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Hydraulic Operation of the Spillway for the Lom Pangar Dam in Cameroon

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Abstract: *This paper lists the restitution capacity of the spillway at different times and then shows by calculation that it is functioning properly. The Numerical modelling of the spillway was carried out using the HEC-RAS software of the US Army Corps of Engineers (USACE) and is limited to a single passage of 8.75m wide. The aim of the calculation is to obtain the flow conditions at the entrance of the restitution bucket where the water flow is at supercritical depth.*

Keywords: *Spillway, flood dissipation, cavitation, bucket, ski jump, Lom Pangar.*

I. Introduction

Lom Pangar is a dam in the Eastern Region of Cameroon and its water is used in the regulation of the downstream water level during the dry season for the Sanaga river. For the evacuation of flood, it is equipped with a surface spillway (Creager classic type) designed for a deca-millennial flood. It consists of:

- 4 Passages 8.75m wide, each equipped with a segment valve of which the height is fixed at altitude 665.75 CGL (Cameroon geodesic level).
- 3 flood discharge gates (2 x 5.9m x4m) and (1 x 3m x2m) at CGL 640 and 643 respectively.
- A free overflow passage of 11m wide on the right bank at altitude 672.70 CGL fitted with a Hydroplus fuse.
- For the hydroplus fuse to tilt, the water level must attain 674.55 CGL at which time the freeboard will be a little above 3m as the crest of dam is at 677.55 CGL

The hydraulic regime on the threshold passage is determined by the flow control section situated at the level of the pier: at this point the draft height is equal to the calculated critical height of the evacuated flow.

II. Evacuation Capacity

The evacuation capacity of the spillway at full overture for the different feasible configurations are shown in the figure 1

At normal reservoir retention the discharge capacity of each gated passage is 320m^3/s.

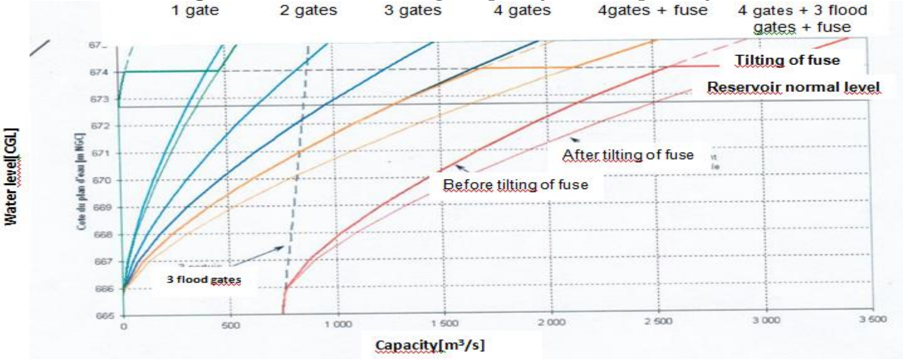
At normal conditions of exploitation (4 gated passage) the discharge capacity is 1280m^3/s plus 3 flood discharge gates with a total capacity of 860m^3/s making a total of 2140m^3/s..

III. Analysis Of The Different Flood Situations

A lamination of the different flood returns for 100, 1000 and10,000 years has been studied for the following configurations:

- Normal conditions: all the hydromechanics equipment are operational
- Special conditions: one of the passageway gates is supposedly blocked

