Technical Sciences
Academy of Romania
www.jesi.astr.ro

Journal of Engineering Sciences and Innovation
Volume 4, Issue 2/ 2019, pp. 185 - 196

D. Civil Engineering

Received ***15 March 2019*** *Accepted* ***30 May 2019***
Received in revised form ***9 May 2019***

A new approach in evaluation of a spillway capacity preserving the safety requirements

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Abstract. The 23 m tall Bawanur dam is a multipurpose project currently being constructed on the Diyala River, Iraq. The project provides storage, flood control, irrigation, power production and recreational uses. The dam body is embankment with clay core. In the initial design the spillway was provided with 9 bays equipped with 14 m span tainter gates capable to discharge 11460 m^3/s, corresponding to 10000 years return period outflow. The discharge from the Darbandikhan Dam (H = 128 m, V= 3 bil.m^3) located 52 km upstream was the main contributor to design inflow hydrograph into Bawanur reservoir. In the improved design the outflow from Darbandikhan spillway was reviewed by considering both the inflow discharge and the reservoir initial water level as independent random variables. The variability of initial reservoir level was defined based on recorded maximum annually reservoir water levels during the period 1962 – 2013. The outflow from the Darbandikhan dam corresponding to 1:10 000 years return period was reduced to 5298 m^3/s, significantly less than the one in the previous approach that was 11000 m^3/s. The spillway has now only 6 bays.

Key words: discharge, spillway capacity, hydrological regime, probabilistic approach.

1. Introduction

The proposed Bawanur dam, with the maximum height of 23 m, is a multipurpose project comprising storage, flood control, irrigation, power production (32 MW) and recreational uses. Bawanur dam is located on the Sirwan River, which is a left tributary of the Tigris River. River Valley lies on the regional tectonic line.
The hydrological regime of the Sirwan River is strongly influenced by the upstream already constructed dams. Darbandikhan reservoir is sited on the Sirwan River, 45 km upstream from the Bawanur project. With a storage volume of 3

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bil.m^3, it has a strong effect on the hydrseological regime of the Sirwan River. Consequently, the discharge capacity of Bawanur spillway was established based on the 1/ 10000 years return period flood routing through the Darbandikhan reservoir [1]. The effect of the inter-catchment between the Darbandikhan and Bawanur reservoirs as well as the effect of the Dewana reservoir (a small storage on a tributary) on the resulting flood-control effect of the Bawanur reservoir has also been assessed (figure 1).

