

Coupled spillway devices and energy dissipation system at St-Marc Dam (France)

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ABSTRACT: The physical modeling tests for the rehabilitation of St-Marc Dam with a PKW are presented. A particular focus is put on the energy dissipation downstream from the PKW. The adopted solution is a leaned “ski-jump gutter” placed at the contact line between the downstream face of the dam and the natural foundation rock. It consists of a cylindrical profile, developed around an inclined axis and closed by a horizontal reach at the end of the structure. The aim of this solution is to guide the flow issued from the PKW to the stilling basin of the left existing spillway. The experimental tests consider various operation conditions which required pressure measurements at different impact zones.

Structural design of the PKW is impacted by the fact that the concrete of the dam is subjected to a noteworthy blowing reaction. For that reason, the new structure could not be anchored in the existing dam. Thus, the spillway behaves as a gravity structure.

1 INTRODUCTION

The Saint-Marc dam, property of EDF (France) is a 40 m high concrete gravity dam with an overall crest length of 170 m. It was built between 1926 and 1930 and is located 20 km upstream of Limoges, France (Fig. 1 left). The created reservoir covers an area of 150 ha, for a retention volume of 20 mio.m³. The power plant is located at the toe of the dam providing 13.5 MW through three Francis turbines. The original spillways include 3 sluices with Creager type crests: one 7.5 m wide equipped with a radial gate on the right bank and two identical 10 m wide also fit out with radial gates. Normal and maximum water levels are fixed at elevation 282.00 and 283.50 m NGF respectively, while spillway crests level lies at 278.50 m NGF. Before the construction of the PKW, the maximum spillway capacity was 623 m³/s.



Figure 1. Downstream view of St-Marc dam with the original spillway devices (left), and with the additional PKW (right).

The update of hydrological studies according to the Gradex method (CFGB, 1994) has shown that the existing discharge capacity is smaller than the 1'000 years return period design flood ($750 \text{ m}^3/\text{s}$). In order to satisfy the required dam safety, EDF-CIH undertook a feasibility study to provide an additional discharge work to the existing spillways. A technical-economical comparison of several designs was carried out, among them the Piano Key Weir (PKW) turned out to be the best solution (Fig. 1 right). This alternative presents a high performance regarding risk analysis (reliability of free surface flow spillway) and maintenance costs (no mechanical or electrical device).

The PKW introduces however particular hydraulic behaviors requiring special attention. Among them, aeration of the jet napes issued from the weir crest as well as reception and guidance on the downstream part of the dam have to be mentioned. The overflow also requires an adapted energy dissipation device, in good agreement with existing works at the toe of the dam.

2 DESIGN CRITERIA AND EXPERIMENTAL RESULTS

2.1 *Experimental setup and device*

The experimental model of the St-Marc's Dam constructed at LCH includes part of the reservoir, the dam body, the existing spillways and the designed PKW (crest at elevation 282.15 m NGF). On the downstream side, the model reproduces the stilling basin of the left spillway, the channel of the right bank spillway and the current topography of the valley. The model extends approximately 130 m upstream and downstream of the dam (Fig. 2 left).

The existing gated spillways and the PKW, as well as the stilling basin and channel are made of PVC. The topography of the reservoir and the reach of the Taurion River downstream of the dam are reproduced with a cement cover. The experimental setup is constructed with a geometrical scale factor 1:30 (Fig. 2 right). The model operates with respect to Froude similarity.