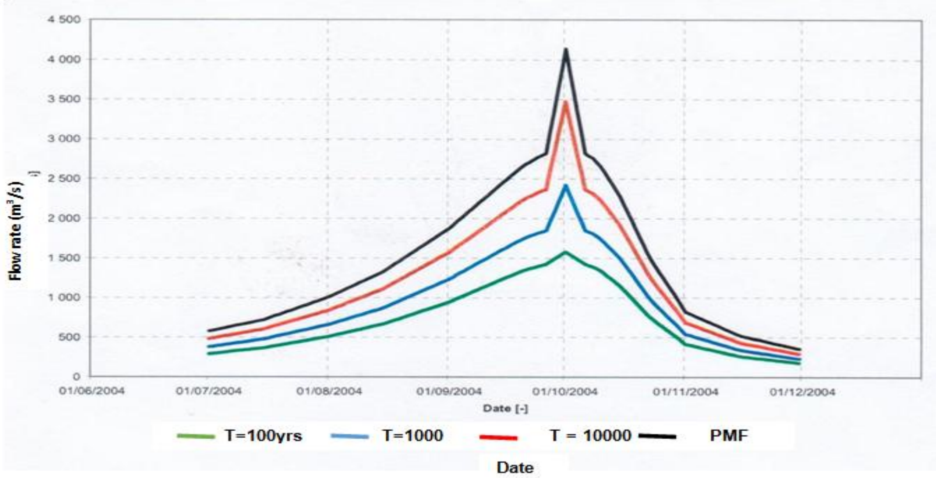
Figure 1: Flood Discharge capacity of the spillway



IV. Flood Hydrographs

A hydrograph for the different flood used (From studies and simulations carried out in pre-project doc No. 10108-NDC-0100)

Figure 2: Hydrograph of flood (T=100, 1000, 10,000, and PMF)



The flood dissipation capacity was then analysed for normal conditions (4 segment gates operational) and special conditions (3 segment gates operational) as shown below

Figure 3 : Analysis of the PMF – Normal conditions (4 gates + 3 flood discharge gates + supplementary outlet)

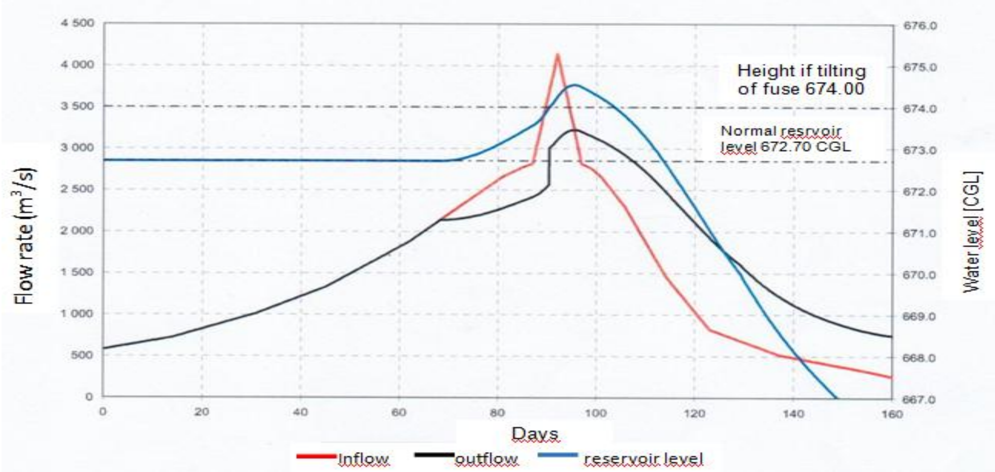


Figure 4: Analysis of the PMF – Exceptional conditions (3 gates + 3 flood discharge gates + supplementary outlet)