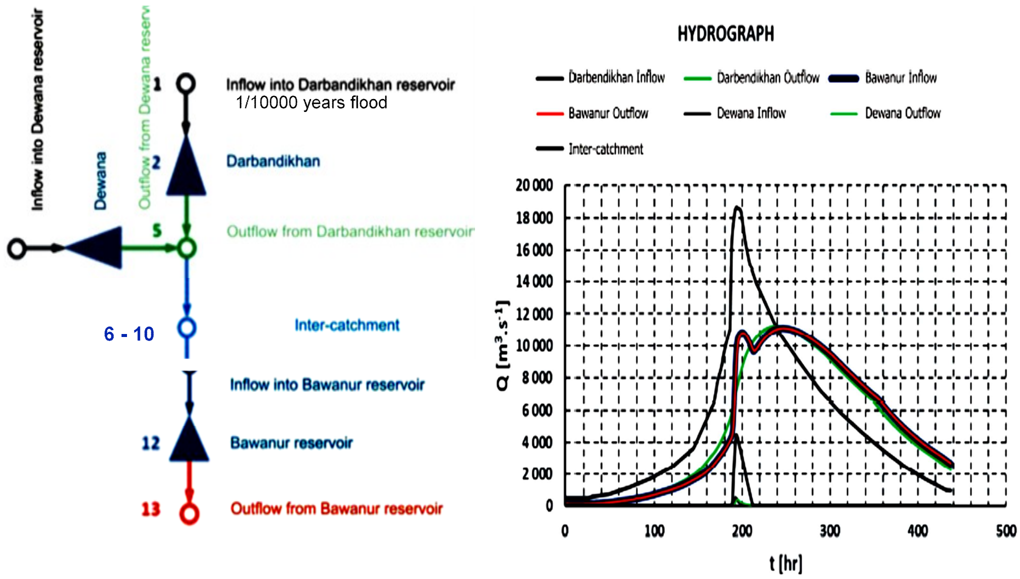

Fig. 1. Evaluation of flood inflow for Bawanur spillway.

Figure 1 shows that the design hydrograph of the inflow into the Darbandikhan reservoir and of the attenuated outflow. The attenuation provided by the reservoir is decisive for the design spillway capacity of the Bawanur project.

2. The goal of the study

Usually, when reservoir flood wave attenuation is computed, one start with the dimensionless inflow hydrograph. A discharge with an excedence probability is selected and the ordinates of this hydrograph are multiplied by that value. The usual method for the attenuation is the Modified Puls which considers that the initial water level in the reservoir is the normal highest full reservoir level, constant, no matter what exceedance probability is analyzed for the inflow discharge. As a random variable only the discharge is considered. But usually the initial water level in the reservoir is less than the normal highest full reservoir level. Consequently, there is another variable that can be considered as a random variable. The discharge and the water level are independent random variables and so the exceedance probability of the studied event can be regarded like a statistical intersection according to the low of composition of independent random events:

$$\text{Prob}\left(\bigcap_{i=1}^{n} A_i\right) = \prod_{i=1}^{n} \text{Prob}\left(A_i\right).$$

As an example, the maximum outflow discharge having 0.01% exceedance probability can be a result of a flood wave attenuation with 0.1% exceedance probability for the maximum discharge and considering the initial water level in the reservoir with 0.1% exceedance probability. Several combinations can be considered:

$$D_{\frac{1}{10000}} \leftarrow \begin{cases} Q^{\text{inflow}}_{\frac{1}{10}} \text{ with } \mathrm{H}^{\text{reservoir}}_{\frac{1}{1000}} \\ Q^{\text{inflow}}_{\frac{1}{100}} \text{ with } \mathrm{H}^{\text{reservoir}}_{\frac{1}{100}} \\ Q^{\text{inflow}}_{\frac{1}{1000}} \text{ with } \mathrm{H}^{\text{reservoir}}_{\frac{1}{10}} \end{cases}$$

We will estimate the outflow from Darbandikhan reservoir by assuming that the initial water level in the reservoir is a random variable. For that, we are adjusting several statistical distributions for the annually maximum inflow discharges and also for the maximum water levels in the reservoir. For each combination a flood attenuation is computed and we choose the derogatory result. This algorithm is then repeated for each of the necessary exceedance probabilities for the inflow discharge.

3. The analysis of the maximum annually inflow discharge

For the Darbandi-Khan reservoir we had a set of 52 values of maximum annually inflow discharge, recorded between 1962 and 2013 [2]:

Table 1. Recorded values for maximum annually inflow discharges, 1962 – 2013.

Year	1962	1963	1964	1965	1966
Qmax [m^3/s]	384	1045	876	671	592
Year	1972	1973	1974	1975	1976
Qmax [m^3/s]	1451	756	5816	891	2393
Year	1982	1983	1984	1985	1986
Qmax [m^3/s]	689	546	894	1190	744
Year	1992	1993	1994	1995	1996
Qmax [m^3/s]	1480	913	1190	874	1581
Year	2002	2003	2004	2005	2006

