STOCHASTIC APPROACH TO HYDROLOGICAL DAM SAFETY IN THE FRAMEWORK OF THE NEW DAM SAFETY STANDARDS IN SPAIN.

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Summary. The approval of the Technical Safety Standards for dams and reservoirs in Spain in 2021 increased the hydrological safety requirements, being necessary to check if measures have to be implemented in dams designed according to the prior standards. Some of these dams have scarce instrumentation and lack of hydrometeorological data. This work proposes a simple but sounded fully-stochastic methodology for assessing the hydrological safety of dams within this situation, potentially applicable outside academia. The methodology has been applied to a case study based on a real dam, and compared the results obtained to those of the standard deterministic procedure. We have found that the fully-stochastic methodology leads to higher maximum reservoir levels for the same return period, leading therefore to more conservative results for the case study. On this regard, potential flaws have been detected regarding the hypothesis of the standard deterministic procedure, discussed and solved by the proposed fully-stochastic methodology, which accounts for the stochastic nature of variables such as rainfall temporal patterns and antecedent runoff conditions. Moreover, the proposed methodology can be adapted for accounting for operational variables such as the initial reservoir level when the flood event occurs, providing results that resemble more to the reality of nature and the reality of dam-reservoir systems.

1 INTRODUCTION

Throughout history, there have been situations in which dams designed and built in accordance with good engineering practice have suffered accidents with catastrophic consequences. Many of these dam failures were caused by overtopping, according to the stats published by the International Commission on Large Dams (ICOLD 1995).

Even though there are continuous advances in academia regarding hydrological methods for assessing the probability of overtopping, the methods used within the standard engineering practice have evolved little in recent decades (Sordo-Ward et al. 2013). Specifically, in Spain, the engineering approach has commonly been based on the characterization of a "Design Storm", which by means of a rainfall-runoff model turns into the "Design Flood", for 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 hours 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, even though there is a high degree of uncertainty about them. Moreover, in many cases, the probability of flood events tells us little about the hydrological safety of the dam, as overtopping can be reached due to other circumstances, such as low available storage capacity within the reservoir or operational spillway faults (Gabriel-Martin et al. 2017).

Thus, it is essential to change the focus of hydrological dam safety from floods to an overall overview of the dam-reservoir system. In Spain, this need has been reflected with the approval of the Technical Safety Standards for dams and reservoirs ("*Normas Técnicas de Seguridad de las presas y sus embalses*", in Spanish, (MITECO 2021)) in 2021, which improve previous legislation associated to dam safety, providing a new legal framework, influencing both the design of new dams and adaptation of existing dams to the new criteria required. Within the prior regulations in Spain, as previously noted, hydrological dam safety was characterized by the return period of the design flood; which was assumed to correspond to the same return period as the design storm. However, within the new Technical Safety Standards for Dams and Reservoirs, hydrological dam safety is characterized by the return period of the maximum reservoir level during the storm. This new standard aligns with the research outcomes of academia, in which the hypothesis of considering that rainfall events generate hydrographs with peak-inflows and maximum reservoir levels with the same return period has been widely discussed (Alfieri et al. 2008; Sordo-Ward et al. 2014; Gabriel-Martin et al. 2017).

Apart from the aforementioned change of criteria, the Technical Safety Standards for dams and reservoirs increase the hydrological safety requirements for dams in terms of return periods with respect to prior regulations. These requirements are applicable not only to the new dams but also for the ones in operation. Considering that most of the dams in Spain were designed according to the prior standards, hydrological dam safety has to be checked in order to assess if any additional technical measures are to be implemented, such as the increase of capacity of the dam spillway, the increase of the dam crest elevation or even the placement of protections against overtopping.

It is also worth mentioning that some of the dams in Spain that will need to be checked are operated by the private sector, that may have limited technical and economic resources. The instrumentation of these dams is usually scarce, lacking from long-term series of inflows for their hydrological assessment. Moreover, in many of the cases, these dams are located in small ungaged basins which difficulties the hydrological dam safety analysis.

In view of the abovementioned points, a general stochastic methodology, potentially applicable in standard engineering practice, is presented within this work, focused on dams in with scarce hydrological data. Specific objectives are highlighted: a) to present and apply a sounded but simple fully-stochastic methodology, potentially applicable in professional practice, to a case study based on a dam in operation with limited available data, b) to assess its hydrological dam safety according to the new Technical Safety Standards and compare it with the traditional deterministic approach considered for its design, c) to analyze the limitations of the standard practice approach, focusing on the assumptions regarding return periods, and d) to analyze and discuss about other factors that may influence overtopping, accounting for both, those related to runoff generation processes and those associated to the dam reservoir operation system.

In the first part of the paper, we define a methodology in order to pursue these objectives and present the case study. Afterwards, we present the obtained results, discussing the outcomes of the research. Finally, the main findings and conclusions are highlighted.

2 METHODOLOGY

The national regulations of different countries establish a return period for the design of dams. In Spain, the Technical Safety Standards for dams and reservoirs establish (MITECO 2021) different return periods up to 10,000 years, depending on the dam typology (concrete, embankment…) and the potential impact a dam failure would have on the downstream areas. These return periods are associated to two different reservoir levels:

- Design Flood Level (DFL): Reservoir level which the dam has been designed to withstand, considering the abatement in the dam, with an adequate freeboard, in Design Flood situation.
- Check Flood Level (CFL): Reservoir level which the dam has been designed to withstand, considering the abatement in the dam, without producing overtopping, in extreme flood (Check Flood) situation.

Table 1 summarizes the different return periods to be considered as per the Technical Safety Standards in Spain:

Table 1: Return period to be considered (in years) depending on the dam characteristics and potential hazard classification as per the Technical Safety Standards in Spain (MITECO 2021).

