gate failure. Comparing the two assumptions of Z_0 (Figure 7.9: left for MNL and right for variable initial reservoir level), it can be seen that the affection of gate failure was reduced by a half when variable Z_0 was accounted (from 240, 160 and 93 to 110, 80 and 41 for $C_{15-15\%}$, $C_{5-15\%}$ and $C_{5-5\%}$ respectively). The influence in MOs of gate failure was also studied. When analyzing MOs for all the scenarios, $Tr_{C5-5\%}$, $Tr_{C5-15\%}$ and $Tr_{C15-15\%}$ ranged 0.8 to 1.0 times $Tr_{C0-0\%}$ when MNL assumed, while the range changed to 0.7 to 1.1 when variable Z_0 accounted.

Gate failure had little influence regarding to MOs for both either constant or variable Z_0 (as stated, overtopping neutralized the decrease of discharge capacity).

The results obtained had some similitudes and differences with those presented by other authors. [Carvajal et al. \(2009\)](#) and [Aranda Domingo \(2014\)](#) obtained MWRL frequency curves and compared the results between accounting or not variable Z_0 . [Aranda Domingo \(2014\)](#) also compared initial reservoir level assumptions for the MO frequency curve. Furthermore, in [Chapter 6](#), the influence of initial reservoir level in MWRL and MO frequency curves was studied for a Dam in the Tagus river basin. Two of the dams studied had a gated spillway, and the other three a fixed-crest spillway. In the gated dam studied by [Carvajal et al. \(2009\)](#), variable Z_0 had no influence to the MWRL frequency curve (Case 3-[Carvajal et al. \(2009\) Table 7.8](#)). However, in the case of Riaño and the studied Dam of [Chapter 6](#), the influence was apparent ([Table 7.8](#)). From the different characteristics associated to the dams, it seemed that fluctuation of reservoir levels ([Table 7.8](#)) is the determinant that affected the most the results obtained. The variability in reservoir levels is associated to the operation of the dam, which depends on the main purpose of the dam. Irrigation dams might be the ones where accounting for variable Z_0 is more important ([Table 7.8](#)). It seems that the typology of spillway did not affect the results, however it is noticeable to point that no gate failure was assumed in any of the study cases shown in [Table 7.8](#) (Riaño case shown is $C_{0-0\%}$).

Regarding the analysis of the effect of gate failure, it has been shown in the case of study the importance of accounting for gate functionality in order to analyse hydrological safety (MWRLs). Other authors also accounted for failure scenarios (eg: [Lewin et al., 2003](#); [Micovic et al., 2016](#)). [Lewin et al. \(2003\)](#) estimated qualitatively that considering gate reliability produced an increase of overtopping probability by a ratio that varies between two and 250 for individual dams. The ratios for Riaño for all gate failure scenarios (240 to 93 depending on the gate failure rate assigned, obtained dividing by the Tr in the gate failure scenario for the same MWRL in which $Tr_{C_{0-0\%}}$ equals to 10,000 years), were within the qualitative range stated by [Lewin et al. \(2003\)](#). In the study of [Micovic et al. \(2016\)](#), the same ratios for the dams affected by gate failure were also within the range exposed for a return period of 10,000 years. Furthermore, for higher return periods than 10,000 years these ratios increased proportionally, which matches what happened in the Riaño case ([Figure 7.9](#): The higher the return period was in $C_{0-0\%}$, the higher was the affection of gate failure). The width in the range proposed by [Lewin](#)

et al. (2003) is associated to the different causes that affect it. Lewin et al. (2003) stated that the larger the number of gates, the less relative increase of overtopping risk (the factor for 21 gates was two). In Riaño case, as it has two gates, the ratios were in agreement with Lewin et al. (2003) statement. Lewin et al. (2003) also stated that the higher the volume of maximum flood storage with respect to discharge capacity, the less influence gate failure has. The results of this chapter agree showing a reduction of the ratios by a half, when variable Z_0 was assumed (the flood storage capacity increased). Therefore, variable Z_0 has an influence on the affection of gate failure to MWRL frequency curve.

Table 7.8: Results obtained by different studies comparing the assumption of variable initial reservoir level to MNL in the obtainment of maximum water reservoir levels (MWRL) and maximum outflows (MO) frequency curves. Tr_{VAR} represents the return period which corresponds to the same MWRL or MO that has a return period of 1,000 years if MNL is assumed as initial reservoir level. The return periods shown for other studies are estimated, as they were collected from the graphs provided in their results. No failure of gates is considered within any of the results shown in this table

Case study	Basin Area (km ²)	Fluctuation	Main purpose	Spillway	Tr _{VAR} , which corresponds to Tr _{MNL} =1000 years	
					MWRL	MO
Riaño	582	Major	Irrigation	Gated	3,200	3,300
1-Carvajal et al. (2009)	2	Major	Irrigation	Fixed-crest	9,000	-
2-Carvajal et al. (2009)	20	Slight	Water supply	Fixed-crest	2,000	-
3-Carvajal et al. (2009)	200	Slight	Hydropower	Gated	1,000	-
Aranda Domingo (2014)	671	Major	Irrigation	Fixed-crest	5,000	5,200
Chapter 6	1,850	Major	Irrigation	Gated	7,063	7,064

The results obtained respect to Figure 7.10, pointed out that both MO and MWRL should be analysed together. MOs frequency curve was almost unchanged, but MWRLs frequency curve alteration increased as the return period increased. This effect should be more critical in embankment dams as they are designed with higher return periods than concrete dams. Furthermore, the results for MWRLs were very sensitive to the determination of the gate failure rate, being these differences more perceptible as MWRLs Trs increased (Figure 7.9).

Finally, the relation between inflow hydrograph variables and maximum water levels was analysed. For the case study, the dam could be overtopped because a combination of the failure of discharge elements simultaneously with a small flood. This agrees with the conclusions of Micovic et al. (2016). As stated, most of the overtopping cases (96-98%) in the current study were caused by peak-inflows with return periods

lower than 1000 years for the entire gate failure scenarios. As failure of gates decreases the discharge capacity, smaller floods can increase the maximum reservoir water levels reached in the reservoir. As shown in [Table 7.7](#) and [Figure 7.11](#), variable Z_0 also affected as 58.5 % of the cases in all the scenarios studied ($C_{0-0\%}$, $C_{5-5\%}$, $C_{5-15\%}$ and $C_{15-15\%}$) were below MNL when taking this assumption. The percentage therefore for MWRLs above MNL decreased. Flood storage volume, incoming hydrographs, gate failure and initial reservoir level, among others; affect the maximum reservoir water level reached in the reservoir. Small return periods of the inflow hydrograph relied to overtopping (and therefore high return periods of the maximum reservoir water level). High return periods of the inflow hydrograph resulted into maximum reservoir water levels below MNL (and therefore a low return period of the maximum reservoir water level). This opens a discussion about how hydrological dam safety is determined and which variable best characterizes the hydrological safety of a dam. Design return periods are commonly associated to the return period of the design flood hydrograph, considering that rainfall events generate hydrographs with peak-inflows with the same return period. Many authors questioned this hypothesis (e.g: [Adams and Howard, 1986](#); [Alfieri et al., 2008](#); [Viglione and Blösch, 2009](#); [Sordo-Ward et al., 2014](#)). It is also well-known that return periods associated to peak-inflows are not the same as those associated to hydrograph volume, existing a trend to analyse this by using copulas for multivariate flood frequency analysis (e.g.: [De Michele et al., 2005](#); [Shiau et al. 2006](#); [Salvadori et al., 2011](#); [Requena et al., 2013, 2016](#); [Aranda Domingo, 2014](#); [Volpi and Fiori, 2014](#)). This question can be extended to the return period associated to the hydrological safety of dams. Therefore, it should be noted the existence of

uncertainty associated to the concept of return period, considering that the variable that best represents the hydrological safety and the corresponding return period is the maximum level reached on the reservoir, being downwards river safety represented by the maximum outflows. The methodology proposed can be used to provide a homogeneous tool to obtain the hydrological loads input to a complete risk analysis of a system of dams, important in countries like Spain where many dam spillways will require an upgrade due to new safety regulations ([San Mauro, 2016](#)).