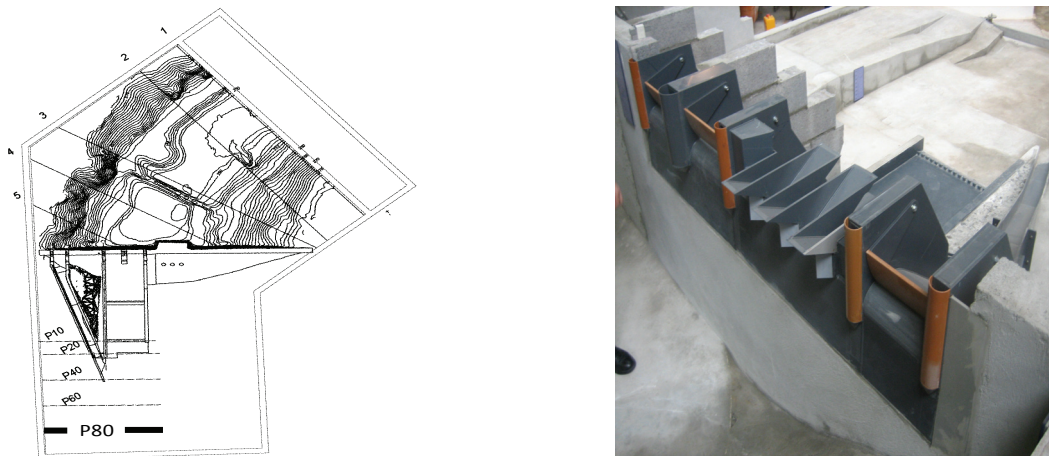


Figure 2. Limits of the hydraulic model (left) and upstream view of the spillway devices, including the PKW (right).

2.2 *Upstream and downstream flow conditions*

The upstream boundary conditions of the spillway devices revealed to be positively influenced by the PKW operation during floods. This impact could be put in evidence by the analysis of the superficial flow field velocities based on LSPIV (Large Scale Particle Image Velocimetry) measurements (Kantoush et al., 2008). Tests were conducted for different spillway operations, with and without PKW. When comparing the average flow fields it could be noticed that before the construction of the PKW, the main flow is deflected to the left side of the basin (Fig. 3 left). Moreover, the approach velocity is reduced near the gates due to a circulation cell forming in the right corner. As a result of the PKW construction, the flow patterns completely change. The flow orientates more straight and perpendicular to the gates, the right corner circulation is suppressed and the approach velocity is more uniform (Fig. 3 right).

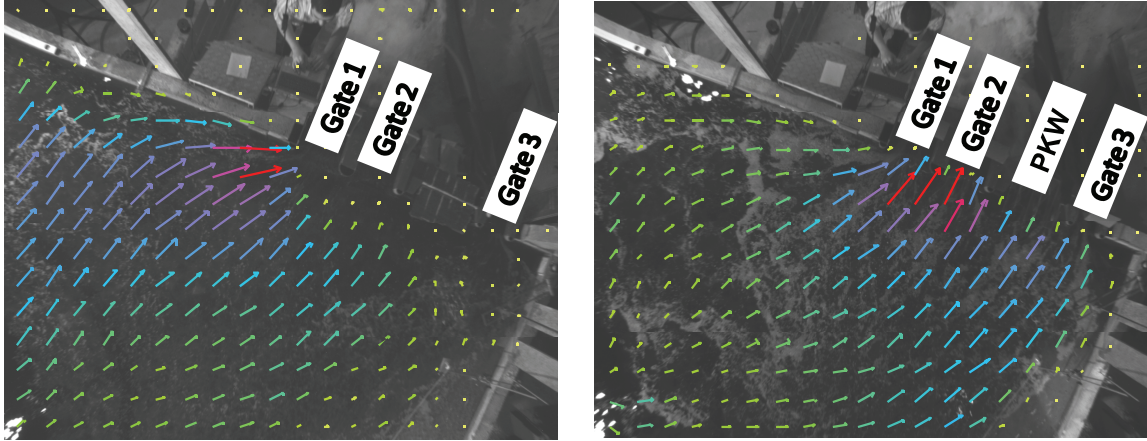


Figure 3. Average flow pattern in the reservoir without (left) and with PKW (right) at maximum operation level.