Hazard Potential Classification	DFL	CFL	
		Concrete and other non-embankment dams	Embankment dams
A	1,000	5,000	10,000
B	500	1,000	5,000
C	100	500	1,000

Three different Procedures for hydrological dam safety assessment are applied within this research (Figure 1), with the purpose of comparing the results obtained by its application, summarized as follows:

- Procedure 1-Standard-deterministic, designed to represent the methodology usually applied in professional practice in Spain. The Design Flood and Check Flood are simulated through a rainfall-runoff model, considering that rainfall events generate hydrographs and maximum annual reservoir levels with the same return period, resulting in the DFL and CFL. The variables involved in the process are considered in a deterministic way, with the exception of the annual maximum daily rainfall which is obtained probabilistically based on the return periods studied.
- Procedure 2-Standard-stochastic, which represents the transition between the standard practice and the stochastic methodology proposed in this paper. Accounting that annual maximum daily rainfall is obtained probabilistically within Procedure 1, an ensemble of 2,000,000 return period values is obtained within a Monte Carlo framework, resulting each of them in an annual maximum daily rainfall value. Keeping the rest of variables within the rainfall-runoff model deterministic, a set of 2,000,000 maximum annual reservoir levels is obtained from which hydrological dam safety is checked.
- Procedure 3-Fully-stochastic, which is the one proposed in this research, for dams located in small ungaged basins, in which instrumentation and available data is scarce. Apart from generating an ensemble of 2,000,000 return period values as in Procedure 2, the variability of temporal rainfall event distribution and antecedent runoff conditions is considered within the rainfall-runoff model. Hydrological dam safety is checked from the set of 2,000,000 maximum annual reservoir levels obtained.

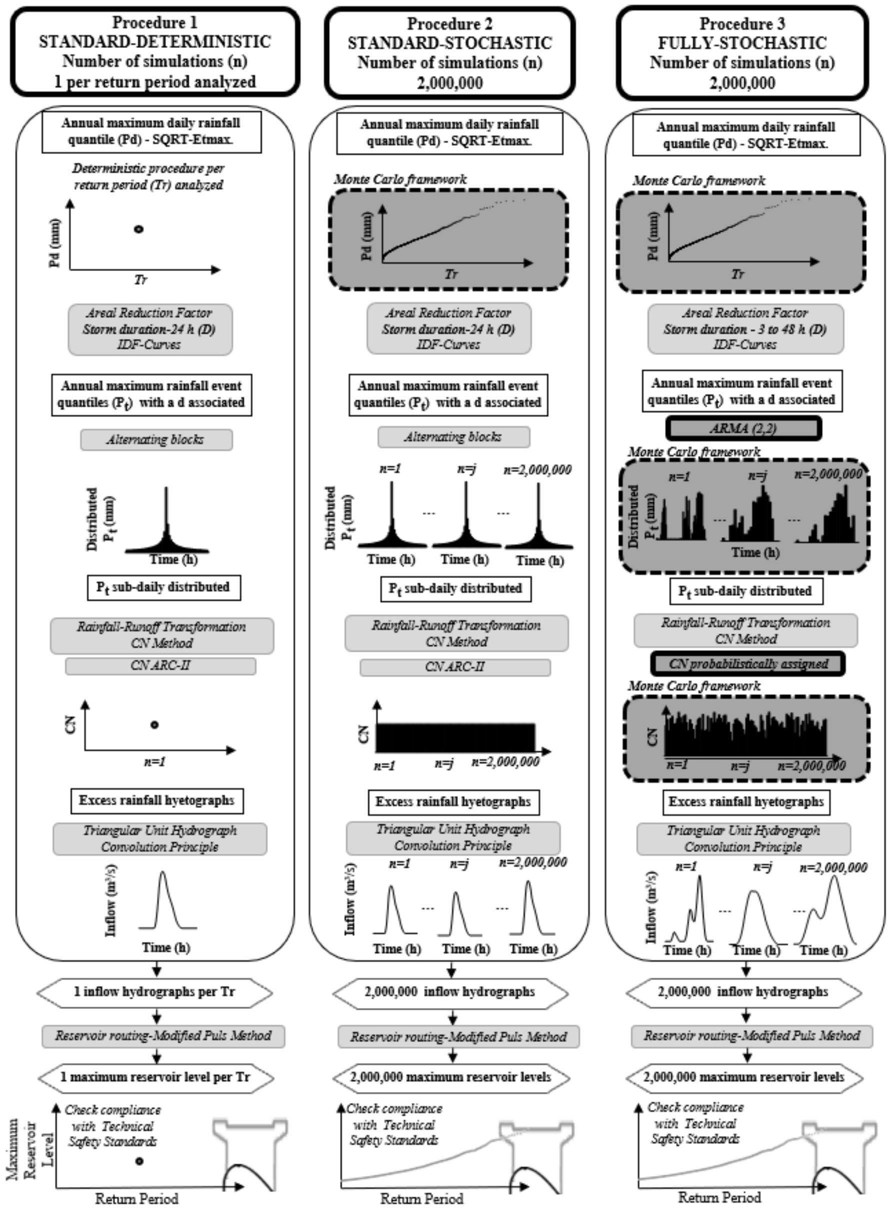

Figure 1: Conceptual scheme of the three Procedures applied within the Methodology. Stochastic processes are highlighted in dark grey.

2.1 Deterministic procedure versus Monte Carlo framework

As previously aforementioned, Procedure 1 is mainly deterministic, whereas Procedure 2 and 3 are developed within a Monte Carlo framework:

- For the deterministic case, in this research, two different return periods (Tr) are analyzed, representative of the DFL and CFL.
- In the case of the Monte Carlo framework, an ensemble of p-values within the range from zero to one is randomly generated for each of the stochastic processes involved.
  For stochastic approaches, in literature, sample sizes from ten to more than one hundred times the maximum analyzed return period are recommended to ensure representative results (Aronica and Candela 2007, Carvajal et al. 2009, Gabriel-Martin et al. 2017). On this regard, as the Technical Safety Standards in Spain require analyzing return periods up to 10,000 years, we have analyzed the number of simulations needed to reach stable

results within this range in Procedure 3 (which involves three stochastic processes), which we found to be 2,000,000 (section 4.1). For the sake of consistency, we performed the same number of simulations in Procedure 2.

2.2 Rainfall-runoff model

For each element in the sample of Tr generated (2 within Procedure 1 and 2,000,000 each in Procedures 2 and 3), a rainfall-runoff model is applied.

Accounting for each Tr value, the annual maximum daily rainfall quantile is obtained, which is turned into an areal storm event with a determined duration, intensity, total depth and temporal distribution. Afterwards, by means of an event-based hydrological model, we applied a rainfall-runoff transformation consisting of the application of the Curve Number method adapted to Spain (MARM 2011) to obtain the excess rainfall and the application of the unit triangular hydrograph proposed by Témez (Ferrer Polo 2000) and the convolution principle (Chow et al. 1988) to obtain the flood event.