7.2. The influence of gate failure combined with initial reservoir level variability: Partial conclusions.

The main conclusions extracted from this research are as follow:

- To account for variable initial reservoir level is important when assessing hydrological safety, especially in dams which main purpose is irrigation. (Tr_{VAR}/Tr_{MNL} . ranged from 2.0 to 4.1 regardless of gate malfunction).
- To account for variable initial reservoir level implied that many of the incoming floods produced no outflow, increasing the estimated downstream river safety in comparison with common design practice. (Tr_{VAR}/Tr_{MNL} . ranged from 2.5 to 6.1 regardless of gate malfunction).
- Gate failure decreased the return period of maximum water reservoir levels obtained in the common design practice situation (constant initial reservoir level) up to 240, 160 and 93 (depending on the gate failure scenario considered). This affection was reduced by a half when variable initial reservoir level was considered.
- The higher the return period of maximum water reservoir levels, the higher the affection of gate failure and the differences by assessing different values to the gate failure rate.
- When overtopping occurred, maximum outflows increased and neutralized the decrease of discharge capacity associated to gate failure scenarios. Maximum outflows return periods assuming gate failure were almost the same as return periods of the non-failure scenario, regardless considering constant or variable initial reservoir level.
- The variables that best characterize the hydrological dam safety are the return period of the maximum reservoir water level and maximum outflow frequency curves.

It should be noted that the methodology and analysis is based on a case study and needs to be extended to other systems in order to validate the conclusions presented.

8. On the definition of Maximum Conservation Levels

The aim of this Chapter is to solve the conflicts that arise in the operation of multipurpose reservoirs when determining Maximum Conservation Levels, as they may imply a reduction in the reliability of the water supply system.

The presented procedure consists on the joint optimization of water supply reliability as well as dam and downstream river safety optimizing water demand supply reliability as well as dam and downstream river safety. Two different scenarios were analysed: accounting and not accounting for the variability of initial reservoir level prior to extreme flood events. In order to achieve the global optimum configuration of Maximum Conservation Levels for each season, the optimization of three variables was proposed: maximum reservoir level (return period of 1,000 years), maximum released outflow (return period of 500 years) and maximum reservoir yield with 90 % reliability.

To do so, a stochastic methodology is proposed to determine the seasonal Maximum Conservation Levels which combines three main innovative aspects:

- Stochastic assessment of hydrological dam and downstream river safety through return periods related to maximum reservoir levels and maximum outflows.
- Determination of MCLs that increase/maintain the water yield for a specific reliability while maintaining/improving the hydrologic dam safety.
- Determination of MCLs accounting for the variability of initial reservoir level prior to flood events.

The methodology is applied to a gated-spillway dam located in Spain, but can be applied to any reservoir regardless its location or spillway typology.

8.1. Application to the Dam in the Douro river basin

To obtain the optimal set of seasonal Maximum Conservation Levels (MCLs) accounting for both hydrological safety (dam and downstream) and water supply, the methodology explained in [Section 3.7](#) has been applied to Riaño Dam, which main characteristics have been defined in [Chapter 7 \(Section 7.1\)](#). Flood damage analyses summarized in the Dam Master Plan concluded that discharges above 700 m³/s create a

flood emergency condition in the downstream reach. This value (O_{EMER}) was adopted to identify catastrophic flood damages. The following data were used to perform the study:

- Simulation of flood control operation. Besides the flood control structures and dam configuration shown, a gauge located right downstream Riaño reservoir with 30 years of daily flows was used, as in [Chapter 7](#). With this time series, the 100,000 seasonal synthetic flood hydrographs were generated.
- Simulation of regular operation. A monthly time series of inflows from 1940 to 2013, environmental flow restrictions, evaporation rates, monthly demand distribution, Storage-Area-Height Reservoir Curves and Dead Storage Volume (all data obtained from Duero River Basin Management Plan ([CHD, 2015](#))) and the reservoir characteristics previously stated were used. An mean annual demand of 545 hm^3 was accounted.

8.1.1. Definion of the study set of Maximum Conservation Levels and proposed solutions

A set of MCLs that comprised from 651 hm^3 (volume at MNL) to 400 hm^3 was studied. For the sake of simplicity, the flood hydrograph volume of $T_r=5,000$ years (247 hm^3) was estimated, and a maximum flood control volume of 251 hm^3 was defined. The ranges of the difference between the volume in the reservoir at MNL and the different MCLs were discretized in intervals of 5 hm^3 . Thus, a set of $k=51$ possible MCLs per season was defined (associated to a volume in the reservoir of 651, 645, 640, ..., 405, 400 hm^3).

According to [Section 8.1.2](#), three main seasons regarding to maximum annual floods exist within Riaño: season 1 from the beginning of November to the end of January, season 2 from the beginning February to the end of April, and season 3 from the beginning of May to the end of October.

Therefore, as $n=3$ seasons, a set of $51^3=132,651$ configurations of MCLs per scenario was analysed.

8.1.2. Seasonal hydrographs

A set of 100,000 annual synthetic flood hydrographs representative of the 30 years of observed events (1954–1984) as illustrated in [Figure 8.1a](#), previously generated in

Chapter 7 (Section 7.1.2) was used. The marginal distributions of the synthetic peak-flows and hydrograph volumes (100,000 each) were analysed by using Gringorten plotting position formula (Gringorten, 1963).

Figure 8.1: a) Sample of the 100000 simulated inflow hydrographs associated to different seasons (color dots: blue, season 1; magenta, season 2 and green, season 3) and its comparison to the observed floods (black dots). b) Determination of the range of thresholds analysing the mean of the exceedances above the threshold ($X_S - S$) (right axis) and the mean annual exceedance (λ) above threshold test (left axis). c)

Validation of the threshold range using the Dispersion Index (DI) Test. The blue line represents the different values of DI for the different thresholds. The blue shaded area represents the values of DI within the confidence intervals determined with a level of significance of 0.05. d) Delimitation of the seasons in Riaño Basin applying the seasonality test based on Ouarda et al. (1993). Grey plots represent the test for different values of the S thresholds. Straight dashed-lines represent the different seasons.