Qmax [m³/s]	638	598	503	2446	1842
Year	2012	2013			
Qmax [m³/s]	343	689			
Year	1967	1968	1969	1970	1971
Qmax [m³/s]	788	913	2751	931	1704
Year	1977	1978	1979	1980	1981
Qmax [m³/s]	393	1183	1492	1438	752
Year	1987	1988	1989	1990	1991
Qmax [m³/s]	923	2028	980	953	1040
Year	1997	1998	1999	2000	2001
Qmax [m³/s]	830	2674	643	413	217
Year	2007	2008	2009	2010	2011
Qmax [m³/s]	553	225	189	441	335
Year					
Qmax [m³/s]					

The minimum value recorded was 189 m^3/s recorded in 1989 and if we ignore the value for 1974 (5816 m3/s) the maximum value was 2751 m^3/s recorded in 1969. If the range to be considered is between 189 and 2751 m^3/s, it is obvious that the value of 5816 m^3/s recorded in 1974 is an outlier, and can be considered as one when adjusting theoretical statistical distributions. So, if only 51 values of recorded maximum discharges are considered, 6 theoretical distributions were adjusted: The Log-Logistic, the Log-Normal, the Inverse Gaussian, The Generalized Extreme Value, the 3 parameters Fréchet and the 3 parameters Pearson distributions. The quality of the adjustments varies from one distribution to another like in figure 2 below:

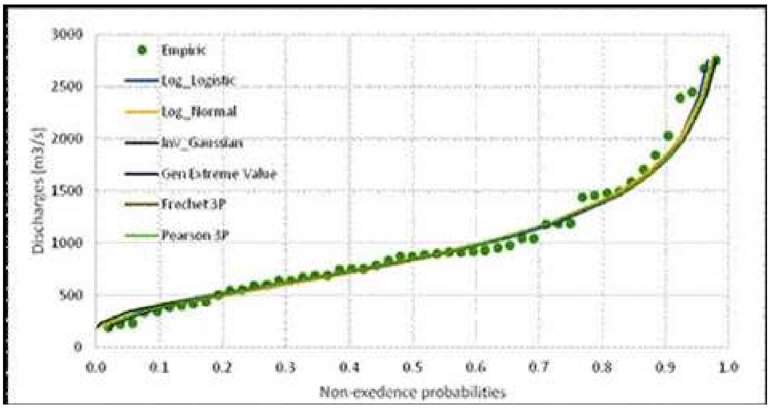

Fig. 2. Non-exceedance probabilities for 51 values of data for the discharges.

According to those 5 distributions, the discharges for several average recurrence intervals are presented in table 2

Table 2. Discharge values for various recurrence intervals when 51 values are used.

Discharge m^3/s	**Log-Logistic**	**Log-Normal**	**Inverse Gaussian**	**Extreme Value**	**Fréchet 3P**	**Pearson 3P**
Q - 1/10	1,823	1,848	1,798	1,791	1,789	1,800
Q - 1/100	4,218	3,542	3,207	3,396	3,448	3,418
Q - 1/1000	9,459	5,700	4,692	5,611	5,823	5,671
Q - 1/10000	21,142	8,432	6,233	8,708	9,265	8,898

The value of 5816 m^3/s recorded in 1974 (see table 1) is somewhere around 100 years average recurrence interval, which is very plausible. The quality of the adjustment was estimated by three GoF criteria, Kolmogorov-Smirnov, Anderson-Darling and the Chi Squared. Based on the results, the Fréchet and the Pearson distributions were adopted further. The adopted values are presented in table 3.

Table 3. The values for the discharges adopted.

Discharge m^3/s	**Fréchet 3P**	**Pearson 3P**
Q - 1/10 [m^3/s]	2,011	2,024
Q - 1/100 [m^3/s]	4,650	4,499
Q - 1/1000 [m^3/s]	9,639	8,782

4. The analysis of the maximum water level in the reservoir

For the same period of time like the maximum discharges, we had 52 values of the maximum water levels in the Darbandi-Khan reservoir:

Table 4. Recorded values for maximum annually reservoir water levels, 1962 – 2013.

