Energy dissipation downstream of Piano Key Weirs – Case study of Gloriettes Dam (France)

M. Bieri, M. Federspiel & J.-L. Boillat
Ecole Polytechnique Fédérale de Lausanne (EPFL), Laboratory of Hydraulic Constructions (LCH), Lausanne, Switzerland

B. Houdant, L. Faramond & F. Delorme
Electricité de France (EDF), Centre d'Ingénierie Hydraulique (CIH), Le Bourget-du-Lac, France

ABSTRACT: The Gloriettes concrete arch dam in the French Pyrenees showed a deficit of 80 m³/s for the new design flood of 150 m³/s. To address the problem, a Piano Key Weir (PKW) as an additional spillway was designed. The existence of a geotechnically unstable zone downstream of the PKW site, as well as requirements for environmental integration, did not allow a direct trajectory of the tailrace channel, but imposed an abrupt change in direction halfway. A stepped channel including an intermediate stilling basin was conceived for this purpose. The energy dissipation over the total length of the restitution channel for this condition was found to be about 90 per cent. Since the construction works could only be carried out in summer, the works were split into two stages: summer 2009 (excavation of the stepped reaches and the stilling basin) and summer 2010 (construction of the PKW and completion).

1 INTRODUCTION

The Gloriettes dam, a concrete arch structure on the Gave d'Estaubé River, operated by Electricité de France (EDF) is located in the French Pyrenees (Fig. 1). It was constructed between 1949 and 1951. The initial spillway consists of four free-overflow sluices on the dam crest at 1667 m NGF. Its capacity is about 70 m³/s at the maximum operating level of 1667.8 m NGF. For the new design flood of a 1000 year return period, a peak discharge of 150 m³/s was defined. In order to compensate for the deficit of 80 m³/s, a complementary spillway on the right bank had to be constructed. Two different shapes of Piano Key Weir (PKW) with weir crests at the same level as the existing spillway have been designed and evaluated by physical model tests. The results have been published in Bieri et al. (2010).

The existence of a geotechnically unstable zone downstream of the new PKW site, as well as requirements for environmental integration, did not allow for a simple and direct trajectory of the tailrace channel, but instead there must be an abrupt change in direction. The main design purpose was to produce continuous and maximal energy dissipation, by avoiding overflows.

The first design of the tailrace channel involved two straight reaches, interconnected by a 120º curve. The preliminary evaluation was based on an analysis of the water level, flow velocity and energy dissipation. The energy grade line was computed by applying the Bernoulli equation on steep slopes and the energy dissipation was based on the classical Colebrook-White equation. The original design allowed for only a low level of energy dissipation, leading to high flow velocities of about 25 m/s. The cavitation index σ at the bottom was computed, showing that the threshold value for cavitation risk of 0.2 was exceeded in several locations.

On the other hand, it was shown by using the empirical Knapp formula (Hager & Schleiss 2009), that the initial incurved part of the channel with its small radius could generate water levels of up to 6 m as a result of the centrifugal acceleration and stationary waves. This hydraulic behaviour led to a completely new design for the restitution channel with two stepped reaches and an intermediate stilling basin, allowing the 120º change of direction halfway.
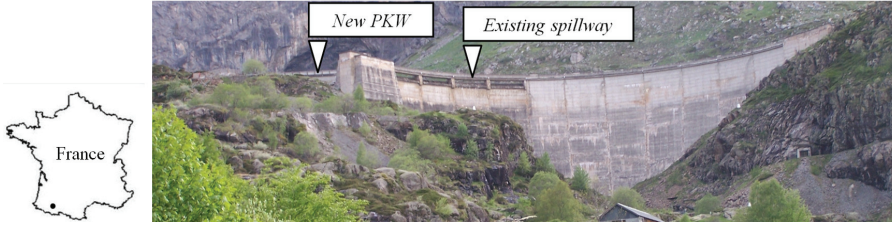
Figure 1. Downstream view of Gloriettes dam with existing spillway and construction site of the PKW.