The downstream hydraulic boundary conditions regulated on the model have a significant influence on the energy dissipation at the toe of the dam. For that reason, the hydraulic profiles of the Taurion River were computed by numerical *Hec-Ras* 1D-Simulation (USACE, 2002) over the reach comprised between the next reservoir of Chauvan and cross section P80 corresponding to the downstream limit of the model (Fig. 2 left). Different river roughness coefficients were considered in the frame of a sensitivity analysis, varying the Manning's coefficients in a $\pm 20\%$ range. Computations confirm that the water level at dam of Chauvan does not modify the flow conditions at the outlet of St-Marc. However, the bridge St-Martin Teressus located in the downstream vicinity influences the rating curve at section P80. The roughness sensitivity analysis reveals a maximal difference of 30 cm in the range of highest discharges. For the hydraulic tests, the smoothest configuration was adopted, placing the results on the safe side concerning the energy dissipation behavior at the toe of the dam.

2.3 Spillway capacity

2.3.1 PKW capacity

The PKW design was constrained by the available space on the dam crest as only a 20 m wide pass was affordable. Two different shapes of PKW, a trapezoidal and a rectangular, were tested in the model (Leite Ribeiro et al., 2009; Laugier et al., 2009). The final rectangular design includes two wide inlet keys and three smaller upstream ones (Fig. 2 right). The bottom part of the PKW is 5 m wide. The PKW sidewalls are 12.05 m long and 4 m high. The structure is cantilevered upstream and downstream equally. It is thus well self-balanced. The minimum thickness of vertical walls is not less than 25 cm, principally for durability reasons and construction terms. The PKW has a capacity of 134 m³/s at 283.50 m NGF, which corresponds to an upstream hydraulic head of 1.35 m.

The adjustment of the rating curves for the tested PKW was based on the classical equation (1) for the linear crest spillways, as presented in Leite Ribeiro et al (2007a, b). In this fitting, the PKW is considered as a linear crest spillway with the effective length (L_{eff}) decreasing with the increase of head (H).

$$Q = C_{dL} L_{eff} \sqrt{2g} H^{\frac{3}{2}} \quad (1)$$

The discharge coefficient C_{dL} was assumed for a sharp-crested weir over the total developed length of the PKW. This coefficient is based on the equation of SIA (Carrier, 1972) as follows:

$$C_{dL} = 0.410 \left(1 + \frac{1}{1000H + 1.6} \right) \left[1 + 0.5 \left(\frac{H}{H+P} \right)^2 \right] \quad (2)$$

It is a function of the hydraulic head (H) and the height of the vertical walls of a linear weir (P). For the PKW, P varies along the crest and was defined as a first approximation, by its mean value considered equal to $P/2$, for $H=0$, P being the height of PKW at inlet key entrance. For computation of L_{eff} , the proposed model defines the effective length in function of the total developed length of the PKW (L), the width of the pass (W), the total head upstream of the PKW (H) (Pralong et al., 2011) and two coefficients, k and n depending on the geometric parameters of the structure.

$$L_{eff} = W + \frac{L}{\left(kH + \frac{L}{\sqrt[n]{L-W}}\right)^n} \quad (3)$$

In the present case, the rating curve for the PKW was fitted with $k=0.055$ and $n=5$ for the rectangular crest shape, respectively $k=0.055$ and $n=7$ for the trapezoidal shape. Figure 4 (left) presents the rating curve for both tested PKWs by comparison with the theoretical rating curve of a linear Creager shape with crest length equal to the width of the pass $W=14.50$ m. This result demonstrates the efficiency of the PKW under low heads. The fitted curve agrees with measurements and denotes a coherent convergence towards the classical broad crested spillway of length W for increasing H .

2.3.2 Total overflow capacity

The curves presented on Figure 4 (right) for the 3 gated weirs were obtained experimentally and correspond very well to the computed ones according to classical theory of standard weirs (Hager & Schleiss, 2009). The total overflow capacity of the St-Marc dam with the PKW is $757 \text{ m}^3/\text{s}$ at the maximum operation level. This value is higher than the required capacity.