2.2.1 Annual maximum daily rainfall quantiles

For each Tr (2 within Procedure 1 and 2,000,000 each in Procedures 2 and 3), we obtained the annual maximum daily rainfall quantile (P_d) by using the results of the study "Maximum Daily Rainfall in Continental Spain" (Ministerio de Fomento 1999), as it is common in standard practice in Spain when there is lack of rainfall data series.

In such study, 1545 rainfall stations were selected in the continental Spain with over 30 years of daily records. The quantiles of annual maximum daily rainfall were estimated for different return periods, by means of a regional adjustment of the SQRT-ETmax distribution law. The SQRT-ETmax (Etoh et al., 1986) is an extremal distribution function (F(x)) with two parameters:

$$If\, x \geq 0\ F(x)=e^{-k\left[\left(1+\sqrt{\alpha\cdot x}\right)e^{\left(-\sqrt{\alpha\cdot x}\right)}\right]}\ else\ F(x)=0 \tag{1}$$

where α (α>0) is the scale parameter and k (k>0) the shape parameter.

Regionalization was carried out by means of variable index method, obtaining reference isoline maps with values of mean annual maximum daily rainfall ($\overline{P}$) and Coefficient of Variation (CV), which can be used to estimate the quantiles of P_d for different return periods. Further details can be found in Ministerio de Fomento (1999) and Ferrer Polo (2003).

2.2.2 Annual maximum rainfall event quantiles for an associated duration at basin scale

It is to be noted that P_d is estimated at a local scale. In order to transpose it to the basin scale, we multiplied it by an areal reduction factor (ARF). The determination of the ARF coefficient is complex and requires subdaily rainfall data for its calculation. For this reason, in the absence of data, ARF in Spain is usually estimated from the empirical expression proposed by Témez (MARM 2011), in which an ARF (dimensionless) is defined as a function of the basin area (A, in km^2); being 1 if the basin areas is less than 1 km^2 and as follows in the case of basin areas equal or over 1 km^2:

$$ARF=1-\frac{log_{10}A}{15} \tag{2}$$

Furthermore, in order to obtain the annual rainfall event depth quantiles at basin scale (P_t) of a certain duration D based on P_d, application of Intensity-Duration-Frequency (IDF) curves is necessary. In Spain, in the absence of data, it is possible to use the IDF curves included in Ministerio de Fomento (2019), which have been routinely applied in hydrologic infrastructure design in Spain across a wide range of applications.

Accounting for these IDF curves and ARF, for each P_d (2 within Procedure 1 and 2,000,000 each in Procedures 2 and 3) we estimated P_t (in mm) of a certain duration (D, in hours) within the basin applying the following equation:

$$P_t=\frac{P_d{\cdot}ARF}{24}\cdot\left(\frac{I_1}{I_D}\right)^{\frac{28^{0.1}-D^{0.1}}{28^{0.1}-1}}\cdot D=\frac{P_d{\cdot}ARF}{24}\cdot(F_T)^{\frac{28^{0.1}-D^{0.1}}{28^{0.1}-1}}\cdot D \tag{3}$$

where:

- P_d represents the annual maximum daily rainfall quantile, as explained in section 2.2.1, in mm.
- ARF stands for the Areal Reduction Factor (dimensionless).
- F_T stands for the torrentiality factor (dimensionless), which expresses the ratio between the expected maximum hourly (I_1) and mean daily (I_D) precipitation intensity. This parameter has been regionalized for the whole territory of Spain (Ministerio de Fomento 2019). It is to be noted that this factor is independent of return period and varies from 8 (less torrential, as Galicia or southern Andalusia) to 12 (more torrential, as eastern part Balearic Islands) in Spain.

Regarding the duration of the storm events (D):

- For Procedure 1 and 2, a duration of 24 hours was considered. This assumption was based on standard practice approaches in Spain where, in the absence of rainfall data to evaluate the duration of storm events in the area, a duration of 24 hours is usually considered (Ferrer Polo 2000).
- For Procedure 3, different storm durations were considered: 3, 4, 5, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24 and 48h.

2.2.3 Temporal distribution of the annual maximum rainfall event quantiles at basin scale for an associated duration

For each P_t value (2 within Procedure 1 and 2,000,000 each in Procedures 2 and 3), we distributed the rainfall in time intervals of 0.5 hours (d) alongside the duration of the event (D), obtaining a rainfall hyetograph as follows:

- In Procedure 1 and 2, a synthetic hyetograph based on the alternating blocks method (Chow et al. 1988), following the IDF curves expression described in section 2.2.2. in each interval.
- In Procedure 3, each storm event of duration D has a probabilistic hyetograph with D/d rainfall pulses obtained by applying an autoregressive moving average (ARMA) (2,2) model with the parameters AR(1)=0.9; AR(2)=0.1; MA(1)=-0.2 and MA(2)=0 previously proposed by Sordo-Ward et al. (2013), since they were used within Spain country national level with adequate results. Therefore, each storm event has a different temporal distribution caused by the ARMA(2,2) model.

2.2.4 Rainfall-runoff transformation

For the rainfall-runoff transformation we applied the Curve Number method (Chow et al., 1988), in which the excess rainfall (P_e, in mm) depends on the cumulated rainfall (P, in mm) and the maximum potential retention (S, in mm):

$$If\ \ P>0.2{\cdot}S,\ P_e=\frac{(P\text{-}0.2S)^2}{P+0.8S};\ else\ P_e=0 \tag{4}$$

It is to be noted that the maximum potential retention (S, in mm) can be obtained from curve number (CN, dimensionless) as follows:

$$S=25.4\cdot\left(\frac{1000}{CN}-10\right) \tag{5}$$

The Curve Number was obtained from a national study (MARM, 2011; based on the results of Ferrer (2003)) that provided Curve Numbers (CN) in a 500 by 500 meters grid based on different parameters (soil characteristics, land uses, infiltration data…), corresponding to average median runoff conditions (ARC II). To obtain the value of the curve number for the dry antecedent runoff conditions (ARC I) and wet antecedent runoff conditions (ARC III), the following expressions are used (Chow et al. 1988):

$$CN(ARC\ I) = \frac{4.2\cdot CN(ARC\ II)}{10-0.058\cdot CN(ARCII)} \tag{6}$$

$$CN(ARC\ III) = \frac{23\cdot CN(ARC\ II)}{10+0.13\cdot CN(ARCII)}$$

As noted by Hjelmfelt (1991), CN is not a constant, but varies from event to event. Therefore, it can be considered as a random variable. Furthermore, Hjelmfelt (1991) showed that the maximum potential retention can be fitted to a lognormal distribution, defining ARC in terms of probability of runoff occurrence, with values of 10% (ARC I), 50% (ARC II) and 90% (ARC III).