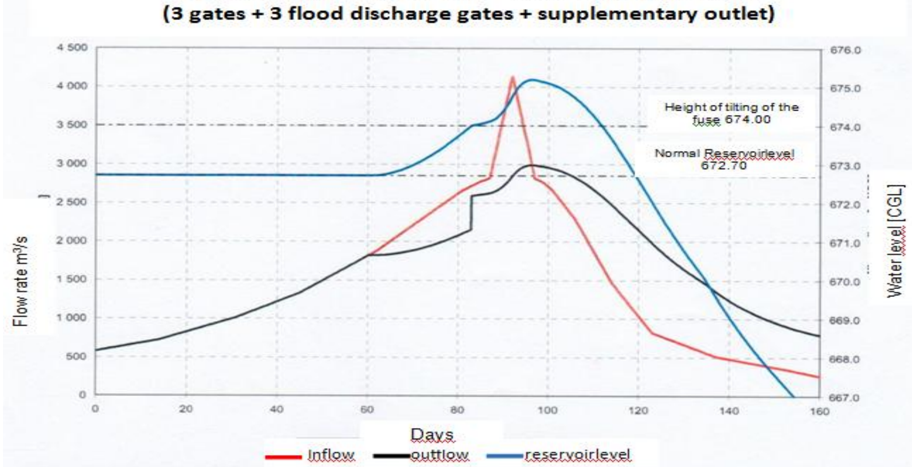
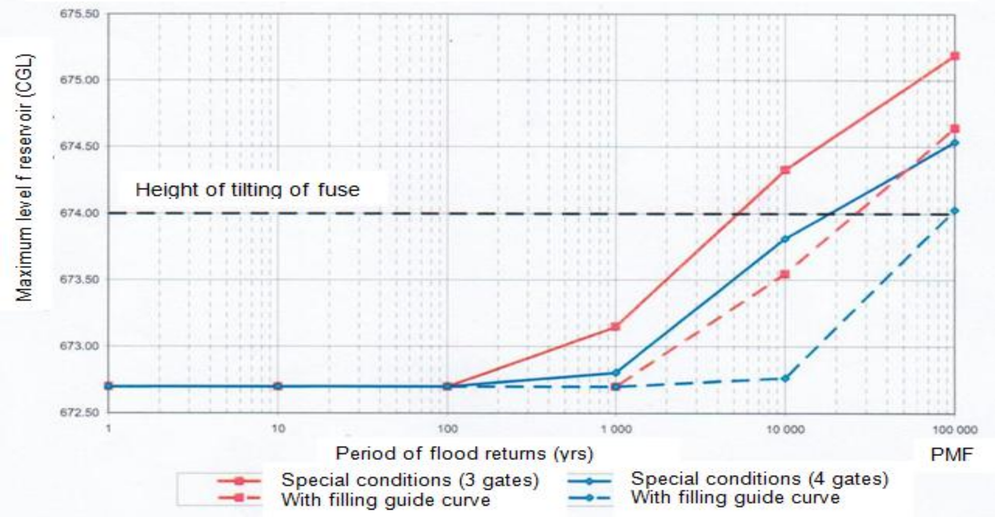


Figure 5: Maximum water level with respect to the return flood



Water level during 1/10000 year flood returns reach the height of 674.33 and if one of the gates is non-operational, this could cause a tilting of the fuse, a very rare condition.

V. Hydraulic Functioning Of The Spillway[3]

5.1 Numerical modelling

The principal hypotheses used in this modelling are as follows: (i) the surface of the water line is modelled in steady state for laminated flow (ii) the Strickler coefficient (of concrete) in the passageway of the spillway is considered being equal to 70 (iii) The entrance of the bucket is fixed at altitude 647.50 CGL.

5.2 Analytical method

In applying the Bernoulli equation, the results obtained are identical to some degree, to the section situated between upstream of spillway (Z=672.70 CGL, V=0) and the bucket taking into account the coefficient of load loss is equal to 0.9.

VI. Studying The Risk Of Cavitation

Defining the cavitation index

The cavitation index σ_1 is non-dimensional value defined classically in free surface flow as follows:

$$\sigma_1 = \frac{P_{atm} + h - P_{vap\,sat}}{\frac{v^2}{2g}} \qquad (1)$$

Where:

P_{atm}	atmospheric pressure mH_2O,
H	height of water flow in mH_2O,
$P_{vap\ sat}$	vapour saturation pressure in mH_2O,
v	average speed in m/s,
g	acceleration due to gravity

The recommendations by US Bureau of Reclamation concerning the cavitation index are as follows:

- For all projects in which $\sigma_1 < 0.1$ must be reconsidered
- Projects having $0.1 < \sigma_1 < 0.2$ need measures of aeration to limit cavitation risks
- Projects in which $\sigma_1 > 0.2$ may not need aeration on condition that the concrete finishing is in adequacy with the value of σ_1.