2 DESIGN CRITERIA

2.1 *Boundary conditions*

The upstream boundary conditions of the restitution channel are imposed by the PKW. Several tests were carried out to evaluate the hydraulic capacity of two types of PKW in relation to the existing spillway on the dam crest (Fig. 2). The results relating to the hydraulic aspects of the weir system are discussed in Leite et al. (2009). These tests led to a maximum discharge of 80 m³/s in the restitution channel for the design flood of 150 m³/s.
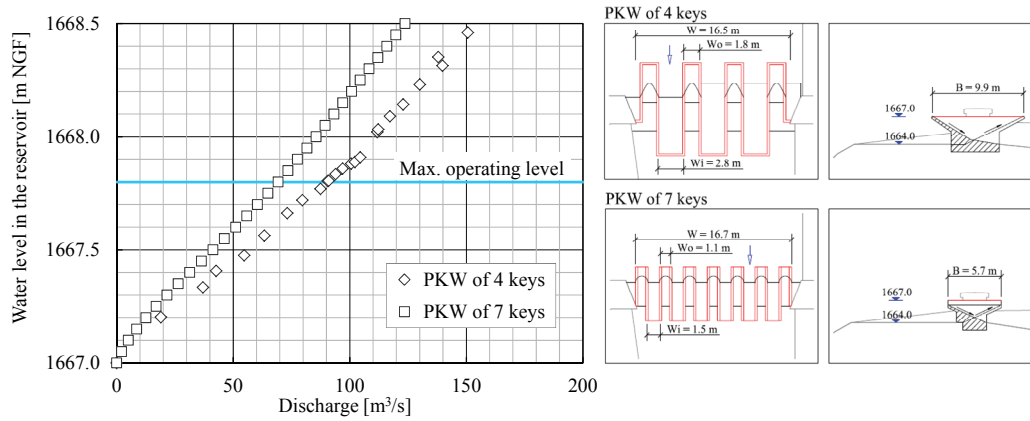

Figure 2. Capacity and geometrical characteristics of the two tested PKW.

2.2 *Stepped channel reaches*

To achieve maximum energy dissipation on the two channel reaches, steps were provided along the profile. The preliminary design was based on a research project on stepped spillways (André et al. 2008a, b). Using Froude similarity, scale effects related to air transport and velocity profiles are negligible for scale factors λ between 5 and 15 (Boes 2000). In models exceeding this factor, air bubbles are proportionally too large and air transport capacity tends to be lower than for an equivalent prototype. Furthermore, scale effects are small for Reynolds numbers over 10^5 and Weber numbers higher than 100 (Boes & Hager 2003), which was the case for the conducted tests. For quasi-uniform flow, low discharges generate nappe flow and high discharges skimming flow conditions. Nappe flow dissipates energy by flow impact on the steps. Skimming flow is less efficient, due to the development of entrapped flow cells between the steps. When both flow regimes are partly present, the regime is called transitional flow. The onset can be defined by Equation 1 for transitional flow and by Equation 2 for skimming flow, where h_c (Equation 3) is the critical water depth, h_s the step height, l_s the step length, q_w the unit flow discharge and g the gravity acceleration (Fig. 3a):

$$(h_c / h_s) = 0.743 \cdot (h_s / l_s)^{-0.244} \tag{1}$$

$$(h_c / h_s) = 0.939 \cdot (h_s / l_s)^{-0.367} \tag{2}$$

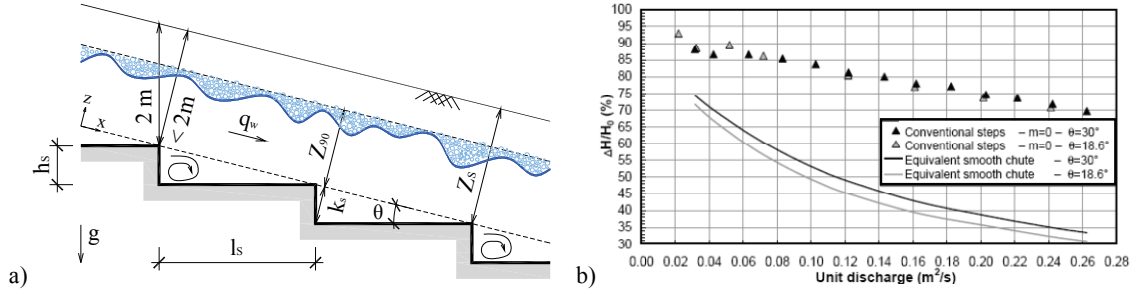$$h_c = (q_w^{\,2} / g)^{1/3} \tag{3}$$

Figure 3. Longitudinal profile of the channel with skimming flow conditions (on the left) and relative energy loss $\Delta H/H_0$ (on the right) for conventional stepped (without macro-roughness elements $m = 0$) and smooth chutes for two chute slopes θ (André 2004).