Accounting for the aforementioned, for each rainfall hyetograph in the sample, we considered the following CN in order to obtain the excess rainfall hyetographs (2 within Procedure 1 and 2,000,000 each in Procedures 2 and 3):

- In Procedure 1 and 2, CN (ARC II) was considered for all the hyetographs.
- In Procedure 3, a CN was randomly assigned within a Monte Carlo Framework. To do so, a lognormal distribution was fitted to S, with values of probability of runoff occurrence 10% (ARC I), 50% (ARC II) and 90% (ARC III); obtained by applying equations 5 and 6. From this distribution, a random S value is obtained for each hyetograph, which by means of equation 5, is transformed back into its corresponding CN.

2.2.5 Hydrograph generation

For each excess rainfall hyetograph (2 within Procedure 1 and 2,000,000 each in Procedures 2 and 3) we applied the unit triangular hydrograph proposed by Témez (Ferrer Polo 2000) and the convolution principle (Chow et al. 1988) to obtain the resulting flood event.

The unit triangular hydrograph proposed by Témez consists of an adaptation of the SCS unit triangular hydrograph for its application in Spain. For its application, the following expression of concentration time (T_c), proposed also by Témez, is used:

$$T_c=0.3\cdot\left(\frac{L}{J^{0.25}}\right)^{0.76} \tag{7}$$

where:

- T_c is the concentration time, in hours.
- L is the length of the main channel, in km.
- J is the slope of the main channel (dimensionless).

Further details about the unit triangular hydrograph proposed by Témez can be found in Ferrer Polo (2000). Once the unit hydrograph method is applied, trough the convolution principle (based on the principles of superposition and proportionality) the flood event is obtained.

Applying the aforementioned methodology, a set of inflow hydrographs is obtained (2 within Procedure 1 and 2,000,000 each in Procedures 2 and 3). To these hydrographs, base flow should be added depending on the case study.

2.3 Dam-reservoir system routing

Each of the inflow hydrographs is routed through the dam-reservoir system applying the Modified Puls method, also known as storage routing or level-pool routing (Chow et al. 1988). By the application of the method, for each inflow hydrograph an outflow hydrograph and a reservoir level time series are obtained (2 within Procedure 1 and 2,000,000 each in Procedures 2 and 3).

2.4 Hydrological dam safety analysis

In order to assess hydrological dam safety and the compliance with the Technical Safety Standards, we proceeded as follows:

- For Procedure 1, we obtained the maximum reservoir level in each of the time series of reservoir levels (two in Procedure 1) obtained in section 2.3. These maximum reservoir levels correspond each to the DFL and CFL. As priorly mentioned, in the Standard-deterministic Procedure, it is assumed that rainfall events generate hydrographs and maximum annual reservoir levels with the same return period as it is commonly done in standard practice in Spain.
- For Procedure 2 and 3, we obtained the maximum reservoir level in each of the time series of reservoir levels (2,000,000 within each Procedure) obtained in section 2.3. Afterwards, for each Procedure, we derive the frequency curve of maximum reservoir levels by applying Gringorten plotting position formula (Chow et al. 1988).

Finally, based on the results, compliance with the Technical Safety Standards was checked.

2.5 Limitations of the methodology

The methodology applied had some limitations that should be noted:

- The methodology has been proposed on the basis of its application in dams with lack of instrumentation and unavailable hydrometeorological data. It should be noted that, if available, data should be taken into account in the analysis, either as inputs to the hydrological model or for calibration purposes. For instance, the scale parameter of the variable index method accounted in section 2.2.1 (represented by the mean annual maximum daily rainfall ($\overline{\mathrm{P}}$) obtained by means of kriging interpolation in Ministerio de Fomento (1999) study), can be estimated, if available, by the use of daily rainfall data in the area, assessing the impact of accounting for local observations. Moreover, if duration regarding storm events in the region is available, they can be accounted, thus not fixing the duration of the storm (section 2.2.2) but accounting for different durations in a probabilistic way. Regarding calibration purposes, in case there are coupled observations and information about hyetograph and hydrograph of flood events, they can be used to calibrate ARC in terms of probability runoff occurrence (we defer to De Michele and Salvadori (2002) and Aronica and Candela (2007) for further details).
- Furthermore, the methodology has been proposed bearing in mind small size basins (<250 km^2). In case of bigger size basins with no dams upstream, discretization in subbasins can be done following an approach similar to Sordo-Ward et al. (2013). In the case of basins with reservoirs upstream the dam under study, abatement and flood control operation rules can be included within the rainfall-runoff model itself, incorporating the reservoirs in the

model and simulating their operation rules, obtaining the outflow hydrographs, calculating their propagation and combination with other downstream hydrographs. For instance, the reader may refer to Micovic et al. (2016) for a case study in which an event-based stochastic approach was applied to a cascade system of three dams.

- The proposed methodology assumes the reservoir uncontrolled for the sake of simplicity. In any case, application of a flood control method can be implemented into the methodology proposed. The reader may refer to Sordo-Ward et al. (2017).
- Even though the methodology is applicable to different case studies, it should be noted that some of the results shown regarding hydrological dam safety (maximum reservoir levels) are specific to the case study basin and dam configurations studied; being therefore case-dependent.

3 CASE STUDY

The case study is based on a concrete gravity dam located within a stream tributary to the Tagus River. It is located in the province of Caceres, Spain, having a river basin extension of 105 km^2.

The dam has a height of 19 meters, having a crest length of 92.5 meters. The Maximum Normal Level (MNL, maximum reservoir level to which the water might rise under normal operation conditions), is located at 322.70 meters above sea level. The crest of the dam (COD) is located at 325.20 meters above sea level.

The main purpose of the reservoir is irrigation and, therefore, reservoir levels fluctuate during the year.