VII. Study Of Cavitation Index Along The Spillway

The water height and speed are determined along the spillway for millennial and deca-millennial flood from hydraulic simulations realized with the HEC-RAS software. The sections used for calculations of the cavitation index correspond to the construction sections defined in the software. The sections used for the calculation correspond to a block from the entrance to the spillway to the entrance to the bucket.

Table 1: Study of cavitation risk from upstream to downstream

Sections considered	Return period	V	h	σ_i	Cavitation
	[Years]	[m/s]	[mH_2O]	[-]	[-]
7	T=1 000 yrs T =10 000 yrs	7.20 7.53	5.10 4.87	5.71 5.14	$\sigma_{i > 0.2}$ $\sigma_{i > 0.2}$
6	T=1 000 yrs T =10 000 yrs	8.39 8.73	4.37 4.20	4.00 3.65	$\sigma_{i > 0.2}$ $\sigma_{i > 0.2}$
5	T=1 000 yrs T =10 000 yrs	9.79 10.13	3.75 3.62	2.81 2.61	$\sigma_{i > 0.2}$ $\sigma_{i > 0.2}$
4	T=1 000 yrs T =10 000 yrs	11.17 11.49	3.28 3.19	2.08 1.96	$\sigma_{i > 0.2}$ $\sigma_{i > 0.2}$
3	T=1 000 yrs T =10 000 yrs	12.56 12.87	2.92 2.85	1.61 1.53	$\sigma_{i > 0.2}$ $\sigma_{i > 0.2}$
2	T=1 000 yrs T =10 000 yrs	13.96 14.27	2.63 2.57	1.27 1.22	$\sigma_{i > 0.2}$ $\sigma_{i > 0.2}$
1	T=1 000 yrs T =10 000 yrs	16.05 16.34	2.29 2.25	0.94 0.90	$\sigma_{i > 0.2}$ $\sigma_{i > 0.2}$
0	T=1 000 yrs T =10 000 yrs	20.79 21.05	1.76 1.74	0.53 0.53	$\sigma_{i > 0.2}$ $\sigma_{i > 0.2}$
-1	T=1 000 yrs T =10 000 yrs	21.48 21.75	1.71 1.69	0.49 0.49	$\sigma_{i > 0.2}$ $\sigma_{i > 0.2}$
-2	T=1 000 yrs T =10 000 yrs	21.66 21.93	1.69 1.67	0.49 0.47	$\sigma_{i > 0.2}$ $\sigma_{i > 0.2}$
-3	T=1 000 yrs T =10 000 yrs	21.72 21.99	1.69 1.67	0.49 0.47	$\sigma_{i > 0.2}$ $\sigma_{i > 0.2}$

The conditions of non-cavitation are verified in each of the sections of the spillway. It is consequently not necessary to envisage the setup of an aerator.

VIII. Analysis Of The Downstream Restitution

In this section, the main results presented are applicable to the evacuation bucket defined by horizontal cylinder of radius 12m of which the angle of exit of water from the jet is equal to 31^0.