The dissipated energy ΔH for different height to length of step ratios is generally between 85 and 95 per cent of the available head H_0. The head loss ΔH (Fig. 3b) is estimated based on an experimentally developed relationship between unit discharge and relative energy loss $\Delta H/H_0$. These results were obtained for 0.06 m high steps in the test flume. For 1 m steps, the scale factor λ is 16.67, and for 2 m, it becomes 33.33. The corresponding discharge scale factors ($\lambda^{5/2}$) are 1.13 and 6.42, respectively. For a discharge of 80 m³/s and a channel width of 7.5 m, the skimming flow regime establishes and the energy dissipation is about 90 per cent for both step heights.

In both cases, the main design criteria was avoiding overflow and minimizing the excavation volume, which led to variable step lengths. By knowing the average chute slopes θ of the two channel reaches, 19° and 18° respectively, the normal height of the steps k_s (Equation 4) and Froude number for steep slope F_θ (Equation 5) could be defined, and consequently the depth of uniform flow mixture $Z_{90,u}$ (Equation 6). This flow depth corresponds to the position where the air concentration is 90 per cent:

$$k_s = h_s \cdot \cos\theta \tag{4}$$

$$F_\theta = q_w / \sqrt{g \cdot k_s^3 \cdot \cos\theta} \tag{5}$$

$$Z_{90,u} / k_s = 0.58 \cdot F_\theta^{0.6\cos\theta} \tag{6}$$

For $h_s = 1$ m and 2 m, $Z_{90,u}$ is about 1.2 m, and 1.3 m respectively. By applying a safety factor of 1.5, the water depth for the maximum discharge of 80 m³/s is 2 m. This depth is required to define the offset Z_s of the channel ground from the surrounding topography. It leads to the lower limit for the channel bottom, which should not be overtopped by the steps. Because of the very long steps, requiring a significant excavation volume, the 2 m high steps were refused.

2.3 Stilling basin

The intermediate stilling basin had to be able to dissipate the residual energy of the flow at the downstream end of the upper reach of the restitution channel. It had to be integrated in the existing topography so as to provide critical flow conditions at the beginning of the second channel reach. To ensure satisfactory performance of the sudden expansion of the stilling basin, a central sill has proved to be effective (Bremen 1990). The optimal design of the sill leads to symmetrical and stable jumps with significant contribution of the lateral eddies to the energy dissipation process, and almost uniform tailwater velocity distribution (Fig. 4a).

For a given discharge, the inflow depth h_1, the flow velocity v_1 and the Froude number F_1 at the outflow of the first reach of the channel can be estimated by applying Equation 6. By using the Bélanger formula, the conjugated downstream flow depth h_2 is calculated. For the design flood of 80 m³/s, $h_1 = 1.1$ m, $v_1 = 9.7$ m/s, $Fr_1 = 3.0$ and as a result $h_2 = 4.1$ m.
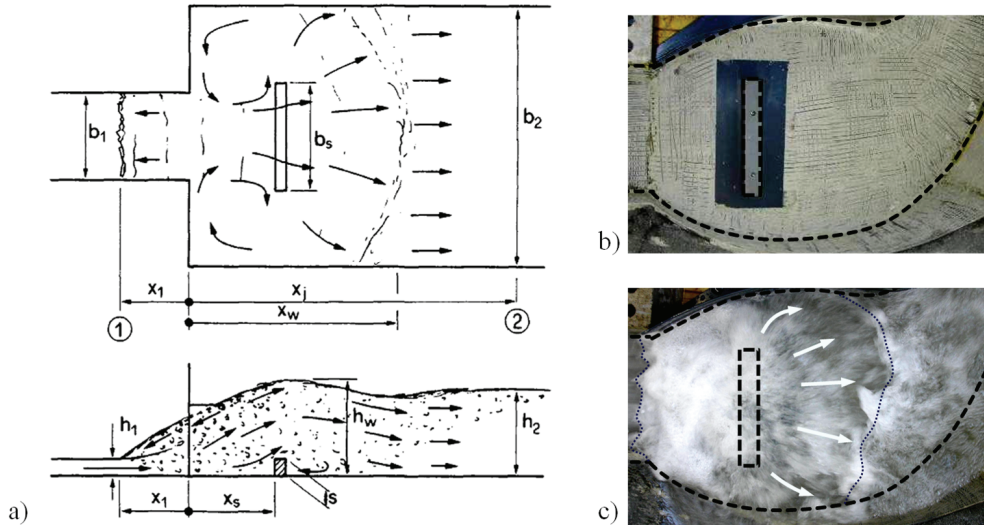
Figure 4. Notation for sill-controlled jumps in sudden expansion (Bremen 1990) (a) and final design of the modelled basin, empty (b) and with overall flow pattern (c).