Year	1962	1963	1964	1965	1966
Hmax [m ASL]	455.4	484.9	480.82	480.42	475.22
Year	1972	1973	1974	1975	1976
Hmax [m ASL]	487.08	479.85	489.83	486.07	485.96
Year	1982	1983	1984	1985	1986
Hmax [m ASL]	476.83	479	473.87	479.87	478.93
Year	1992	1993	1994	1995	1996

Hmax [m ASL]	479.12	483.61	483.92	484.81	485.13
Year	2002	2003	2004	2005	2006
Hmax [m ASL]	480.33	484.15	477.61	483.53	483.57
Year	2012	2013			
Hmax [m ASL]	481.18	477.82			

Year	1967	1968	1969	1970	1971
Hmax [m ASL]	479.12	483	488.62	481.96	484.37
Year	1977	1978	1979	1980	1981
Hmax [m ASL]	477.07	483.34	480	485.62	475.41
Year	1987	1988	1989	1990	1991
Hmax [m ASL]	480.02	485.09	469.76	479.27	479.59
Year	1997	1998	1999	2000	2001
Hmax [m ASL]	481.8	483.31	469.95	463.94	464.13
Year	2007	2008	2009	2010	2011
Hmax [m ASL]	483.39	468.83	464.95	479.78	477.56

Like for the maximum discharges, several theoretical distributions were adjusted. We used the Generalized Logistic, Generalized Extreme Value, Wakeby, Cauchy, Log-Logistic and Log-Pearson 3 distributions. According to the 3 GoF criteria and to the visual inspection, only two of the distribution gave satisfactory results, the Generalized Logistic and the Wakeby distributions.

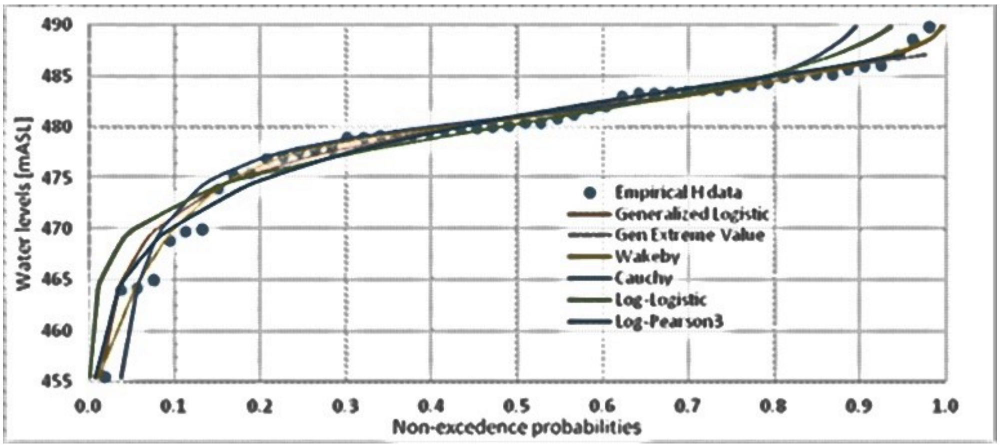

Fig. 3. Non-exceedance probabilities for 52 values of data for the reservoir water levels.

For those two distributions, the water levels according to several years average recurrence intervals were computed, and further adopted – the values are presented in table 5:

Table 5. The values for the reservoir water levels adopted.

	Generalized Logistic	Wakeby
H - 1/10 [m ASL]	486.10	486.26
H - 1/100 [m ASL]	489.19	489.13
H - 1/1000 [m ASL]	490.79	489.97

5. The dimensionless inflow hydrograph

For the inflow hydrograph ordinates, it was used the design hydrograph Q $_{10000}$, indicated by design data as the PMF (figure 4).