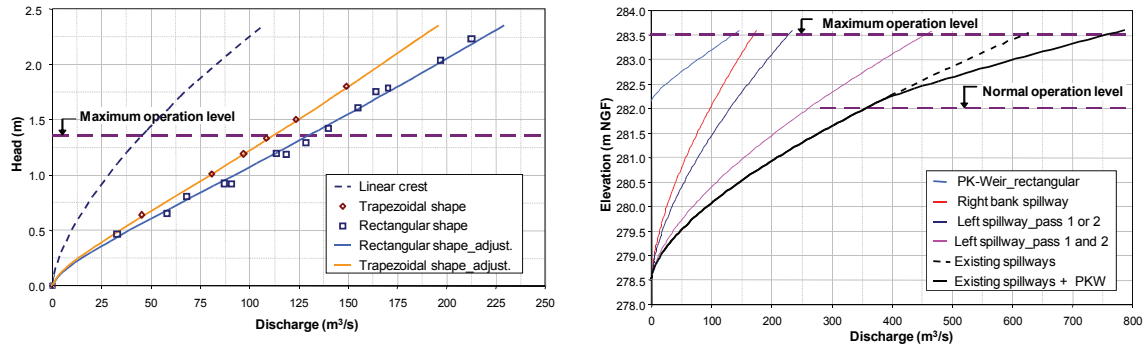


Figure 4. Rating curves of the tested PKW (left) and of the overall spillway devices (right).

2.4 Energy dissipation

The trajectories of the jets issued from both inlet and outlet keys of the PKW impact the downstream slab of the dam before pursuing over the natural topography. In order to dissipate the energy of the flow, six different solutions were designed and tested on the model, including protection of the natural gneiss, baffle blocks on the slab, deflector walls and shifted ski jumps. Figure 5 illustrates the situation without any device and the adopted solution (inclined gutter).

For geological reasons related to the safety of existing structures and to excavated volumes, the best option consisted to divert the flow towards the existing stilling basin. The adopted solution is a leaning 'ski-jump gutter' placed at the contact line between the downstream face of the dam and the natural rock, as shown in Figure 5.

The gutter consists of a cylindrical profile with a constant diameter placed after along an inclined axis with a horizontal reach at the downstream end of the structure. The aim of this solution is to guide the flow from the PKW to the stilling basin of the left spillway.

To improve the energy dissipation and to mitigate the impact length of the flow on the downstream face of the dam, additional measures had to be taken at the outlets keys of the PKW. Some steps were constructed on the inclined slabs in order to contribute to energy dissipation (André et al., 2008a,b) and consequently to shortening the jet's trajectories (Fig. 6).

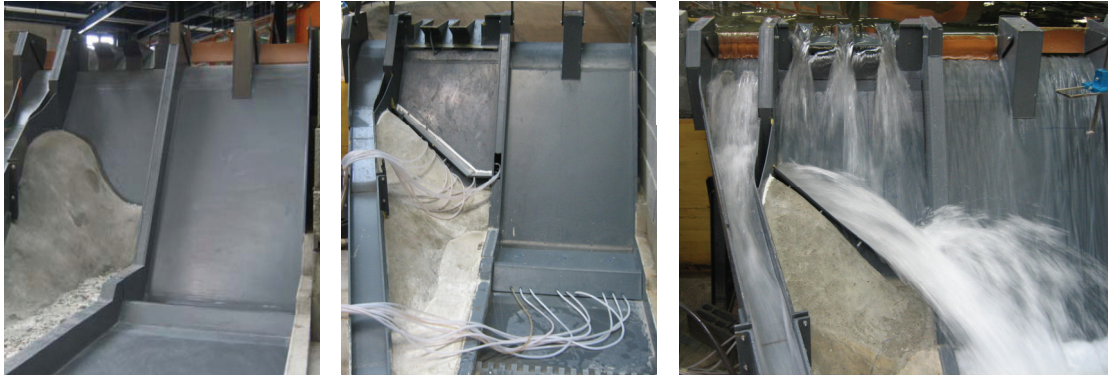


Figure 5. Spillway devices with the natural rock apron downstream from the PKW (left); with the inclined gutter and pressure measuring connections (middle); flow pattern at maximum operation level (right).