To define the seasons, the POT analysis (Section 3.1.2.1) was performed. The range of thresholds by accounting for independence of POTs was defined. Figure 8.1b shows the mean of the number of annual POTs (λ) on the right y-axis and the mean of the

exceedances above the threshold (X_S-S) plotted against the threshold (S). X_S-S was a linear function of S values from $100 \text{ m}^3/\text{s}$ to $190 \text{ m}^3/\text{s}$. Figure 8.1c shows the Dispersion Index (DI) test, with confidence intervals associated to a level of significance of 0.05. It can be seen that the range of values previously defined were within the confidence intervals, being the DI values stabilized between 0.805 and 1.207 for values of S from $120.5 \text{ m}^3/\text{s}$ to $162.5 \text{ m}^3/\text{s}$. This range of S values, that corresponded to values of λ from 3 ($S=120.5 \text{ m}^3/\text{s}$) to 2 ($S=162.5 \text{ m}^3/\text{s}$) (λ values recommended by Lang et al. (1999)) events per year respectively (Figure 8.1b), was the one chosen to define the seasons. Figure 8.1d shows the graphical test based on Ouarda (1993) and Ouarda et al. (1993, 2006). Three different seasons were identified, associating each hydrograph to a season of occurrence within a Monte Carlo Framework.

The seasons defined were: season 1 from the beginning of November to end January, being the probability of occurrence of floods within this season 41.4 %; season 2 from the beginning February to end April, being the probability of occurrence 45.3 %; and season 3 from the beginning of May to end October; being the probability of occurrence 13.3 %.

8.1.3. Simulation of the dam operation rules

Once the configurations of MCLs were defined, for each configuration the values of $MWRL_{TR=1000y}$, $MO_{TR=500y}$ were obtained by simulating the flood operation of the reservoir for Sc.1 and Sc.2. By simulating the regular operation of the reservoir, $Y_{R=90\%}$ was obtained.

Figure 8.2 shows the values of $MWRL_{TR=1000y}$, $MO_{TR=500y}$ and $Y_{R=90\%}$ with respect to the MCLs of each season corresponding to each of the 132,651 configurations. Figure 8.2a shows that variation of MCLs in S1 did not affect $Y_{R=90\%}$. The reasons are: the water demands associated to the reservoir in S1 were less than 1 % of the annual demand (Da), and, as the irrigation season in the Riaño system extends from May to September (98 % of Da), the reservoir was able to recover the reduced volume in S1 within the months previous to irrigation (February, March and April). Figure 8.2b and Figure 8.2d shows that, for the case of Sc.1, the effects of MCLs were similar in the three seasons in terms of hydrological dam safety (Figure 8.2b) and downstream river safety (Figure 8.2d). However, in Sc. 2 (Figure 8.2c and Figure 8.2e), as regular operation was linked to the flood control operation by the initial reservoir level, MCLs in S1 and S3 did not affect

neither the hydrological dam safety nor downstream river safety. Variations of MCL_{S2} are the main affection to values $MWRL_{TR=1000y}$ and $MO_{TR=500y}$ in Figure 8.2c and Figure 8.2e respectively.

Figure 8.2: Representations of the 132,651 values of $MWRL_{TR=1000y}$, $MO_{TR=500y}$ and $Y_{R=90\%}$ obtained respect to the MCLs for each of the seasons studied. MCL_{S1} , MCL_{S2} and MCL_{S3} represent the Maximum Conservation Levels in season 1, season 2 and season 3 respectively. (a) Representation of the values $Y_{R=90\%}$, with a colorbar, which are the same in Scenario 1 (initial reservoir level equal to MCL) and Scenario 2 (variable initial reservoir level). (b, c) Representation of the values $MO_{TR=500y}$ with a colorbar in Scenario 1 (b) and Scenario 2 (c). (d, e) Representation of the values $MWRL_{TR=1000y}$ with a colorbar in Scenario 1 (d) and Scenario 2 (e).

8.1.4. Solutions proposal

First, the configurations which did not fulfil hydrological dam safety ($MWRL_{TR=1000y} > DFL$) and/or downstream safety ($MO_{TR=500y} > O_{EMER.}$) were identified in both scenarios. A total of 826 configurations (0.6 % of 132,651 configurations) had a value of $MWRL_{TR=1000y}$ higher than DFL in Sc.1. (red dots in Figure 8.3a), while none of the configurations had $MWRL_{TR=1000y}$ values higher than the DFL in Sc.2 (Figure 8.3c). It should be noted that, in Sc.1 the 826 configurations that did not fulfil hydrological dam safety, they also did not fulfil the downstream safety condition. On the other hand, 59,013 configurations (44.5 % of the total number of configurations) had a value of $MO_{TR=500y}$ higher than $O_{EMER.}$ in Sc.1, whereas 4,944 (3.7 %) in Sc.2. Figure 8.3a and Figure 8.3c show the pair of values $Y_{R=90\%}$ and $MO_{TR=500y}$ (grey points) for each analysed configuration in Sc.1 and Sc.2 respectively.

In Sc.1, the pareto-solutions consisted of 277 (Figure 8.3a, magenta points) configurations, being 98 configurations those that satisfy: $Y_{R=90\%} \geq D_a$, $MO_{TR=500y} \leq O_{EMER.}$ and $MWRL_{TR=1000y} \leq DFL$ (proposed solutions, cyan in Figure 8.3a). In Sc.2, for the same analysis, 247 configurations (magenta points in Figure 8.3c) and 135 (cyan points in Figure 8.3c) were obtained respectively. For both scenarios, the extreme proposed solutions were identified. On the one hand, in the case of $MO_{TR=500y} = O_{EMER.} = 700 \text{ m}^3/\text{s}$ (Figure 8.3a and Figure 8.3c, black point), $Y_{R=90\%} = 587 \text{ hm}^3$ (for Sc.1) and 600 hm^3 (for Sc.2) it represents an improvement (compared to D_a) of 7.7 % and 10.1 % respectively. On the other hand, in the case of $Y_{R=90\%} = D_a = 545 \text{ hm}^3$ (Figure 8.3a and Figure 8.3c, dark blue point), $MO_{TR=500y} = 452 \text{ m}^3/\text{s}$ (for Sc.1) and $366 \text{ m}^3/\text{s}$ (for Sc.2) representing a 64.2 % and 52.3 % of $O_{EMER.}$, respectively. It is important to point out that, in the case of using a deterministic procedure focused in hydrological dam and downstream safety (conventional procedure), any studied configuration of MCLs could be a potential solution (grey dots within Figure 8.3a and Figure 8.3c). Thus, if $MO_{TR=500y} = O_{EMER.}$, the proposed stochastic procedure presented improvements up to 146 hm^3 (26.8 % of D_a) compared to the worst regular operation configuration ($Y_{R=90\%} = 454 \text{ hm}^3$ in Sc.1; (Figure 8.3a)). The corresponding configurations of MCLs for the extreme proposed solutions are represented in Figure 8.3b (Sc.1) and Figure 8.3d (Sc.2) with the same colour scheme as in Figure 8.3a and Figure 8.3c respectively. In both scenarios, the highest MCLs were associated to S3, while the lowest to S1.

Figure 8.3: (a, c) Representation in grey of the pair of values $Y_{R=90\%}$ and $MO_{TR=500y}$ for Scenario 1 and Scenario 2 respectively. Red points represent the pair of values which do not fulfil overtopping requirements ($MWRL_{TR=1000y} > DFL$). The pareto front is represented with magenta dots, while the proposed solutions are represented in cyan. The black dot represents the solution that maximizes the demand fulfilling the Maximum Outflow released requirement ($MO_{TR=500y} = O_{EMER}$). The blue dot represents the solution in which $Y_{R=90\%}$ equals the current annual demand, minimizing the Maximum Outflow released. (b, d) MCLs for each month/season that correspond to the proposed solutions previously selected (blue and black dots), for S1 and S2 respectively. The red dashed line represents the Maximum Normal Level (MNL), the red dashed-dotted line represents the Design Flood Level (DFL), and the red continuous line represents the Crest of Dam (COD).