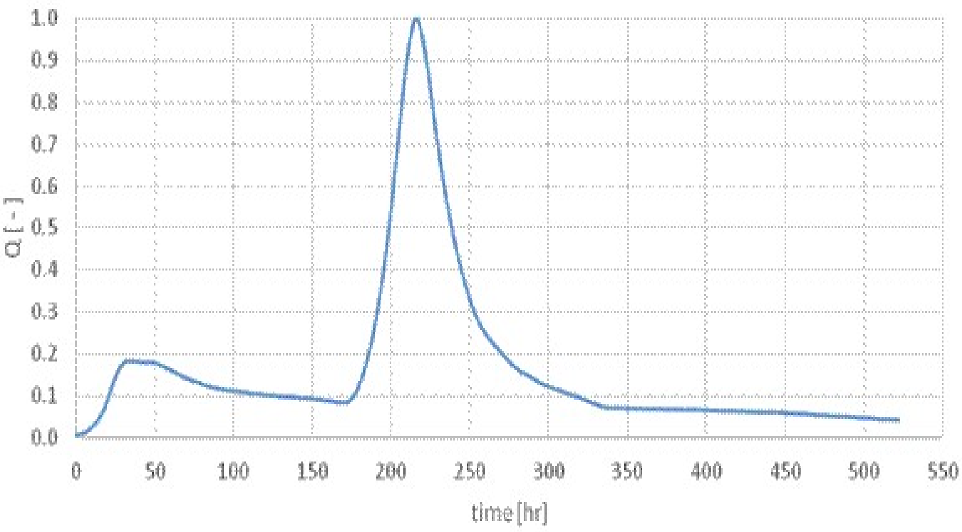

Fig. 4. Dimensionless hydrograph.

6. Data concerning the reservoir volume and the capacity of the dam spillway

The reservoir volume curve (elevation – storage) and the spillway capacity versus reservoir elevation are shown in figures 5 and 6.

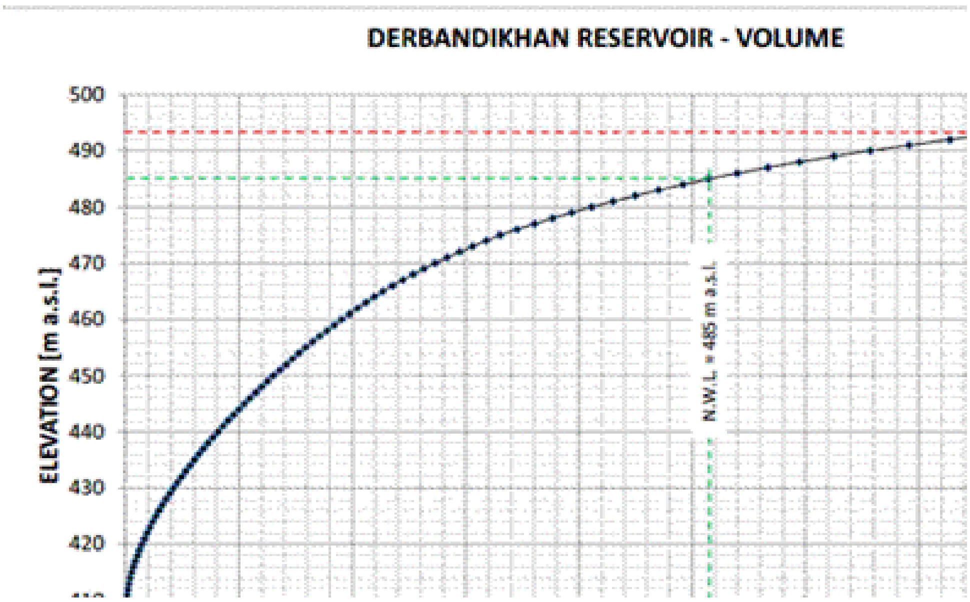

Fig. 5. Reservoir volume curve.