These additional measures have been quite efficient in improving the performance of the ski-jump gutter. The design of the steps was done taking care not to reduce the hydraulic capacity of the PKW over the operating range.

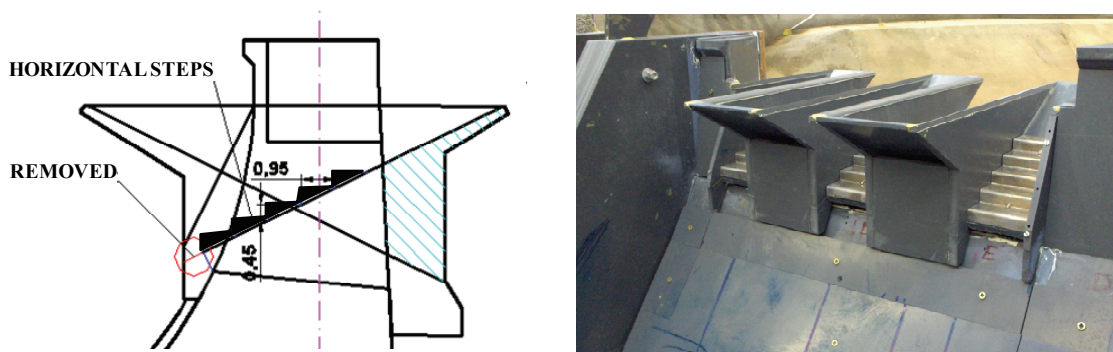


Figure 6. PKW optimization measures: Steps insertion and length shortening of the outlet key. Schematic profile (left) and downstream view of the model (right).

2.5 Pressure measurements

To analyze the behavior of the energy dissipating structure downstream of the PKW, pressure measurements have been made at several points in the inlet of the PKW, on the downstream face of the dam, in the gutter, on the right guide wall of the left spillway and in the stilling basin. An electronic transducer with a precision higher than 0.1 mm and a sampling frequency of 100 Hz was used.

The signal statistical analysis, considered the mean, minimum and maximum values, the standard deviation, the energy spectrum density and the distribution function curve. Following comments can be made, on these measurements:

- Inlet key of the PKW: Vibrations occur on the structure with 0.8 Hz frequencies at prototype scale, if the jet is not aerated. These vibrations disappear with complete aeration of the under nape of the jet.
- Downstream face of the dam: Mean pressure values from 0 to 2 m water column (w.c.) and maximum standard deviation value of 0.5 m w.c. at the point with the maximum mean value.
- Inclined gutter: Mean pressure values from 3.2 to 8.5 m w.c. and a maximum standard deviation of 1.1 m w.c.
- Right guide wall of the left spillway: Mean pressure values lay close to zero. This means that no collision occur between the jet coming from the inclined gutter and the wall. The standard deviation varies from 0.30 to 0.5 m w.c.
- Stilling basin of the left spillway: Pressure measurements have been compared between the existing situation and the project design. Results show that the PKW will not lead to significant variations of the mean values and standard deviations for most points. The highest difference is

observed in the shaded zone of the pier dividing the two existing passes. At this location, pressure values are actually close to 0 m w.c. and will increase to 2.5 m w.c. with the PKW.