8.1.5. Comparison between the proposed configurations in the two scenarios

Finally, the limit proposed solutions in both scenarios, Sc.1 and Sc.2, were compared (Figure 8.4). In the case of $MO_{TR=500y} = O_{EMER} = 700 \text{ m}^3/\text{s}$, $MWRL_{TR=1000y}$ was 1100.6 m.a.s.l. The hydrological dam and downstream river safety were invariant for Sc.1 and Sc.2. However, higher MCLs were obtained for Sc.2, which increased the $Y_{R=90\%}$ of the system. In the case of $Y_{R=90\%} = Da = 545 \text{ hm}^3$, the differences between Sc.2 and Sc.1 for $MWRL_{TR=1000y}$ and $MO_{TR=500y}$ were 0.2 m.a.s.l and $86 \text{ m}^3/\text{s}$, respectively. Moreover, the MCL of season two were lower when the variable initial reservoir level was considered. This is because of the increase of MCLs in the other seasons. Despite

this, the demand supplied with a reliability of 90% was the same, accounting for lower values of $MWRL_{TR=1000y}$ and $MO_{TR=500y}$ if variable initial level was considered.

Figure 8.4: Spider plot comparing the value of the maximum conservation levels for different seasons (MCL_{SX}), $MWRL_{TR=1000y}$, $MO_{TR=500y}$, and $Y_R=90\%$. In black, solutions that maximize the volume supplied with $MO_{TR=500y} = O_{EMER}$. In blue, the solutions that supply the current annual demand minimize the maximum outflow released. The continuous line corresponds to Sc.1 and the dashed line to Sc.2.

Within the framework of the present study, accounting for the variability of initial reservoir level implied a reduction of flood control volumes (increase of MCLs) maintaining the risk of overtopping and downstream river safety. Consequently, the volume for satisfying the demands increased, providing a more reliable system in terms of regular operation.

8.2. On the definition of Maximum Conservation Levels: Partial conclusions

The main conclusions extracted from this study are as follow:

- The use of a stochastic methodology allowed to assess hydrological dam safety and downstream safety by obtaining the frequency curves of Outflow and Maximum Reservoir Water Levels, while accounting for the variability in hydrological loads with respect to deterministic procedures. As a drawback, it implied a more complex procedure and computational effort.
- Considering the initial reservoir level equal to the Maximum Conservation Level for each season, a set of 98 non-inferior solutions is proposed, and 135 possible configurations by considering variable initial reservoir level. From the proposed configurations, dam stakeholders are able to decide which configuration to use depending on their preferences: increasing dam and downstream hydrological safety or increasing water supply (with a specific reliability) in the water resources system. In Riaño case study, the presented procedure showed improvements in the regular operation that can satisfy an increase up to 10.1% of the current annual demand of 545 hm³ (with a reliability of 90%) while maintaining the same hydrological dam safety.
- Accounting for initial reservoir variability resulted in the possibility to supply an extra demand of 13 hm³ (2.4% of the current annual demand) compared to the optimal solution without accounting for initial reservoir level variability.
- The proposed stochastic procedure can improve the results obtained by deterministic procedures, increasing supply up to 26.8 % the current annual demand, from the worst regular operation configuration (not accounting for initial reservoir level variability) to the optimal configuration (accounting for initial reservoir level variability) of Maximum Conservation Levels.

9. Conclusions

The conclusions that have been reached as a result of the present research, the original contributions to hydrological dam and downstream safety assessment, as well as possible further research are presented in this final chapter.

9.1. Main conclusions

The present research provides methodologies for guiding practitioners to perform hydrological dam and downstream safety assessments through stochastic approaches while accounting for operational variables such as the initial reservoir level, the availability of spillways or operational flood management procedures among others.

In this regard, the present research has focused in analysing and improving key aspects in hydrological dam and downstream safety assessment.

9.1.1. Hydrological forcing: The Hybrid method

The proposed procedure has been divided into two main focuses: a) providing guidance on selecting the minimum number of storm events for deriving the flood frequency curve through event-based approaches and b) proposing a methodology that combines the advantages of distributed event-based and continuous models, for the derivation of frequency curves of maximum flow and maximum annual flood volume, by combining a stochastic continuous climate generator with a physically-based distributed hydrological model based on both event and continuous simulations.

To do so, the methodology applied compares the resulting flow of coupling a stochastic weather generator with a fully distributed physically-based model with the flow: a) resulting from selecting only a limited number of storms per year b) resulting from the application of the proposed Hybrid method.

For the Case Study of Peacheater Creek, regarding to the number and criterion for selecting the storms to be simulated in an event or hybrid based modelling approach:

- When analysing the correspondence between storms and maximum annual peak-flow and volume, the best ranking storm criterion is total depth, followed up by maximum intensity. Considering storms up to rank five sorted by the total depth criterion,

resulted in a probability of generating the maximum annual peak-flow of 94% and the maximum annual volume of 99%.

- In the case of the total depth criterion, the higher the return period of peak-flow or volume is, the lower the storm rank needed.
- For preserving the seasonality of maximum annual peak-flows and volumes, total depth was also the best criterion.
- When estimating the parameter of the copula family selected (Gumbel), the relative error was less than 2% if storms up to rank five were considered, and was less than 0.5% if storms up to rank ten were considered. Regarding Kendall's return period, considering the same number of storms produces Kendall's return periods of 10 and 50 years very similar to those obtained by continuous simulation, and the 100 years return period is slightly underestimated.

Regarding to the application of the Hybrid method in Peacheater Creek Case Study, by applying the criterion of selecting the five highest storms per year in terms of total depth:

- Independence between rainfall events and prior hydrological soil moisture conditions has been proved within the framework of the Hybrid method developed.
- The Hybrid method preserves the seasonality of maximum annual peak-flows and volumes. The method provides less than 8% absolute relative error in the estimation of the mean direction of seasonality of both maximum annual peak-flows and volumes compared to the continuous modelling results. However, regarding the dispersion of seasonality, the Hybrid method provides less dispersion around the mean direction of seasonality, providing a relative error up to 52 % and 32 % in maximum annual peak-flows and volumes respectively.
- The Hybrid method is able to preserve the dependence and bivariate properties of the floods derived from the continuous simulation. The correlation between maximum annual peak-flows and volumes is preserved by the Hybrid-based method similarly to the continuous modelling approach.
- When estimating the parameter of the Gumbel copula family selected, the relative error was less than 13%. Regarding Kendall's return period, Kendall's return period of 10 is very similar to those obtained by continuous simulation. However, for

Kendall's return period of 50 and 100 years, the Hybrid method overestimates volume values, having a good agreement with peak-flows.

- The proposed Hybrid method is capable of reproducing the univariate flood frequency curves with a good agreement to those obtained by the continuous simulation. The maximum annual peak-flow frequency curve is obtained with a Nash-Sutcliffe coefficient of 0.98, whereas the maximum annual volume frequency curve is obtained with a Nash-Sutcliffe value of 0.97.

The proposed Hybrid method permits to generate hydrological forcings by using a fully distributed physically based model reducing the computation times on the order from months to hours.

9.1.2. Spillway operation: The proposed K-Method

The proposed K-Method is a decision-making tool for dam managers that allows analysing the effects of applying different reservoir operation rules for both dam safety and downstream safety. In contrast with other fixed methods, the K-Method allows, for each analysed dam, the identification of the K-value (conducting a calibration process) that best adapts the management strategy to the specific conditions of the basin, the reservoir, the spillway, and to the objectives of flood management.