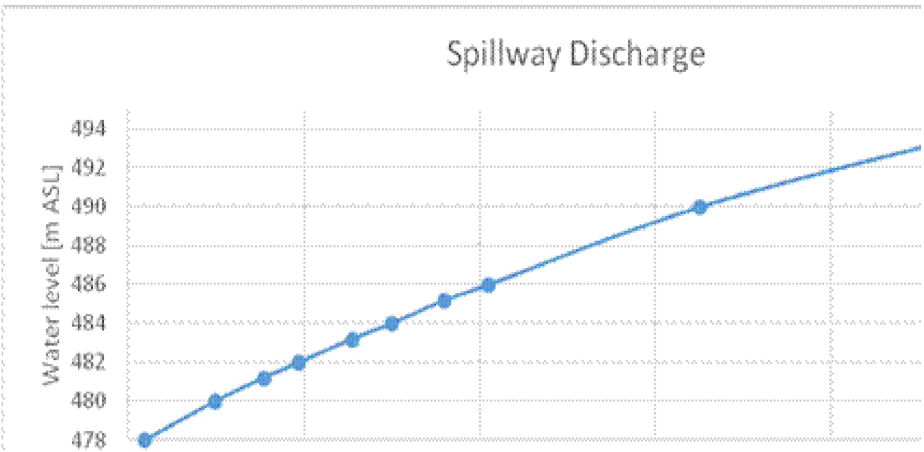

Fig. 6. Spillway discharge/water level relation.

7. Non-exceedance probabilities for attenuated outflow

The 0.01% exceedance probability of the design flow event were evaluated based on several combinations of discharge inflow and reservoir level listed in table 6:

Table 6. Combinations for the inflow discharge and the initial reservoir water levels.

Name	T [years]	Q [m^3/s]	Distribution		T [years]	H [m ASL]	Distribution
Q_1	10	2011	Fréchet	with	1000	490.79	GenLogistic
Q_2	10	2024	Pearson		1000	490.79	GenLogistic
Q_3	10	2011	Fréchet		1000	489.97	Wakeby
Q_4	10	2024	Pearson		1000	489.97	Wakeby
Q_5	100	4650	Fréchet		100	489.19	GenLogistic
Q_6	100	4499	Pearson		100	489.19	GenLogistic
Q_7	100	4650	Fréchet		100	489.13	Wakeby
Q_8	100	Pearson	4499		100	489.13	Wakeby
Q_9	1000	Fréchet	9639		10	486.1	GenLogistic
Q_10	1000	Pearson	8782		10	486.1	GenLogistic
Q_11	1000	Fréchet	9639		10	486.26	Wakeby
Q_12	1000	Pearson	8782		10	486.26	Wakeby

The flood routing through reservoir was computed by using the Modified Puls method. The combinations in table 6 for the discharge inflow and the initial water level values were analyzed one by one and the results were as follows.

When the combination between the 1/10 average recurrence value is used for discharges and the 1/1000 average recurrence value for initial reservoir water level (the combinations denoted by Q_1 to Q_4 in table 6), the inflow and the outflow hydrographs are practically similar, no attenuation is made, because of the small volume of flood (figure 7): The maximum outflow discharge is 1921.4 m^3/s according to the maximum inflow discharge of 2024 m^3/s.