3 STRUCTURAL ASPECTS AND CONSTRUCTION

3.1 Concrete expansion pathology

The PKW structural design is strongly impacted by the fact that the dam's concrete is subjected to a noteworthy expansion (20 up to 40 $\mu\text{m}/\text{m}/\text{year}$), mainly due to alkali-aggregate reaction. Moreover, this pathology affects the resistance to hydraulic erosion of the concrete located on the downstream face of the dam, which is exposed to the PKW outflow. Since the characteristic duration of the floods is sufficiently long to enable severe damages (48 hours), a protection of the downstream face of the dam with a concrete slab is required.

Anchoring classically the new structures in the expansive existing concrete would on the one hand generate very high stresses in structures, and on the other hand require using tremendous quantities of connectors. Besides, the question of long term integrity of an anchor's sealing made in an expansive concrete revealed to be quite complicated. Consequently, it was decided not to anchor the new structure on the dam, but only to ensure its stability by means of compressive and friction forces. Stresses generated by dam expansion are then limited to friction capacity between both materials.

3.2 PKW stability

In order to provide a gravity stability, a counterweight beam was added upstream of the inlet keys. In addition, water tightness (by redundant geomembrane and bituminous joint) and drainage systems are designed to prevent any uplift forces development as presented in Figure 7.

The PKW stability analysis was conducted according to the usual bi-dimensional gravity dams' calculation method and safety standards in force for EDF's dams (Table 1). Safety factors were also checked in a deteriorated tightness configuration, with a linear uplift diagram hypothesis. In this table, values in parentheses correspond to residual safety factors calculated without water tightness in case of both geomembrane and bituminous joint defect.

The seismic effects were taken into account as pseudo-static forces, but with a ground acceleration amplification which was evaluated through a temporal analysis applied to the unmodified dam's profile. As the fundamental mode of the dam matches the seismic spectrum's peak, the 0.1 g ground acceleration reaches 0.35 g at PKW foundation level.

Resulting safety factors are all correct. Nevertheless, a lack of horizontal resistance appears during MCE (Maximum Credible Earthquake) in case of severe tightness defect. For this reason, a stop beam was added downstream of the outlets' ends. The latter and the upstream counterweight beam are designed to resist unaided the maximum horizontal resulting force. In order to allow concrete expansion, a 3cm clearance will be kept between upstream counterweight beam and dam. The impact of the PKW on global dam's stability is less than 2%. It's worth noticing too that for durability improvement, walls thickness have been increased to 25 cm/min.

Table 1 . PKW stability factors

Conditions	Upstream effective stress	Downstream effective stress	Uncompressed length	HSF*	OSF*
Normal Level + Ice	49 (0) kPa	166(166)kPa	0 (0.01) m	2.64 (2.04)	4.17 (1.91)
<i>EDF criterium</i>	$0 < \sigma < 5 \text{ MPa}$	$0 < \sigma < 5 \text{ MPa}$	0 m	> 1.33	> 1.40
PMF	0 (0) kPa	231 (335) kPa	0.04 (3.06) m	2.08 (1.17)	3.18 (1.18)
<i>EDF criterium</i>	$0 < \sigma < 7.5 \text{ MPa}$	$0 < \sigma < 7.5 \text{ MPa}$		> 1.10	> 1.30
MCE	0 (0) kPa	215 (248) kPa	0.20 (1.81) m	1.09 (0.83)	2.71 (1.47)
<i>EDF criterium</i>	$0 < \sigma < 15 \text{ MPa}$	$0 < \sigma < 15 \text{ MPa}$		> 1.05	> 1.20

* *HSF = Horizontal Safety Factor; OSF = Overturning Safety Factor - (with water tightness defect)*

3.3 Downstream face protection slab

Water flow on the downstream face of the dam may last for more than 48 hours and reaches velocities above 20 m/s. The nature of this flow is rather different from the ones which occur in the existing spillway (no water impact on the concrete versus an average impact angle of 25° for the PKW and flow direction parallel to the spillway).

This outflow could thus be linked with cavitations, cracks, development of turbulences and uplift pressures, potential sources of erosion of the concrete on the downstream face. In addition, this concrete is more than 75 years old, with small cement content, affected by expansion pathology and behaves a poor surface quality.