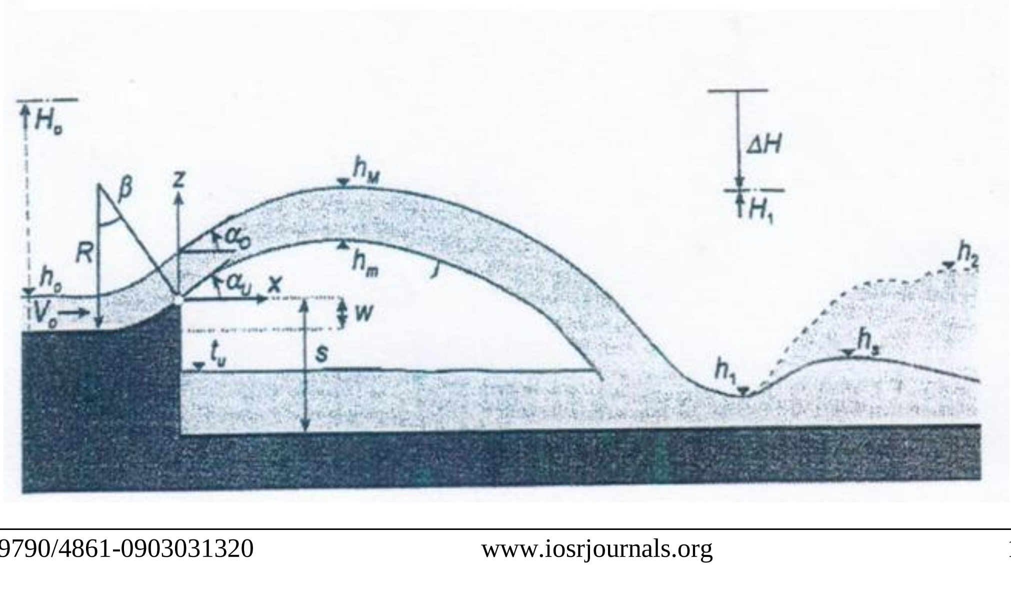
8.1 Recall on the hydraulics of ski jump[2][5[7][8][9]

This type of analysis carries the following conclusions

- The repartition of dynamic pressure at the bucket,
- The conditions of forming a hydraulic jump at the bucket,
- The characteristics of the trajectory of the jet water coming out of the bucket,
- The height of waves at the downstream of flow regulatory structures

The sketch of figure 6 represents the flow in coming out of the bucket of a ski jump and is the set of notations used hereafter.

Figure 6: Flow pattern in coming out of the bucket in ski jump: notations are used hereafter



8.2 Maximum dynamic pressure along the bucket

The theoretical dynamic pressure H_{PT} exerted by a flow rate Q on a bucket of radius of curvature R is calculated as follows:

$$\frac{H_{PT}}{h_0} = B_0^2 \quad (2)$$ [4]

With $B_0 = \sqrt{\frac{h_0}{R}} \times F_0$ where B_0 is the number curvature

Experience has shown that the maximum pressure observed, noted H_{PM} is deduced from the Term H_{PT} as follows:

$$\frac{H_{PM}}{H_{PT}} = \frac{R}{5.h_0} \times \frac{\beta}{40^0} \text{ (on condition that } \frac{R}{h_0} \times \frac{\beta}{40^0} \geq 0.20) \quad (3)$$

The position X_{PM}, completed negatively since the coming out from the bucket to the point of maximum dynamic pressure for an angle of curvature greater than $15^0$is given by the relation below:

$$\frac{X_{PM}}{R.sin\beta} = -(\frac{1.5^0}{\beta})^{1.5} \quad (4)$$

8.3 Characteristic of the trajectory of the jet on coming out of the bucket[4]

If we neglect the friction of air on water, the layers of water inferior or superior to the water jet describe, on coming out of the bucket, a parabola which can be represented using the ballistic equation:

$$z(x) = z_0 + x.\tan(\alpha_j) - \frac{gx^2}{2.V_0^2.cos^2(\alpha_j)} \quad (5)$$

where:

x	horizontal distance
z	height
z_o	height of point of departure of jet
V_0	flow speed at the entrance of bucket
α_j	angle of water blades with respect to the horizontal, on coming out of the bucket