The method was applied to a dam in the Segura river basin:

- For the case study under analysis, the K-Method improved the results obtained with the Volumetric Evaluation Method, by reducing the maximum reservoir levels reached in the dam (the higher the K-value, the lower the maximum reservoir level), while the corresponding increase of outflows did not endanger downstream safety.
- In addition, by carrying out a dam risk analysis, a K-value of 5.25 lowered the global risk index by 8.4% compared to Volumetric Evaluation Method, and represented 17.3% of the maximum possible reduction determined by the Mixed Integer Linear Programming rule of operation method.
- Different behaviours of the method were identified depending on the return periods analysed. For events with a return period ranging from one to 25 years, K-value = 10 resulted in an important decrease in the maximum reservoir levels, while the corresponding increase of outflows did not endanger downstream safety. For events with a Tr ranging from 25 to 100 years, higher K-values reduced the number of events

that simultaneously achieved lower maximum levels and outflows. For events with a T_r higher than 500 years, the Inflow-Outflow rule of operation method simultaneously achieved the lowest maximum levels and outflows. In a practical case, even if the skill required to forecast the inflow hydrograph is limited, according to the general meteorological situation, the dam manager may be able to estimate the return period of the event that is occurring and therefore the conclusions obtained could be applied to decide on the best K-value to adopt.

9.1.3. Operational variables I: Influence of initial reservoir level

The influence of initial reservoir level has been assessed in two Case Studies: two gated-spillway dams, one in the Tagus River system and the other one in the Douro River system. In the case of the Tagus River system, a set of 100,000 simulations of initial reservoir level was carried out, in order to assess the uncertainty associated to this variable.

The main conclusions associated to the influence of initial reservoir level are:

- To account for variable initial reservoir level is important when assessing hydrological safety, specially in dams which main purpose is irrigation. When analysing a portfolio of different reservoirs, a dam that may seem safer than other at first, may not be if this variable is included in the analysis (for example, in an extreme case, comparing a hydroelectric dam, which is usually full, with a flood control reservoir, which is usually empty). Thus, this methodology can help stakeholders when carrying out decisions about prioritizing their investments.
- The use of a stochastic methodology allowed to assess hydrological dam safety and downstream safety by obtaining the frequency curves of Outflow and Maximum Reservoir Water Levels, while accounting for the variability in hydrological loads with respect to deterministic procedures. As a drawback, it implied a more complex procedure and computational effort.

Regarding to the Dam in the Tagus River Basin, results obtained showed:

- For the case study, considering the fluctuation of initial reservoir level provided a more realistic assessment of hydrological dam and downstream safety. When a conventional approach was used, the Design Flood Level was reached at a return period of 751 years, which does not fulfil therefore with the regulation standards (1,000 years). However, when initial reservoir level was accounted, the results

showed that the Design Flood Level was not reached in any of the set of 100,000 simulations carried out.

- Within the case study, the return period considering variable initial reservoir for the value corresponding to 1,000 years assuming initial reservoir level equal to Maximum Normal Level was 7,063 and 7,064 years for the maximum reservoir water level and maximum outflow frequency curve respectively.
- For the case study, it has been found out that the affection of initial reservoir level for the maximum reservoir water level and maximum outflow frequency curve has a small uncertainty (less than 2.1 %) for return periods below 1,000 years, by using 100,000 inflow hydrographs and 100,000 initial reservoir levels.
- Within the case study, the global risk index reduced its value up to 91 % if variable initial reservoir level is accounted, from $1,968.9 \times 10^3$ (initial reservoir level equal to maximum normal level) to a median value of 175.1×10^3 euros (variable initial reservoir level).
- The uncertainty associated to initial reservoir level fluctuation when calculating the global risk index was 4.2 % (values ranging from 163.7 to 191.6 thousands of euros).

Regarding to the Douro River Dam and the impact of initial reservoir level in hydrological dam safety, it has been found out that:

- The return period of maximum reservoir levels reached in the reservoir, increases when considering variable initial reservoir for the same reservoir level. This return period ranged from 2.0 to 4.1 times the one by considering the reservoir at its maximum operative level when the flood arrives, depending on the return period studied.
- To account for variable initial reservoir level implied that many of the incoming floods produced no outflow, increasing the estimated downstream river safety in comparison with common design practice. In this case, return periods for the same maximum outflow ranged from 2.5 to 6.1 times the ones obtained by applying common design procedures (initial reservoir level equal to Maximum Normal Level).

9.1.4. Operational variables II: Influence of gate failure

Regarding the analysis of the effect of gate failure, it has been shown the importance of accounting for gate functionality in order to analyse hydrological safety in the dam located in the Douro River basin:

- Gate failure decreased the return period of maximum water reservoir levels obtained in the common design practice situation (constant initial reservoir level) up to 240, 160 and 93 (depending on the gate failure scenario considered). This affection was reduced by a half when variable initial reservoir level was considered.
- The higher the return period of maximum water reservoir levels, the higher the affection of gate failure and the differences by assessing different values to the gate failure rate.
- When overtopping occurred, maximum outflows increased and neutralized the decrease of discharge capacity associated to gate failure scenarios. Maximum outflows return periods assuming gate failure were almost the same as return periods of the non-failure scenario, regardless considering constant or variable initial reservoir level.

9.1.5. Definition of seasonal conservation levels

When it came to the definition of Maximum Conservation Levels accounting for both regular and flood operation, in the Douro River Dam, it has been found out that:

- Considering the initial reservoir level equal to the Maximum Conservation Level for each season, the procedure presented within this thesis showed improvements in the regular operation that can increase up to 10.1% of the current annual demand of 545 hm³ for the same hydrological dam safety.
- Accounting for initial reservoir variability resulted in an increase of the maximum demand supplied of 13 hm³ (2.4% of the current annual demand) compared to the optimal solution without accounting for initial reservoir level variability.
- The proposed stochastic procedure can improve the results obtained by deterministic procedures up to 26.8 % the current annual demand, from the worst regular operation configuration (not accounting for initial reservoir level variability) to the optimal

configuration (accounting for initial reservoir level variability) of Maximum Conservation Levels.

9.2. Original contributions

As previously exposed, the objective of this thesis was to develop a stochastic methodology to assess hydrological dam and downstream safety while accounting for operational variables such as the initial reservoir level, the availability of spillways or the influence of the flood operational methods.

In this regard, the main contribution of this the thesis is the development of a stochastic framework for the analysis of dam and downstream safety basement for hydrological dam safety assessment (Figure 9.1), which includes:

Figure 9.1: Stochastic proposed framework of hydrological dam and downstream safety assessment.

- **Development of a simulation chain that covers all the process of hydrological dam and downstream safety assessment within a Monte Carlo simulation framework.** This simulation chain includes the processes involved in hydrological forcings, follow up by ways of assessing the impacts of operational variables such as initial reservoir level or the unavailability of spillways. Afterwards, the floods are managed through reservoir operational rules. This procedure provides a set of maximum reservoir level reached in the reservoir and maximum outflows derived through the spillways, which permits to assess hydrological dam and downstream safety and serve as an input for hydrological dam risk analysis.
- **Proposals for stochastic generation of hydrological forcings.** The first link in the simulation chain is the generation of synthetic hydrographs. Two approaches have been developed within this thesis.