Regarding the flood evacuation structures, the dam has a gated-controlled spillway, with four Tainter gates of 5.25 meters width and 1.30 meters height. The four gates are located in four spans, with a Creager crest located at 321.40 meters above sea level. The spillway has a capacity of 200 m^3/s at 324.20 meters above sea level. Furthermore, the dam has one bottom outlet, which was considered closed during the floods.

Table 2 presents a summary of the main characteristics of the case study, based on the variables required for the application of the methodology proposed.

Table 2: Basin and rainfall-runoff main characteristics in the case study for the application of the methodology proposed. F_T and Tc are calculated by means of equation 3 and 7 respectively.

Basin characteristics		Rainfall			Soil moisture
Area (A) [km^2]	Concentration time (T_c) [h]	Mean annual maximum daily rainfall ($\overline{P}$) [mm]	Annual maximum daily rainfall coefficient of variation (CV) [-]	Torrentiality factor (F_T) [-]	Curve Number (CN) ARC II [-]
105	9	50	0.350	10	73

The reservoir volume is 0.570 hm^3 at MNL and 1.014 hm^3 at COD. In order to assess the influence of abatement capacity; four scenarios have been studied: the first one (Sc. 1), associated to the real dam; the other three, increasing the volume at each reservoir level 5 (Sc. 2), 10 (Sc. 3) and 20 (Sc. 4) times respectively. Figure 2 shows a scheme of each of the scenarios:

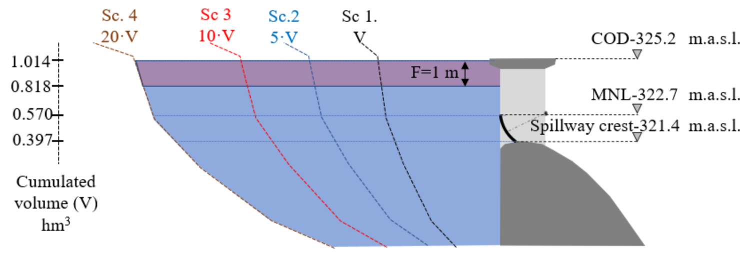

Figure 2: Scenarios accounted to assess the influence of abatement capacity. Scenario 1 (Sc.1) shows the case study associated to the real dam. Scenarios 2 (Sc.2 in blue), 3 (Sc.3 in red) and 4 (Sc.4 in brown) are synthetic scenarios in which the volume at the same reservoir level is multiplied by 5, 10 and 20 times respectively.

We have considered for all the simulations an initial reservoir level equal to MNL when each flood event occurs (common assumption in standard practice) for assessing its hydrological dam safety. Furthermore, for the sake of simplicity, the spillway was considered to be uncontrolled; thus, fully opening the gates at the beginning of the flood event so the maximum level leads to the maximum release. For the case study, the freeboard (F) considered is 1 m.

Finally, we have considered the dam has a hazard potential classification of “C” (less restrictive category in terms of dam safety), as its potential failure (breach) or mis-operation can cause moderate property damage but will not affect populated areas or essential services.

From a hydrological dam safety point of view, considering the case study is a concrete gravity dam classified as “C”, the Technical Safety Standards establish, among other conditions, that the following two requirements should be fulfilled:

- DFL<COD-freeboard, being DFL the maximum reservoir level associated to a return period of 100 years (Table 1).
- CFL<COD, being CFL the maximum reservoir level associated to a return period of 500 years (Table 1).

4 RESULTS AND DISCUSSION

4.1 Monte Carlo framework. Sensitivity analysis.

As noted in section 2.1, we have analyzed the number of simulations needed in Procedure 3 to reach stable results within a range of return periods up to 10,000 years. To do so, we carried out ten runs with different sample sizes (n): 20,000; 200,000 and 2,000,000 values. Afterwards, we compared the frequency curves of maximum reservoir levels (obtained for Scenario 1 (Sc.1)), finding out that a sample size of 2,000,000 values results in stable results for the return periods of interest as shown in Figure 3.

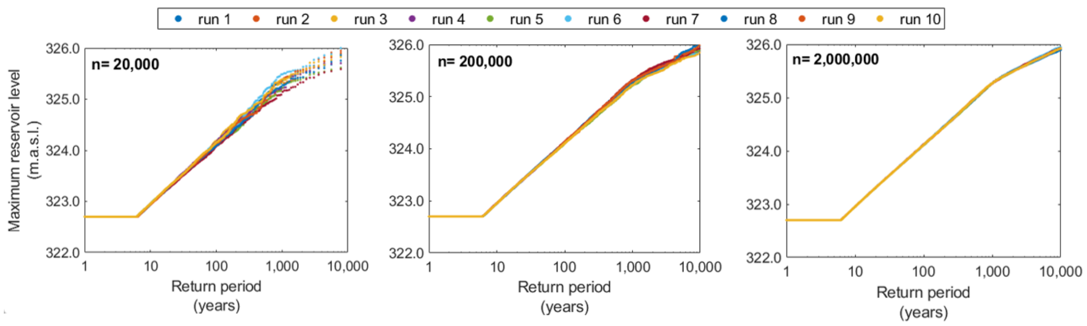

Figure 3: Sensitivity study of the sample size to be accounted. The figure shows the frequency curves of maximum reservoir levels of ten different runs with a sample size (n) of 20,000; 200,000 and 2,000,000 values.

4.2 Hydrological dam safety assessment.

By the application of the aforementioned methodology to the Sc.1 (Scenario which represents the real dam) of the case study, we obtained the DFL (Tr=100 years) and CFL (Tr=500 years) as per Procedure 1 (Standard-deterministic), assuming that the maximum reservoir level has the same return period as P_d. In the case of Procedure 2 (Standard-stochastic) and Procedure 3 (Fully-stochastic), we obtained the frequency curve of maximum reservoir levels from each sample of 2,000,000 maximum reservoir levels. It is to be noted that, in the case of Procedure 3, the sample of storm duration (D) 12 hours was selected, as it was the one which generated the highest maximum reservoir levels for Tr=100 years and Tr=500 years. Figure 4 shows the results for the three Procedures:

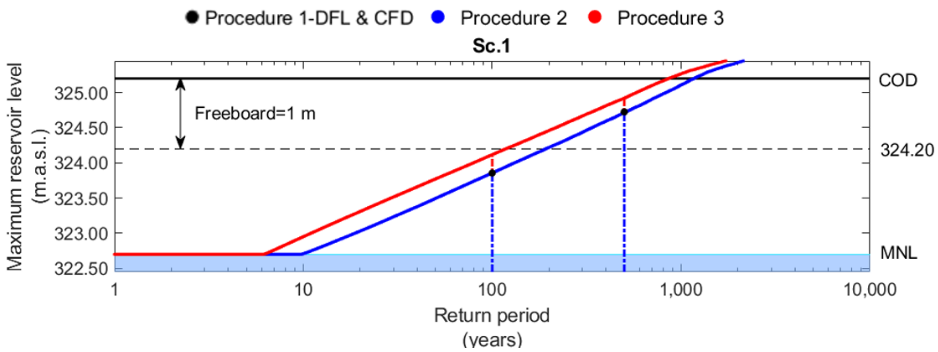

Figure 4: Maximum reservoir levels obtained within the three Procedures in Scenario 1. Black dots represent the DFL and CFL obtained within Procedure 1, whereas the blue and red dots represent the frequency curve obtained in Procedure 2 and 3 respectively. Dashed-dot lines represent the return period of 100 years (associated to DFL) and 500 years (associated to CFL).

As can be appreciated within Figure 4, within Procedure 1 (Standard-deterministic) and 2 (Standard-stochastic), DFL and CFL are similar, being equal to 323.86 m.a.s.l. and 324.71 m.a.s.l. respectively. In the case of Procedure 3 (fully-stochastic), DFL and CFL are 324.12 m.a.s.l. and 324.92 m.a.s.l. respectively. Therefore, the Technical Safety Standards are being fulfilled within the case study, as DFL< 324.20 m.a.s.l. (COD-freeboard) and CFL<325.20 m.a.s.l. (COD) according to the results obtained through the three Procedures.

Even though the Technical Safety Standards are being fulfilled regardless the Procedure, Procedure 3 leads to higher reservoir levels for the same return period. Moreover, overtopping occurs at 860 years of return period within Procedure 3; whereas the return period for overtopping in Procedure 2 is 1172 years of return period. Therefore, Procedure 3 leads to more conservative results for the case study. A deeper analysis regarding the flood hydrographs leading to the maximum reservoir levels in each Procedure and the influence of flood storage capacity is carried out in section 4.3.

4.3 Analysis of flood hydrographs and influence of abatement capacity.

As priorly noted, in order to assess the influence of abatement capacity; four scenarios have been studied: the first one (Sc. 1), associated to the real dam; the other three, increasing the volume at each reservoir level 5 (Sc. 2), 10 (Sc. 3) and 20 (Sc. 4) times respectively. We obtained the frequency curve of maximum reservoir levels in each Scenario, applying Procedure 2 and 3, as shown in Figure 5.

Furthermore, we analyzed the flood hydrographs properties (peak-flow and volume) which led to the maximum reservoir levels in each of the Scenarios and for each Procedure, by means of Figure 6.

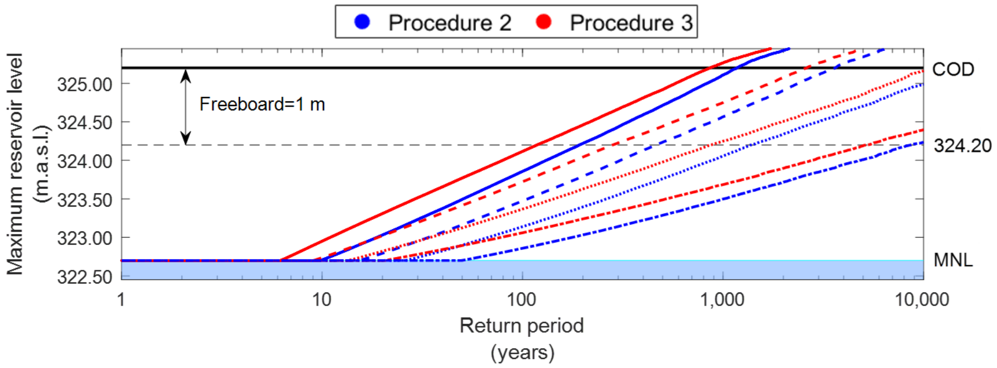

Figure 5: Maximum reservoir levels frequency curve obtained within the Procedure 2 (blue) and 3 (red) in Scenario 1 (continuous lines), Scenario 2 (dashed lines), Scenario 3 (dot lines) and Scenario 4 (dash-dot lines).

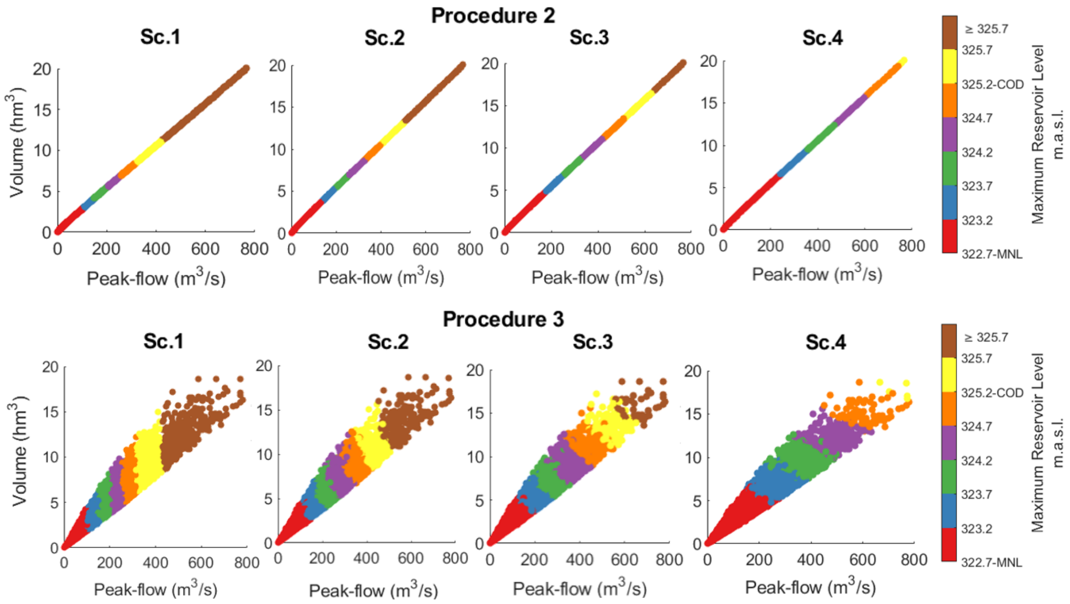

Figure 6: Representation of the 2,000,000 pairs of peak-flows and volumes resulting from the hydrographs of Procedure 2 (upper part) and Procedure 3 (lower part). Each figure is colored by the ranges of resulting maximum reservoir levels, with a similar color range for all the Scenarios.