In order to take into account the non-negligible effects of friction due to air at this speed, the velocity at upstream of bucket V_0 is corrected by coefficient taken equal to 0.85. The angle of water of the lower layer, denoted as α_U, as well as that of the upper layer denoted as α_o are given by the following equations:

$$\frac{\alpha_o}{\beta} \times (\frac{70^o}{\beta})^{1/6} = \frac{1}{2}[1 + \exp(-8(h_o/R)^2)] \text{ for } 0 \leq h_o/R \leq 1 \quad (6)$$

$$\frac{\alpha_U}{\beta} \times (\frac{140^o}{\beta})^{1/6} = \frac{1}{2}[1 + \exp(-8(h_o/R)^2)] \text{ for } 0 \leq h_o/R \leq 1 \quad (7)$$

Where h_o is the height of water at the entrance of bucket

8.4 Height of waves downstream and at zones of recirculation

At the zone of impact, the flow generates turbulence and shockwaves. Observations from reduced models have permitted us obtain a relation (8) linking the height of the wave downstream to the Froude number at the entrance of the bucket and the angle of deflection of the bucket. Similarly, the expression (9)permits us to determine the height of the recirculation zone to the jet situated between the point of impact of the jet and the downstream facing of the structure.

$$(\frac{h_s}{h_o} - 1)\sqrt{sin\beta} = 0.85\,(F_o - 1) \qquad \text{for } 2 \leq (F_o - 1) \leq 10 \quad (8)$$

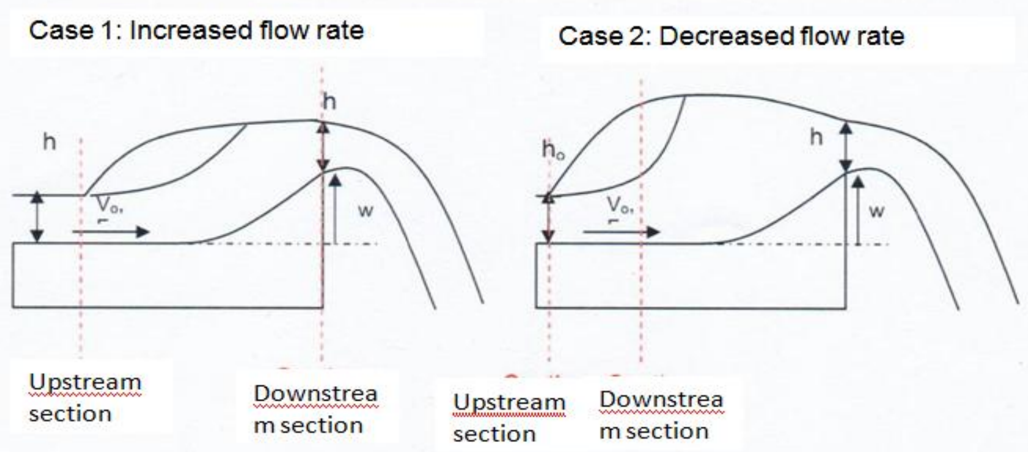
$$\frac{t_u}{h_o} = (5F_o^{-1})^2 \qquad \text{for } 3 \leq F_o \leq 10 \quad (9)$$

8.5 Condition for the formation of a hydraulic jump in the bucket

Classically, each ski jump is designed so that the flow remains in torrential flow in the bucket so as to propel water as far as possible thus avoiding erosion at the foot of the downstream facing. In case where the flow at the entrance of the bucket does not have a large Froude number, a hydraulic jump can occur and inundate the bucket. The jet does not take off and the water is disbursed at the extreme of downstream of the bucket. Two types of flow are possible:

- Case 1: Flow increase regime
- Case 2: Flow decrease regime

Figure 7: Condition of forming a hydraulic jump in the bucket



The application of Euler's theory to the volume comprised between the upstream and downstream sections in a hypothesis of forming a hydraulic jump leads us to the following results:

Case 1: $W^+ = \frac{W}{h_o} = 0.6(F_o - 1)^{1.2}$ $\quad 1 < F_o < 4$

Case 2: $W^+ = \frac{W}{h_o} = 0.9(F_o - 1)^{0.9}$ $\quad 1 < F_o < 4$

For a given geometry and characterised by its W / h_o ratio, we are capable of determining the flow rate at which a jump is formed in the bucket. For flows superior to this value, the flow remains torrential.