The first one, consists on the use of a physically based model, distributed, coupled to a climate generator. This method can be applied to case studies in which high resolution spatio-temporal data is available. It provides a high-detailed modelling framework in which hydrological processes can be accounted in all the units in which the basin has been discretized.

In cases in which high-detailed analysis is not necessary or possible, the stochastic data-based hydrological forcing generator proposed can be applied. This generator provides a simpler approach of multivariate flood modelling using a lumped stochastic event-based model that allows the generation of hydrographs from historical floods observed at the dam entrance.

- **Development of a simple flood management procedure improving the current procedure implemented in most of the dams in Spain.** This thesis develops the K-Method, a simple method which can be used to define real-time operation rules for gated spillways. The K-Method is defined to improve the performance of the Volumetric Evaluation Method largely used for the specification of dam management rules during floods in Spain.
- **Assessment of the impact of initial reservoir levels in hydrological dam and downstream safety.** Hydrological dam safety assessment methods traditionally assume that the reservoir is initially full when it receives the design flood, thus staying in the conservative side when designing a new dam. However, the distribution of reservoir levels at the beginning of flood episodes takes more importance for evaluating the real risk associated to dams in operation. This thesis has shown the importance of accounting for this variable. Furthermore, a procedure to quantify the uncertainty associated to the affection of initial reservoir level to hydrological dam safety has been proposed.
- **Impact of the availability of the spillways of a certain dam in its hydrological safety.** This thesis has highlighted the importance of considering gate failure scenarios in hydrological dam and downstream safety, providing guidelines for its incorporation within a stochastic framework.
- **Hydrological dam and downstream safety as frequency curves of maximum reservoir levels reached in the reservoir and maximum downstream outflows.** In the standard engineering approach, hydrological assessment of dam and downstream

safety is associated to the Design flood for a given return period. Throughout this thesis it has been shown that the return periods of maximum levels reached in the reservoir and maximum outflows are the variables that best represent dam and downstream hydrological safety. This thesis has shown that these return periods do not have to correspond to those associated to the Design Flood, moreover if operational variables are accounted.

- **Maximum reservoir levels reached in the reservoir and maximum downstream outflows as inputs for hydrological risk analysis.** Within the framework of this thesis, it has been shown how the frequency curves of maximum reservoir levels and outflows can serve as inputs for complete hydrological risk analysis. An economic index has been proposed which combines the hydrological risk for the dam, linked to the maximum water level reached in the reservoir during the flood routing, and the flood risk in the downstream river reach, linked to the discharge releases from the dam.
- **A proposal for the allocation of flood available storage accounting for initial reservoir levels.** A conflict arises in the operation of multipurpose reservoirs when determining Maximum Conservation Levels, as they may imply a reduction in the reliability of the water supply system. This thesis presents a new procedure that consists on the joint optimization of water supply reliability as well as dam and downstream river safety. This procedure provides dam stakeholders a set of pareto-optimal configurations depending on their main focus: hydrological safety or water supply reliability.

9.3. Further research

Future research studies to continue the present line of research could be related to:

- **Combination of all the proposals of this thesis.** Throughout the thesis, different parts of the proposed methodology have been developed and improved. However, the methodology must be understood as a whole unit. For instance, the influence of initial reservoir level and gate-failure can be assessed accounting for the use of the proposed flood operational procedure (K-Method), including the definition of Maximum Conservation Levels accounting for all this factors.

- **Analysis of reservoir systems.** The proposed hybrid hydrological model can be applied to case studies with one or more dams. The use of a distributed hydrological model provides a framework for accounting for the operational variables of all the dams located within the modelled basin.

Furthermore, when applying water resource management models within this thesis, only the operation of one reservoir has been accounted. However, for assessing the impact of initial reservoir level, it would be interesting to account for the variations of all the water supply infrastructures managed under the same water resources system.

Thus, the affection of upstream dams into the downstream reservoirs can be accounted, providing a more realistic assessment of the hydrological downstream safety assessment.

The simulation of a reservoir system is very important since poor flood management in headwater reservoirs can have catastrophic consequences downstream, but introduces a large number of variables which increase its complexity.

- **Improvement of the assessment of potential spillway failures.** Other further research objective could be finding a more robust technique combining historic data and experts' judgement to obtain the gate failure rate. This research pointed out the importance between choosing one gate failure or another. Furthermore, the dependence of failure respect to intensity, magnitude and season of floods could be better analysed.
- **Application of the proposed K-Method to other case studies.** Although the results are promising, further research should be developed to ensure that the K-method improves the operation of flood control rules through a wide range of gated-spillway dams.
- **Impact of Climate and Land Use changes in hydrological dam and downstream safety.** Within the hydrological hybrid modelling framework proposed, climate change impacts on risk analysis can be assessed. For instance, stochastic weather generators can be perturbed in order to assess the impact of climate change in flood events. Also, future modifications in land use can be considered in distributed modelling frameworks which, combined with population growth and changes in reservoir operation policies, can affect the complete cycle of dam risk assessment.

- **Assessment of the uncertainties associated to the different variables involved in the stochastic methodology.** Within this the thesis, the impact of the uncertainty associated to initial reservoir levels has been analysed. However, other aspects such as the uncertainty in the hydrological forcings, affection of gate failure rates or the uncertainty in the flood management strategies could be assessed in future works.
- **Proposal of simple risk key performance indicators.** Through the application of the proposed methodology to a representative set of dams, it would be possible to propose simple indicators to assess, for instance, the influence of variable initial reservoir levels and gate failure, which can be used by practitioners within the dam risk assessment context or by the development of national dam safety plans.

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APPENDIX: Front and last page of JCR published papers

1.1. Influence of initial reservoir level and gate failure in dam safety analysis. Stochastic approach.

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Research papers

**Influence of initial reservoir level and gate failure in dam safety analysis.
Stochastic approach**

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ABSTRACT

This study proposes a stochastic methodology to assess the influence of considering variable reservoir levels prior to the arrival of floods in hydrological dam safety; introducing probability associated to gate failure scenarios. The methodology was applied to the Ríaño dam (northern Spain) by analyzing the effects of incoming floods with return periods ranging from one to 10,000 years. We studied four scenarios with different gate failure rates and compared the results assuming initial reservoir level equal to the maximum level allowed in the reservoir under normal operation conditions with those considering variable initial reservoir levels. The ratio of the return periods associated to different reference levels reached in the reservoir considering variable over constant initial level ranged from 2.0 to 4.1. The ratio of the return periods obtained assuming gate failure and no failure for the same reference reservoir level ranged up to 93, 160 and 240 depending on the gate failure rate assigned. The ratio of the return periods associated to different maximum spillway discharges considering variable over constant initial reservoir level ranged from 2.5 to 6.1. However, the ratio of the return periods obtained assuming gate failure and no failure for the same discharge ranged from 0.7 to 1.1, showing no influence of gate failure. For the study case, our analysis highlighted the importance of considering the fluctuation of the initial reservoir levels and different gate failure scenarios, emphasizing that the return periods of maximum levels reached in the reservoir and maximum outflows are the variables that best represent dam and downstream hydrological safety.