It therefore appears difficult to assess what could be the consequences of a long lasting water flow on the existing downstream face concrete. It was consequently decided to construct it with a 50 cm thick concrete slab. This protection layer shall be drained.

The stability of the downstream face protection slab is ensured by its dead load and by clamping to the gutter. A draining geogrid will prevent any uplift forces development, in addition with draining walls. Nevertheless, the slab will be subjected to negative pressures due to the flow. Final design is carried out according to the pressure measurements presented above. A preliminary technico-economical study led to a slab thickness of 0.5 m, which was considered a comfortable dimension with regards to the behavior under vibrations.

3.4 "Ski jump" gutter

As for the PKW and the downstream face protection slab, the gutter shall not be anchored to the dam. In fact it will be simply resting on steps cut at the foot of downstream face of the dam. The interface between gutter and dam will be drained to avoid uplift pressures.

Three types of loads are therefore applied to the gutter: self weight, hydraulic loads coming from the water flow deviation and loads transmitted by the downstream face protection slab which is clamped to the gutter.

The component of the gutter self weight which is parallel to the gutter slope forms a 31° angle with the horizontal. This load is taken by the friction reaction at the interface considering a 30° friction angle between new concrete and existing concrete. The standard safety margin is given by the friction and abutment of the horizontal part of the gutter. Hydraulic loads add "weight" to the gutter because of their direction and the way loads are transmitted to the gutter.

The horizontal component of loads transmitted by the downstream face protection slab will be supported by the abutment to the rock and the moment will be taken by the gutter self weight. There is no existing rock to support the horizontal loads applied to the bottom horizontal part of the gutter. This area will be supported by the upper sloped part of the gutter and will behave as a cantilever beam with regard to horizontal loads. Design calculations were carried out with extreme pressure values measured on the model.

3.5 Aeration

In order to aerate the flow on the PKW, two air intakes (400 mm diameter) were implemented (Fig. 7). Both structures start below outlet keys and come out to the dam crest, both sides of the PKW. They are protected by stainless steel covers and connected to each other in order to allow redundancy of air supply in case of blockage of one pipe.

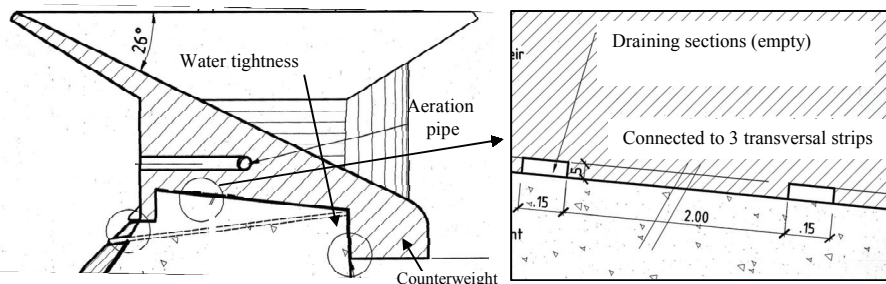


Figure 7. PKW schematic section with details of the aeration and drainage systems

4 CONCLUSIONS

The Saint-Marc hydraulic scheme presented a lack of spillway capacity after updating the 1'000 years return period flood. In order to provide the capacity in deficit, a PKW was designed. The flow guidance and energy dissipation behind the new structure required original and innovative solutions due to delicate geological context and existing works. The experimental study carried out at LCH on a physical model with scale factor 1/30 allowed the validation of the proposed PKW and the design of an energy dissipation structure for the new spillway.

Among the tested shapes of the PKW, the rectangular one has been adopted to the rehabilitation project. Its maximum capacity of 134 m³/s allows increasing the total spillway capacity up to 757 m³/s, slightly higher than the design flood. Experimental tests also demonstrated the efficiency of the PKW under lower heads. LSPIV measurements of the approaching field velocities in the reservoir confirmed that no negative interactions occur for different gate operations, with and without PKW.