Regarding the maximum reservoir levels frequency curve (Figure 5), as expected, the increase of flood control volume leads to higher return periods for the same maximum reservoir level. Regardless the Scenario, Procedure 3 is leading to more conservative results than Procedure 2.

When it comes to the hydrographs leading to the maximum reservoir levels, as can be inferred from Figure 6, the hydrographs that led to these values have different characteristics (Procedure 2, upper part of Figure 6; Procedure 3, lower part of Figure 6). In the case of Procedure 2, due to its deterministic nature, as all the hydrographs have the same shape, each pair of peak-flow and volume leads to one maximum reservoir level (upper part of Figure 6) for all scenarios. The main difference between Scenarios is that, as the flood control capacity increases, the same hydrograph leads to a lower maximum reservoir level.

However, within Procedure 3 (lower part of Figure 6), which accounts for the stochastic nature of the temporal rainfall distribution and antecedent runoff conditions, the hydrographs have different shapes and characteristics. The lower part of Figure 6 shows that different hydrographs with different peak-flow and volume led to similar maximum reservoir levels and, therefore, several flood events led to DFL or CFL, as can be appreciated from the curves that separate different reservoir level ranges. It is to be noted that, if the hydrograph volume frequency curve is analyzed, the volume quantile corresponding to 500 years return period in Procedure 3 equals to 6.90 hm^3. Accounting that the flood control capacity (COD-spillway crest) equals to 0.62 hm^3, 3.09 hm^3, 6.17 hm^3 and 12.34 hm^3 for Sc.1, 2, 3 and 4 respectively, the volume quantile of 500 years return period corresponds to, approximately, 11; 2; 1; and 0.5 times the flood control capacity. Stepper curve slopes appear in reservoirs with low abatement capacity, such as Scenario 1, in which maximum reservoir levels mostly depend on flood peak-flows, as can be appreciated in figure 6. On the other hand, milder slopes appear in reservoirs in which the flood storage capacity plays an important role, such as Scenario 4, as can be seen within Figure 6.

In addition, Figure 6 shows for Procedure 3 that hydrographs of lower peak-flows and volume than others can sometimes lead to higher maximum reservoir levels, and therefore, not only the relation between peak-flow and volume of the hydrograph affects hydrological dam safety, but also other characteristics as its temporal distribution and number of peaks.

Finally, it should be noted that the duration of the storm events in Procedure 2 equals to 24 hours, whereas in Procedure 3 equals to 12 hours (as it was the duration that generated the highest maximum reservoir levels for Tr=100 years (DFL) and Tr=500 years (CFL)). Annual maximum rainfall event quantiles at a basin scale (P_t) associated to Tr=100 years and Tr=500 years are 108.5 mm and 138.6 mm for a duration of 24 hours in Procedure 2, being 92.6 mm and 118.9 mm for a duration of 12 hours in Procedure 3. Even though higher rainfall depth quantiles are noted in Procedure 2; Procedure 3 assumes the CN as a random variable instead of considering a constant CN (ARC II); and therefore: a) rainfall quantiles of higher return periods can lead to lower volumes than those of lower quantiles; depending on the CN randomly assigned; b) higher rainfall quantiles in Procedure 2 than in Procedure 3 may not lead to higher rainfall volumes due to the assumption considered regarding the CN.

When it comes to peak-flow values, even though it is generally assumed in standard practice that one-peak synthetic storms (generated by alternating blocks method (Procedure 2) within this research) lead to more conservative (higher) peak-flow values than multi-peak distributions (as the ARMA model of Procedure 3) (Aron and Adl 1992); some studies have highlighted that, depending on the characteristics of the basin and subbasins discretization of the hydrological model; one-peak synthetic storms may lead to similar (or even slightly lower) peak-flows than multi-peak distributions (Sordo-Ward et al. 2014). Moreover, many authors (among others, Aronica and Candela 2007; De Michele and Salvadori 2002) pointed out that ARC could be the most important factor influencing peak-flows; and therefore, considering CN as a random variable instead of a constant value can have an important influence on the peak-flows obtained.

Unfortunately, a generalization of results would require repeating the analysis in a large number of basins, and it is outside the scope of the present study. However, what can be inferred from the methodology is that Procedure 3 relaxes constant CN (ARC II) and temporal rainfall distribution hypothesis, considering both as random processes, and provides a methodology with resembles more to the reality of nature.

4.4 Return periods analysis.

Finally, the hypothesis, currently assumed in standard practice, that the return period of a rainfall event is equal to the return period of the flood event and maximum reservoir level is questioned. As shown in Figure 7 for Scenario 1, when considering Procedure 2, the return periods of the main characteristics of rainfall, are coincident with those of the flood and those related to hydrological dam safety. However, when analyzing the results for Procedure 3, which considers the stochastic nature of rainfall temporal patterns and antecedent runoff conditions, the return periods differ, not only between rainfall and floods, but among the different characteristics of each process (maximum intensity and total depth in the case of rainfall, and peak-flow and volumes in the case of floods). Furthermore, an almost linear relation is shown between peak-flows and maximum reservoir levels in Procedure 3; which is caused by the little flood abatement capacity of Scenario 1. In other scenarios, such as Scenario 4 (not shown), the quasi-linear relation disappears in Procedure 3 as the flood volumes have a more predominant role in the resulting maximum reservoir levels.