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1. Introduction

Overtopping due to hydrologic causes is a common mode of failure in the history of dams (ICOLD, 1995). In the standard engineering approach, hydrological assessment of dam safety for a given return period is analyzed with deterministic methods. A design flood hydrograph is obtained and routed through the reservoir-dam system assuming constant reservoir level prior to the arrival of the flood and equal to the maximum normal operating level (MNL). However, many of the involved variables have a stochastic nature (Carvajal et al., 2009; Sordo-Ward et al., 2013). In recent decades, many authors proposed probabilistic approaches accounting for the randomness associated with different variables (e.g.: Eagleson, 1972; Amaud and Lavabre, 2002; De Michele and Salvadori, 2002; De Michele et al., 2005; Carvajal et al., 2009; Sordo-Ward et al., 2012, 2013; Paquet et al., 2013; Bianucci et al., 2013, 2015; Brigode et al., 2014; Flores-Montoya et al., 2015, 2016). Several authors had been able to obtain accurate maximum peak-inflow frequency curves within a Monte Carlo framework (Loukas, 2002; Rahman et al., 2002; Amaud and Lavabre, 2002; Aronica and Candela, 2007). However, it is a matter of importance to characterize not only the extremal incoming reservoir floods but the hydraulic behaviour and response of dams, as their failure could have catastrophic socio-economic consequences (Serrano-Lombillo et al., 2010, 2016). In hydrological dam safety assessment, the maximum water level reached in the reservoir is the variable that best represents the hydrological safety of the dam (Bianucci et al., 2013; Serrano-Lombillo et al., 2012a; Aranda Domingo, 2014; Micovic et al., 2016). This way, the hydrological dam safety analysis does not only depend on the hydrological forcing, but also on the dam and reservoir characteristics and operation rules. Gate functionality scenarios should also be taken into account (Lewin et al., 2003; Patev and Putcha, 2005; Escuder-Bueno et al., 2012; SPANCOLD, 2012; Micovic et al., 2016) because gated-spillway dams represent about 30% of the large dams around the world (ICOLD, 2003). Gates may not open when needed because of human, mechanical, or electrical failures, among others (Lewin

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Alfieri et al., 2008; Viglione and Blöschl, 2009; Sordo-Ward et al., 2014). It is also well-known that return periods associated to peak-inflows are not the same as those associated to hydrograph volume, existing a trend to analyze this by using copulas for multivariate flood frequency analysis (e.g.: De Michele et al., 2005; Shiau et al., 2006; Salvadori et al., 2011; Requena et al., 2013, 2016; Aranda Domingo, 2014; Volpi and Fiori, 2014). We extend this question to the return period associated to the hydrological safety of dams. Therefore, we highlighted the uncertainty associated to the concept of return period, considering that the variable that best represents the hydrological safety and the corresponding return period is the maximum level reached on the reservoir, being downwards river safety represented by the maximum outflows. The methodology proposed can be used to provide a homogeneous tool to obtain the hydrological loads input to a complete risk analysis of a system of dams, important in countries like Spain where many dam spillways will require an upgrade due to new safety regulations (San Mauro et al., 2016).

6. Conclusions

The main conclusions extracted from this research are as follow:

- To account for variable initial reservoir level is important when assessing hydrological safety, especially in dams which main purpose is irrigation. (Tr_{VAR}/Tr_{MNL} ranged from 2.0 to 4.1 regardless of gate malfunction).
- To account for variable initial reservoir level implied that many of the incoming floods produced no outflow, increasing the estimated downstream river safety in comparison with common design practice. (Tr_{VAR}/Tr_{MNL} ranged from 2.5 to 6.1 regardless of gate malfunction).
- Gate failure decreased the return period of maximum water reservoir levels obtained in the common design practice situation (constant initial reservoir level) up to 240, 160 and 93 (depending on the gate failure scenario considered). This affection was reduced by a half when variable initial reservoir level was considered.
- The higher the return period of maximum water reservoir levels, the higher the affection of gate failure and the differences by assessing different values to the gate failure rate.
- When overtopping occurred, maximum outflows increased and neutralized the decrease of discharge capacity associated to gate failure scenarios. Maximum outflows return periods assuming gate failure were almost the same as return periods of the non-failure scenario, regardless considering constant or variable initial reservoir level.
- The variables that best characterize the hydrological dam safety are the return periods of the maximum reservoir water level and maximum outflow frequency curves.

It should be noted that the methodology and analysis is based on a case study and needs to be extended to other systems in order to validate the conclusions presented. Further research can be focused on:

- Improving the synthetic hydrograph generation, by coupling a stochastic weather generator (e.g.: Ivanov et al., 2007; Fatchi et al., 2011) with continuous and event-based physically-based distributed hydrological models (e.g.: Liuzzo et al., 2010; Vivoni et al., 2011; Flores-Montoya et al., 2016)
- Finding a more robust technique combining historic data and experts' judgement to obtain the gate failure rate. This research pointed out the importance between choosing one gate failure

or another. Furthermore, the dependence of failure respect to intensity, magnitude and season of floods could be better analyzed.

- Finding simple indicators to assess the influence of variable initial reservoir levels and gate failure, through the application to a representative set of gated-spillway dams.

Conflicts of interest

The authors declare no conflict of interest.

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1.2. Dependence Between Extreme Rainfall Events and the Seasonality and Bivariate Properties of Floods. A Continuous Distributed Physically-Based Approach.

Article

Dependence Between Extreme Rainfall Events and the Seasonality and Bivariate Properties of Floods. A Continuous Distributed Physically-Based Approach

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Abstract: This paper focuses on proposing the minimum number of storms necessary to derive the extreme flood hydrographs accurately through event-based modelling. To do so, we analyzed the results obtained by coupling a continuous stochastic weather generator (the Advanced WEather GENerator) with a continuous distributed physically-based hydrological model (the TIN-based real-time integrated basin simulator), and by simulating 5000 years of hourly flow at the basin outlet. We modelled the outflows in a basin named Peacheater Creek located in Oklahoma, USA. Afterwards, we separated the independent rainfall events within the 5000 years of hourly weather forcing, and obtained the flood event associated to each storm from the continuous hourly flow. We ranked all the rainfall events within each year according to three criteria: Total depth, maximum intensity, and total duration. Finally, we compared the flood events obtained from the continuous simulation to those considering the N highest storm events per year according to the three criteria and by focusing on four different aspects: Magnitude and recurrence of the maximum annual peak-flow and volume, seasonality of floods, dependence among maximum peak-flows and volumes, and bivariate return periods. The main results are: (a) Considering the five largest total depth storms per year generates the maximum annual peak-flow and volume, with a probability of 94% and 99%, respectively and, for return periods higher than 50 years, the probability increases to 99% in both cases; (b) considering the five largest total depth storms per year the seasonality of flood is reproduced with an error of less than 4% and (c) bivariate properties between the peak-flow and volume are preserved, with an error on the estimation of the copula fitted of less than 2%.

Keywords: stochastic weather generation; AWE-GEN; distributed hydrological model; tRIBS; storm identification; bivariate flood frequency curve

1. Introduction

Distributed physically-based hydrological models (DHMs) appeared in the 1960s and have been the object of critics due to their complexity and difficulty of use [1]. Nowadays, the availability of higher resolution spatio-temporal datasets, the appearance of high performance computers, and the development of parallel-computing [2–5] have opened the possibility of using these models for large size basins and long-term hydrological continuous simulations. Thus, challenges such as the influence of land-use changes [6,7] or the impact of climate change [8,9] on the involved hydrological processes can be analyzed with these approaches.

Another use of distributed hydrological models is the estimation of extreme floods for the design of civil infrastructures, such as bridges, levees, or dams. For these structures, it is necessary to estimate

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1.3. Hydrological Risk Analysis of Dams: The Influence of Initial Reservoir Level Conditions.