For the energy dissipation of the jet issued from the PKW, the adopted solution is a leaned “ski-jump gutter” placed at the contact line between the downstream face of the dam and the natural rock with the objective to guide the flow from the PKW to the existing stilling basin of the left spillway.

Dynamic pressure measurements performed on the physical model gave additional information needed to the final design of the rehabilitation project, particularly concerning static and dynamic solicitation of the structures and also the jet aeration.

The structural design of all hydraulic structures (PKW, downstream protection slab and “ski-jump” gutter) is strongly impacted by the concrete expanding pathology. It was decided not to anchor the new structures to the dam, but only ensure their stability by means of compressive and friction forces.

REFERENCES

- André, S., Boillat, J.-L. & Schleiss, A. 2008a. Ecoulements aérés sur évacuateurs en marches d'escalier équipées de macro-rugosités - Partie I: caractéristiques hydrauliques. *La Houille Blanche – Revue internationale de l'eau* (N° 1): 91-100.
- André, S., Boillat, J.-L. & Schleiss, A. 2008b. Ecoulements aérés sur évacuateurs en marches d'escalier équipées de macro-rugosités - Partie II: dissipation d'énergie. *La Houille Blanche – Revue internationale de l'eau* (N° 1): 101-108.
- Carlier, M. 1972. *Hydraulique Générale et Appliquée*. Ed. Eyrolles, France.
- CFGB, Comité Français des Grands Barrages. 1994. Les crues de projet des barrages : Méthode du gradex, *18^{ème} congrès du Comité International des grands barrages*, Durban, Afrique du Sud.
- Hager, W.H. & Schleiss, A.J. 2009. *Traité de Génie Civil Volume 15 - Constructions hydrauliques, Ecoulements stationnaires*. Lausanne : PPUR - Presses Polytechniques Romandes.
- Kantoush, S. A., et al. (2008), Flow field investigation in a rectangular shallow reservoir using UVP, LSPIV and numerical modelling, *Flow Measurement and Instrumentation*, 19, 139-144.
- Laugier, F., Lochu, A., Gille, C., Leite Ribeiro, M., Boillat, J.-L. 2009. Design and construction of a labyrinth PKW spillway at Saint-Marc dam, France. *Hydropower & Dams*, Issue Five, pp. 100-107.
- Leite Ribeiro, M., Bieri, M., Boillat, J.-L., Schleiss, A., Delorme, F. & Laugier, F. 2009. Hydraulic capacity improvement of existing spillways – design of a piano key weirs. *23rd Congress of the Int. Commission on Large Dams CIGB-ICOLD*; Proc. (Volume III, Q.90-R.43), 25-29 May 2009, Brasilia.
- Leite Ribeiro M., Boillat J.-L., Schleiss A., Laugier F., Albalat C. 2007(a). Rehabilitation of St-Marc Dam - Experimental Optimization of a Piano Key Weir. *Proceedings of the 32nd Congress of IAHR. Venice, Italy*.
- Leite Ribeiro M., Boillat J.-L., Kantoush S., Albalat C., Laugier F., Lochu A. 2007(b). Rehabilitation of St-Marc dam – Model studies for the spillways. *Proceedings of Hydro 2007 "New approaches for a new era"*. Granada, Spain.
- Pralong, J., Blancher, B., Laugier, F., Machiels, O., Erpicum, S., Piroton, M., Leite Ribeiro, M., Boillat, J.-L., and Schleiss, A.J. 2011. Proposal of a naming convention for the Piano Key Weir geometrical parameters. *International Workshop on Labyrinth and Piano Key weirs*, Liège, Belgium.
- USACE, US ARMY Corps of Engineers (2002). HEC-RAS River Analysis System, User's and Reference Manuals; Version 3.1, *Hydrologic Engineering Center*, Davis, CA, November 2002.