IX. Evaluating The Risk For A Hydraulic Jump In The Bucket

The following table synthesizes the results obtained. The conditions of formation of a hydraulic jump is never verified except for a flow of 10m^3/s.

Table 2: Evaluation of the formation of a hydraulic jump in a bucket

Flowrate per passage	**h_o**	**F_o**	**W**	F_o^+	F_o^-	**Jump?**
[m^3/s]	**[m]**	[-]	[-]	[-]	[-]	
10.00	0.18	5.39	2.17	12.06	13.19	$F_o < F_o^+, F_o^-$
50.00	0.31	9.92	2.17	7.00	8.75	$F_o > F_o^+, F_o^-$
100.00	0.59	8.14	2.17	3.69	5.53	$F_o > F_o^+, F_o^-$
150.00	0.88	6.66	2.17	2.47	4.25	$F_o > F_o^+, F_o^-$
200.00	1.14	5.96	2.17	1.90	3.62	$F_o > F_o^+, F_o^-$
250.00	1.40	5.24	2.17	1.55	3.21	$F_o > F_o^+, F_o^-$
300.00	1.71	5.30	2.17	1.27	2.87	$F_o > F_o^+, F_o^-$
350.00	1.85	5.15	2.17	1.17	2.75	$F_o > F_o^+, F_o^-$
381.00	1.99	4.99	2.17	1.09	2.65	$F_o > F_o^+, F_o^-$

9.1 Maximum dynamic pressure

Maximum dynamic pressures applicable to buckets are presented in the table below:

Table 3: Maximum dynamic pressure in the bucket

Flow rate per passage	**Radius**	**Angle of curvature**	**F_o**	**h_o**	**H_{PT}**	**H_{PM}**	**X_{PM}**
[m^3/s]	**[m]**	**[º]**	**[-]**	**[m]**	**[mH_2O]**	**[mH_2O]**	**[m]**
321	12	31	5.26	1.69	6.59	7.25	0.07
381	12	31	4.99	1.67	5.79	6.45	0.07

9.2 Zone of circulation and height of downstream waves

The level of water downstream has been determined with the help of the flood calibration curve in supposing at the same time the downstream regulation sluices used.

Table 4: Level of water and the height of the downstream waves

$Q_{spillway}$ per passage	**Q_{sluice}**	**Q_{Global}**	**$N_{downstream}$**	**t_u**	**Water height downstream**	**Height of waves on impact h_s**
[m^3/s]	**[m^3/s]**	**[m_3/s]**	**[mCGL]**	**[m]**	**[m]**	**[m]**
321	885	2172	642.3	1.53	8.30	1.92
381	897	2436	643.4	1.68	9.40	0.16

9.3 Trajectory of water layers

Two calculations of the trajectory have been carried out for the each of the two layers of water in order to take into account the friction of air on water.

Table 5: Outlet angles of the inferior and superior layers

Flow rate	h_o	V_o	F_o	α_u	α_o
[m^3/s]	[m]	[m/s]	[-]	[º]	[º]
321	1.69	21.72	5.26	25.08	23.34
381	1.67	21.99	4.99	25.12	22.38