Article

Hydrological Risk Analysis of Dams: The Influence of Initial Reservoir Level Conditions

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Abstract: In this paper, we present a method to assess the influence of the initial reservoir level in hydrological dam safety and risk analysis. Traditionally, in professional practice, the procedures applied are basically deterministic. Several physical processes are defined deterministically, according to the criteria of the designer (usually in the conservative side), although there is a high degree of uncertainty regarding these processes. A relevant variable is the reservoir level considered at the beginning of flood events. Hydrological dam safety assessment methods traditionally assume that the reservoir is initially full when it receives the design flood, thus, staying in the conservative side when designing a new dam. However, the distribution of reservoir levels at the beginning of flood episodes takes more importance for evaluating the real risk for the dams in operation. We analyzed three different scenarios—initial reservoir level equal to maximum normal level, equal to a maximum conservation level, and following the probability distribution from the historical records. To do so, we presented a method applied to a gated-spillway dam located in the Tagus river basin. A set of 100,000 inflow hydrographs was generated through a Monte Carlo procedure, by reproducing the statistics of the main observed hydrograph characteristics—peak flow, volume, and duration. The set of 100,000 hydrographs was routed through the reservoir applying the Volumetric Evaluation Method as a flood control strategy. In order to compare the three scenarios, we applied an economic global risk index. The index combines the hydrological risk for the dam, linked to the maximum water level reached in the reservoir, during the flood routing, and the flood risk in the downstream river reach, linked to the discharge releases from the dam. The results showed the importance of accounting for the fluctuation of initial reservoir levels, for assessing the risk related to hydrological dam safety. Furthermore, a procedure to quantify the uncertainty associated with the effects of initial reservoir level on hydrological dam safety, has been proposed.

Keywords: dam; initial reservoir level; hydrological dam safety; downstream safety; global risk index; stochastic approach

1. Introduction

Failure of large dams is a concern in many countries, due to the high economic and social consequences associated with it. When designing a dam, engineers usually apply techniques to assure that the risk of dam failure is low, but the standards are applied differently, depending on the country in which the dam is located [1]. Even though the risk is low, the associated risk should be recalculated, as the legal regulations, climate conditions, basin, and dam characteristics may vary with time [2].

The field of dam risk assessment has evolved, worldwide, with the appearance of different guides and procedures in several countries [3–5], to support dam stake-holders in the decision-making process related to dam safety. It is also well-known that, due to the high variability of the natural processes,

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1.4. A Parametric Flood Control Method for Dams with Gate-Controlled Spillways.

Article

A Parametric Flood Control Method for Dams with Gate-Controlled Spillways

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Abstract: The study presents a method which can be used to define real-time operation rules for gated spillways (named the K-Method). The K-Method is defined to improve the performance of the Volumetric Evaluation Method (VEM), by adapting it to the particular conditions of the basin, the reservoir, or the spillway. The VEM was proposed by the Spanish engineer Fernando Girón in 1988 and is largely used for the specification of dam management rules during floods in Spain. This method states that outflows are lower than or equal to antecedent inflows, outflows increase when inflows increase, and the higher the reservoir level, the higher the percentage of outflow increase. The K-Method was developed by modifying the VEM and by including a K parameter which affects the released flows. A Monte Carlo environment was developed to evaluate the method under a wide range of inflow conditions (100,000 hydrographs) and with return periods ranging from one to 10,000 years. The methodology was applied to the Talave reservoir, located in the South-East of Spain. The results show that K-values higher than one always reduce the maximum reservoir levels reached in the dam. For K-values ranging from one to ten, and for inflow hydrographs with return periods higher than 100 years, we found a decrease in the maximum levels and outflows, when compared to the VEM. Finally, by carrying out a dam risk analysis, a K-value of 5.25 reduced the expected annual damage by 8.4% compared to the VEM, which represents a lowering of 17.3% of the maximum possible reduction, determined by the application of an optimizer based on mixed integer linear programming (MILP method).

Keywords: flood control; dam safety; gated spillway; Volumetric Evaluation Method; reservoir operation rule; mixed integer linear programming; Monte Carlo framework

1. Introduction

Flood management aims to guarantee dam safety, minimize downstream floods, and maintain the full operational capacity of reservoirs once a flood is over [1–5]. Dams with gated spillways represent around 30% of large dams around the world [6], and provide more possibilities for water conservation and flood abatement than those with fixed-crested spillways [7]. Gate management during a flood event represents a challenge for the dam operator, who makes decisions under pressure and during uncertain conditions. In addition, the time frame for decision making is usually extremely short, the information available is generally sparse, and the predictability of the meteorological situation is limited [8,9]. Real-time flood control operations at dams have been historically approached via predefined rules obtained by simulation techniques [10], or by using simulation methods such as the Volumetric Evaluation Method (VEM) [11], commonly used in Spain. Flood control policies generally establish the discharge at each time step by considering the available information at a previous time, e.g., inflow, reservoir stage, stored volume, and outflow discharge downstream, among others [12].

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1.5. Flood Control Versus Water Conservation in Reservoirs: A New Policy to Allocate Available Storage.

Article

Flood Control Versus Water Conservation in Reservoirs: A New Policy to Allocate Available Storage

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Abstract: The aim of this study is to contribute to solving conflicts that arise in the operation of multipurpose reservoirs when determining maximum conservation levels (MCLs). The specification of MCLs in reservoirs that are operated for water supply and flood control may imply a reduction in the volume of water supplied with a pre-defined reliability in the system. The procedure presented in this study consists of the joint optimization of the reservoir yield with a specific reliability subject to constraints imposed by hydrological dam safety and downstream river safety. We analyzed two different scenarios by considering constant or variable initial reservoir level prior to extreme flood events. In order to achieve the global optimum configuration of MCLs for each season, we propose the joint optimization of three variables: minimize the maximum reservoir level (return period of 1000 years), minimize the maximum released outflow (return period of 500 years) and maximize the reservoir yield with 90% reliability. We applied the methodology to Riaño Dam, jointly operated for irrigation and flood control. Improvements in the maximum reservoir yield (with 90% reliability) increased up to 10.1% with respect to the currently supplied annual demand (545 hm³) for the same level of dam and downstream hydrological safety. The improvement could increase up to 26.8% when compared to deterministic procedures. Moreover, dam stakeholders can select from a set of Pareto-optimal configurations depending on if their main emphasis is to maintain/increase the hydrological safety, or rather to maintain/increase the reservoir yield.

Keywords: hydrological dam safety; initial reservoir level; maximum conservation level; water conservation volume; flood control volume; yield reliability; regular operation; stochastic methodology

1. Introduction

Owing to increasingly risk-averse societies, stronger hydrological safety requirements are being imposed on existing dams in order to fulfil new regulations and prevent dam failures [1]. This problem can be addressed with two different kinds of technical solutions: hard solutions, as the alteration of dam spillways or elevation of dam crest and soft solutions, as the allocation of additional flood control volumes in the reservoir. Hard solutions increase the flood control capacity while maintaining the reservoir storage available for water supply. However, their main drawback is the need to allocate resources for infrastructure works. On the other hand, the implementation of soft solutions is easier and quicker, but can reduce the available volume to supply water with a specific reliability in a water system.

Allocation of flood control volume is addressed by defining a maximum conservation level (MCL), also known as flood-limited water level [2] or flood control level. MCL is the maximum operating level

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