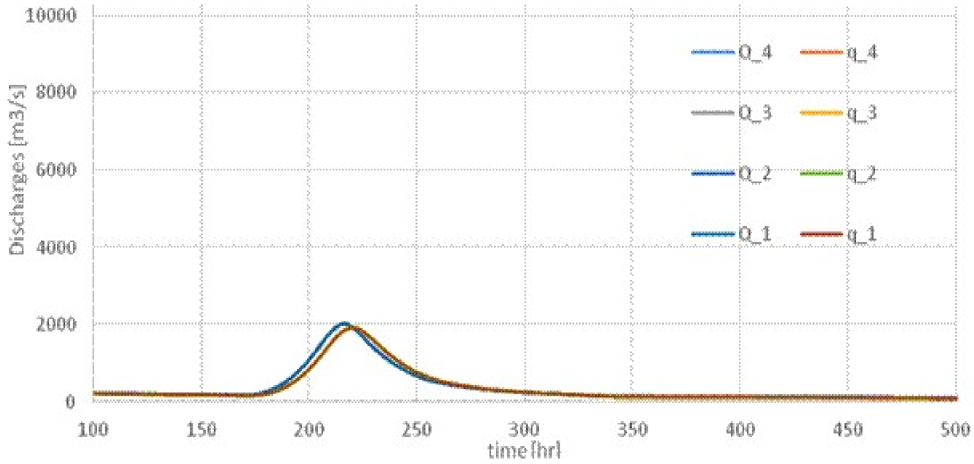

Fig. 7. Flood attenuation results for Q (1/10) and H (1/1000).

When the combination between the 1/100 average recurrence value is used for discharges and the 1/100 average recurrence value for initial reservoir water level (the combinations denoted by Q_5 to Q_8 in table 6), the attenuation is visible, but still the transported water volumes are small (fig. 8). The maximum outflow discharge is 3384.3 m^3/s according to the maximum inflow discharge of 4650 m^3/s.

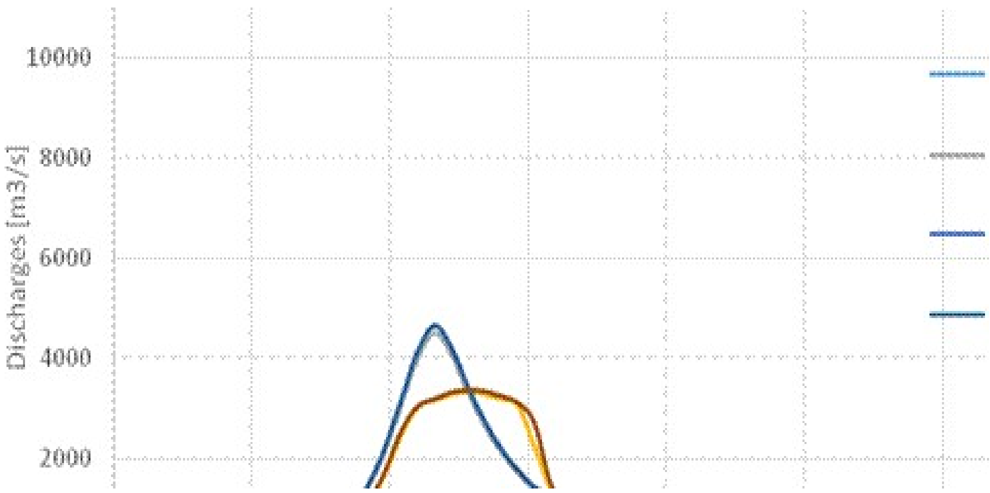

Fig. 8. Flood attenuation results for Q (1/100) and H (1/100).

Finally, the last combination of inflow discharge - initial water level was analyzed, considering the 1/1000 average recurrence value for discharges and the 1/10 average recurrence value for initial reservoir water level (the combinations denoted by Q_9 to Q_12 in table 6). Now there is a significant attenuated volume, the results are shown in figure 9. The maximum outflow discharge is 5298 m^3/s according to the maximum inflow discharge of 9639 m^3/s.