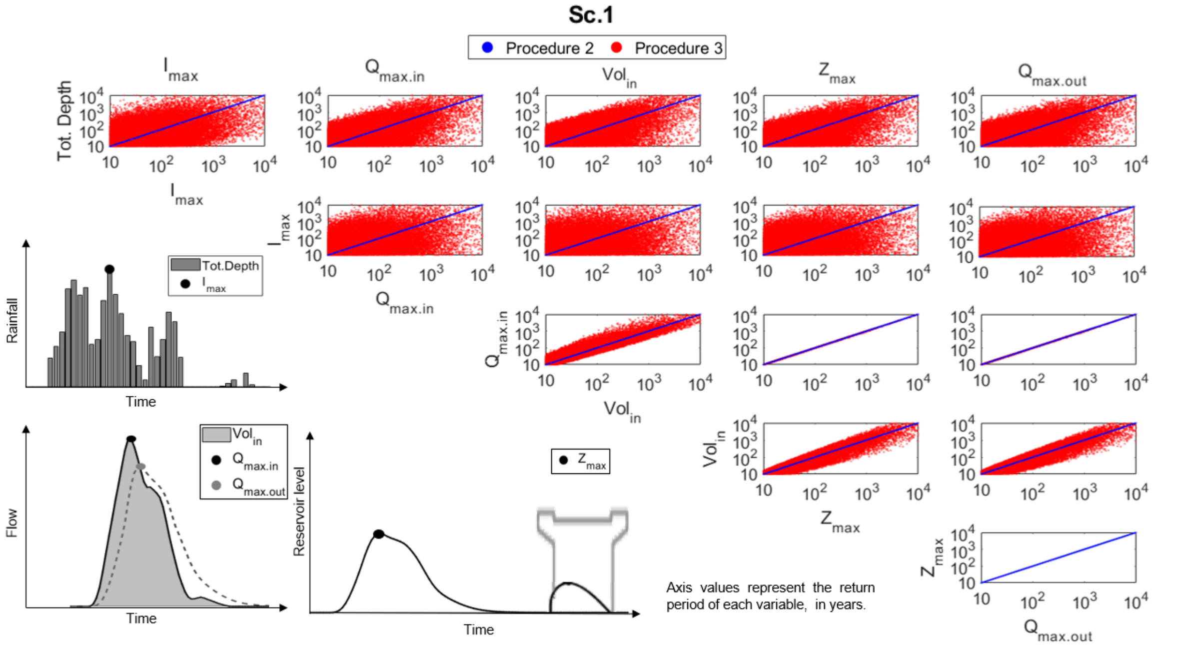

Figure 7: Relationship between the return periods (in years, shown within the range of 10 to 10,000 years for visualization purposes) of characteristics of rainfall events (maximum intensity (I_{max}) and total depth (Tot. Depth)), characteristics of the floods generated (peakflow ($Q_{max.in}$) and volume (Vol_{in})) and of the variables related to hydrological dam and downstream safety (maximum reservoir level (Z_{max}) and outflow ($Q_{max.out}$) for Procedure 2 (blue dots) and Procedure 3 (red dots) in Scenario 1 (Sc.1).

In the case of maximum reservoir levels and maximum outflows, Figure 7 shows that the return periods are aligned in Procedure 2 and 3. The reason behind these results are that an uncontrolled spillway was assumed, so each maximum reservoir level leads to one maximum outflow. However, if flood control structures and their operation were considered, dispersion would also appear among these variables (Sordo-Ward et al. 2013).

It is to be noted that, in the case of gated structures, including gate failure probability (Gabriel-Martin et al. 2017) or different flood operational methods (Sordo-Ward et al. 2017) would have an impact on hydrological dam and downstream safety. Another important variable that affects the maximum reservoir levels reached during a flood event is the initial reservoir level when the flood event occurs. Within the case study, we have assumed that this level is constant and equal to MNL (common assumption in standard practice). Whereas this hypothesis stays in the conservative side when designing a new dam, the distribution of reservoir levels at the beginning of flood episodes takes more importance for evaluating the real risk associated to dams in operation, moreover if the dam is not regularly at its maximum operational level (for instance, dams with irrigation purposes), as shown by different authors (Carvajal et al. 2009;

Micovic et al. 2016; Gabriel-Martin et al. 2017).

5 CONCLUSIONS

The Technical Safety Standards for dams and reservoirs in Spain approved in 2021 increased the hydrological safety requirements for dams in terms of return periods with respect to prior regulations. These requirements are applicable not only to the new dams but also for the ones in operation, being therefore necessary to check the hydrological dam safety of dams designed according to the prior standards.

Two of the major threats for a dam are loss of stability and overtopping, in which the maximum reservoir level plays a predominant role. Therefore, assessing hydrological dam safety by the return period of maximum reservoir levels is more appropriate than characterizing it by the return period of the design flood; as done within previous legislation in Spain.

A fully-stochastic methodology for hydrological dam safety, potentially applicable in professional practice, has been presented within this work, focused on dams with limited hydrological data. This methodology has been compared to the standard deterministic professional approach.

The results obtained show:

- For the case study, based on a real dam hazard potential classification of "C", even though both the standard approach and the fully-stochastic approach lead to a Design Flood Level and Check Flood Level that fulfill with the with the Technical Safety Standards requirements; the fully-stochastic methodology leads to higher maximum reservoir levels for the same return period, leading therefore to more conservative results for the case study. Overtopping occurs at a return period of 860 years of return period by the application of the proposed methodology; whereas the return period for overtopping in the standard approach is 1172 years.
- Whereas in the standard approach, each pair of peak-flow and volume leads to a unique maximum reservoir level, within the fully-stochastic approach, a unique maximum reservoir level is not characterized by one flood, but from an ensemble of floods with different peak-flows, volumes and temporal patterns.
- It has been found that, in the case of dams with little flood abatement capacity, flood peak-flows play a more predominant role than flood volumes, and vice-versa; by considering three synthetic scenarios of the case study in which the reservoir volume at each reservoir level increases 5, 10 and 20 times respectively.
- The hypothesis, currently assumed in standard practice, that the return period of a rainfall event is equal to the return period of the flood event and maximum reservoir level has been questioned. By the application of the fully-stochastic methodology, which resembles more to the reality of nature by accounting for random distributions of rainfall temporal patterns and antecedent runoff conditions, it has been shown that the hypothesis is not fulfilled.
- With the definition included in the new Technical Safety Standards, which characterizes hydrological dam safety by the return period of the maximum reservoir level and therefore, does not account for the aforementioned hypothesis, the randomness of the variables involved, both in the runoff generation process (rainfall patterns, antecedent runoff conditions…), both in the dam-reservoir system (initial reservoir level, gate failure probability, flood control operation strategies…) allows for a more flexible way in which, fully-stochastic procedures as the one presented in this paper are applicable for assessing hydrological dam safety in a more realistic way than the standard approaches in professional practice.

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