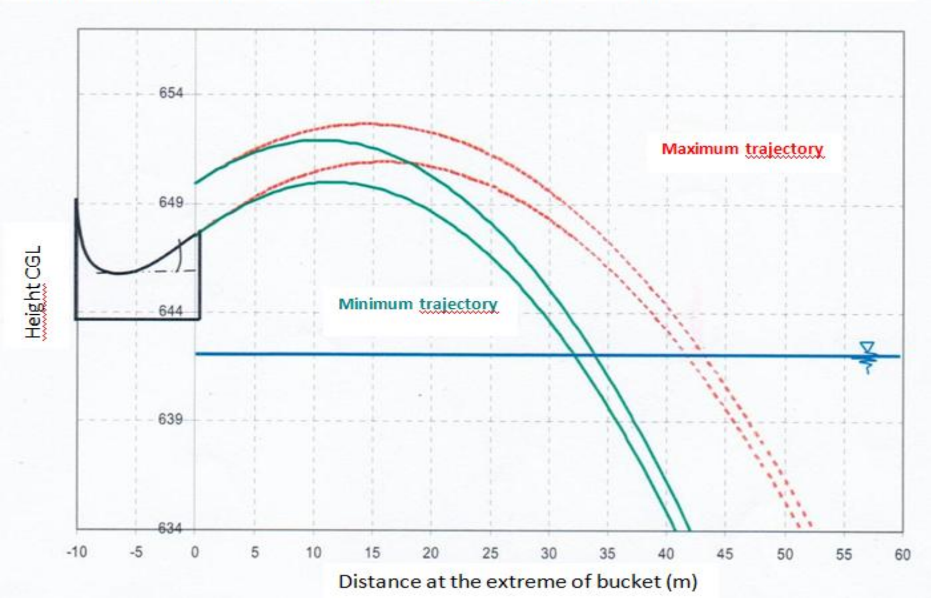
Curves of maximum and minimum trajectories of water coming out of the bucket have been traced for a flow rate of 381 m^3/s which corresponds to deca-millennial flood. The results presented in the table below take into account air to water friction (Figure 9).

Table 6: Spillway – distance between the point of impact of jet and the outlet point of bucket.

Return Period	Laminated flow rate	Global flow rate	Water level downstream	t_u	Distance of impact of inferior layer	Distance of impact of superior layer
[Yrs]	[m/s]	[m/s]	[mCGL]	[m]	[m]	[m]
10 - 10000	321	2172	642.3	1.53	33	34
10000	381	2436	643.4	1.68	31	32.5

The distance of the jet coming out of the bucket is 30 to 35m irrespective of the flood returns.

Figure 8: Trajectories of water layers for a deca-millennial flood



9.4 Depth limit of erosion pits[1][6]

At the level of the impact zone of the jet with the ground, for rare flood, the force of the jet is dissipated in a water mattress downstream creating an erosion pit. The depth of the limit of the scour is calculated using the empirical formula of Veronese after establishing stationary conditions:

$$t + h_2 = 1.9.H^{0.225}.q^{0.54} \qquad (10)$$

with:

- t scour depth measured from the altitude of natural ground
- h_2 depth of downstream water level
- q specific flow rate
- H height difference between normal reservoir water level and downstream water level

Table 7: Spillway – Scouring depth of the erosion pit

Return period	Laminated flow rate	Upstream water level	Unit flow	Depth of scour
[Yrs]	[m_3/s]	[mCGL]	[m3/s.m]	[m]
10 - 10000	321	672.70	36.70	27.10
10000	381	673.55	43.50	29.70

The scour is assumed to be centred on the zone of impact of the jet with the downstream water level. To determine the width of the erosion pit, a final slope of 1/1 is adopted. The results show that the zone concerned by scouring of the downstream soil does not reach the foot of the downstream structure. The impact of the jet is far enough away from the bucket during flood evacuation.

X. Conclusion

Even when the worst of situations occur which is Probable Maximum Flood (PMF), there will be no scouring at the downstream of the dam. During flood the water level does not usually attain the exceptional level of 674.00 CGL. But during deca millennial flood or during PMF the water level can reach 675.45 which may result in the tilting of the fuse. However this is a very rare situation. Also there is no risk of cavitation during flood dissipation as can be seen from values obtained from the experiments and findings for the cavitation number $(\sigma) > 2$. This then gives us the assurance that the spillway dissipation of energy during heavy floods can be guaranteed over the years.

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