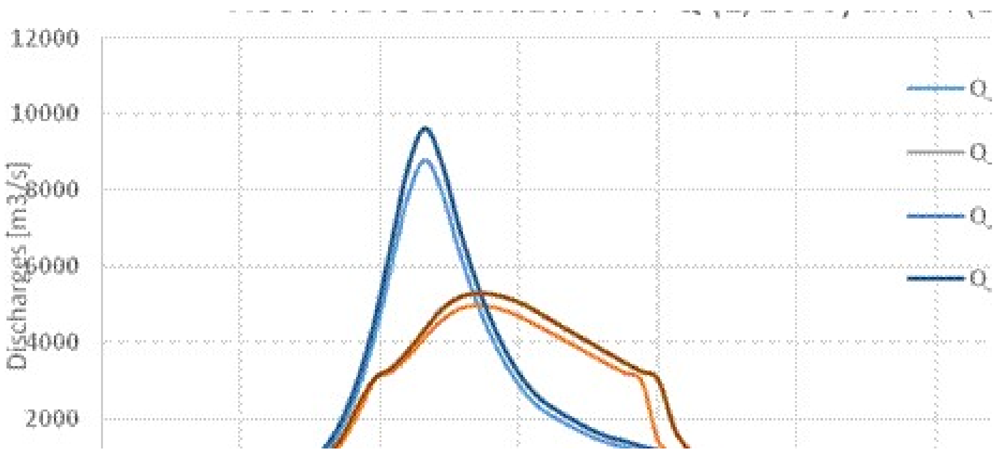

Fig. 9. Flood attenuation results for Q (1/1000) and H (1/10).

The maximum outflow from Darbandikhan reservoir corresponds to 1/1000 average recurrence value for discharges and the 1/10 average recurrence value for initial reservoir water level. The maximum outflow discharge is 5298 m^3/s according to the maximum inflow discharge of 9639 m^3/s (figure 10). The inflow hydrograph is denoted by the capital Q and the outflow hydrograph by small q.

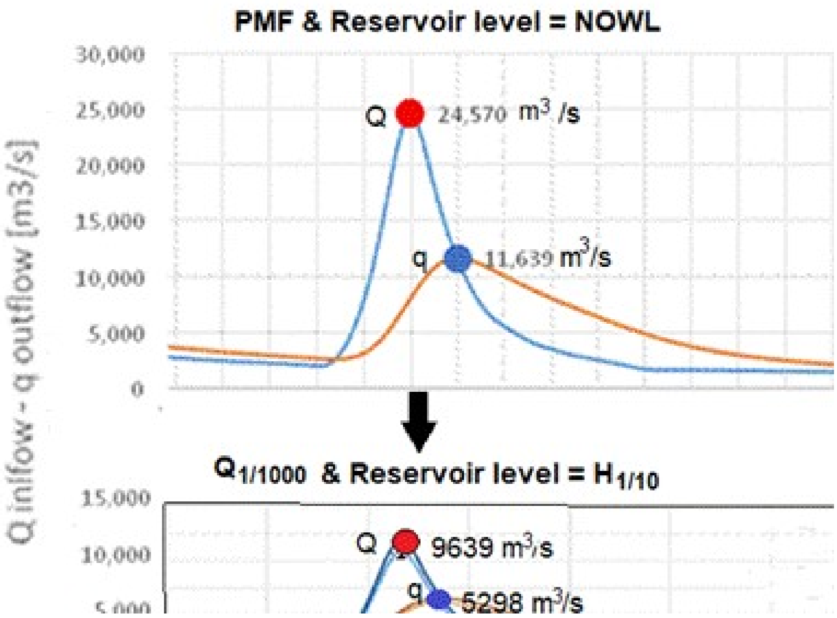

Fig. 10. Inflow (Q) and outflow (q) hydrographs corresponding to the two approaches.

The values are significantly smaller than the ones corresponding to PMF routing with the initial water level in the reservoir at the normal highest full reservoir level. To conclude, considering both the discharge and the water level as independent random variables the 1/10000 event is more realistic evaluated, having as result a significant reduction of the outflow discharges [3].

8. Concluding remarks

In order to match the safety requirements imposed by the engineering practice the improved alternative is based on additional hydrological data acquired during the upstream reservoir operation. It is based on a more realistic approach of the flood routing through the upstream Darbandikhan reservoir that takes into account the variability of the reservoir level independent to the flood inflow event arrival.

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