According to Bremen (1990), the length of the basin x_j is defined by Equation 7. The calculated value of x_j = 16 m was increased here by 5 m, to guide the water in the curved part of the basin towards the outlet:

$$x_j = h_1 \cdot (6.29 \cdot Fr_1 - 3.59) \cong 4.5 \cdot h_2 \qquad (7)$$

The optimal sill geometry mainly depends on Fr_1, the expansion ratio between the basin width b_2 and the channel width b_1 as well as h_1 and b_1. In the present case b_2 = 15 m was chosen twice as b_1 = 7.5 m. The best flow conditions are obtained by a non-dimensional sill position X_p which is higher than 0.8 (Bremen 1990). Considering the progressive enlargement of the inflow part X_p was fixed at 1.75. The optimal position of the sill related to the channel outlet x_s = 6.5 m can be defined by Equation 8:

$$X_p = \frac{x_s}{(b_2 - b_1)/2} \qquad (8)$$

The sill height s = 1.6 m is obtained by Equation 9:

$$s = \frac{x_s / x_j + 0.0116 \cdot Fr_1 - 0.225}{0.155 - 0.008 \cdot Fr_1} \qquad (9)$$

The sill width b_s = 9.4 m can be calculated with Equation 10:

$$b_s = b_1 \cdot (1 + 0.25 \cdot (b_2 / b_1 - 1)) \qquad (10)$$

For reasons of environmental integration, the smoothly rounded side walls of the basin (Fig. 4b) are the only difference between the model configuration and the theoretical case. Flow patterns remained therefore quite similar (Fig. 4c).

3 PHYSICAL MODEL TESTS

3.1 *Experimental setup and device*

For the physical modelling a geometrical scale factor λ of 30 and Froude similarity were applied. The scale factor is out of range for modelling two-phase flow. The possibly lower air transport capacity would reduce the air transport in the model and increase the flow resistance. Therefore energy dissipation would be overestimated. However, quasi-uniform flow is only achieved on the last 15 m of the reaches (Boes & Hager 2003) and the effect of air entrainment can therefore be considered as not relevant.

The experimental setup consisted of two connected parts. The first, reproducing the reservoir, the arch dam and the spillways, was placed in a square steel tank. The second was dedicated to the restitution channel, which was reproduced within surrounding walls, allowing extensions in all three dimensions. The channel reaches and the intermediate stilling basin were made of PVC. The water level in the tank was controlled by two ultrasonic sensors and the discharge by an electromagnetic flowmeter. Flow velocities were measured by a micropropeller with 1 mm/s accuracy. The static and dynamic pressures on the sill in the dissipation basin were measured by piezometric tubes and a piezoresistive sensor at 100 Hz sampling frequency.

3.2 *Test programme and optimisation procedure*

For the performance and optimization tests an iterative and systematic procedure was applied. All the experiments were carried out originally with the design flood of 80 m^3/s. The optimized configuration was verified afterwards, with lower discharges of 20, 40 and 60 m^3/s.

In a first step, only the first reach of the restitution channel was modelled, tested and improved. To check the transversal flow distribution, the water levels on both lateral walls were measured manually. The flow behaviour in the incurved part immediately downstream of the PKW required particular attention, two complementary measures for flow equilibration and overflow obviation were developed and tested. On the one hand, the steps were laterally inclined by reducing the interior height and increasing the external one. In addition, the longitudinal step configuration was adapted. On the other hand, a guide wall at the outer bank was provided, to reduce the risk of overflow and erosion of the nearby foundation of the arch dam. Not only was the height of this structure, but also its shape part of the optimization procedure (Fig. 5a).

The main function of the stilling basin downstream of the first reach of the channel is to allow for a 120º change of direction under subcritical flow conditions. The easily adaptable experimental installation facilitated the optimization of several elements of the basin. The bottom level, the surrounding form, the in- and outflow connections of the basin, the height and form of the sill, as well as the adjoining natural topography were systematically adapted and qualitatively evaluated (Fig. 5c). The static and dynamic pressures on the sill under flood conditions were measured and the energetic spectral density analysis aimed to define the structurally problematic frequencies.

a)   Transversally inclined steps downstream of the PKW
b)   First reach of the restitution channel
c)   Dissipation basin in the modeled topography on the left
d)   Second reach of the restitution channel with roughness layer
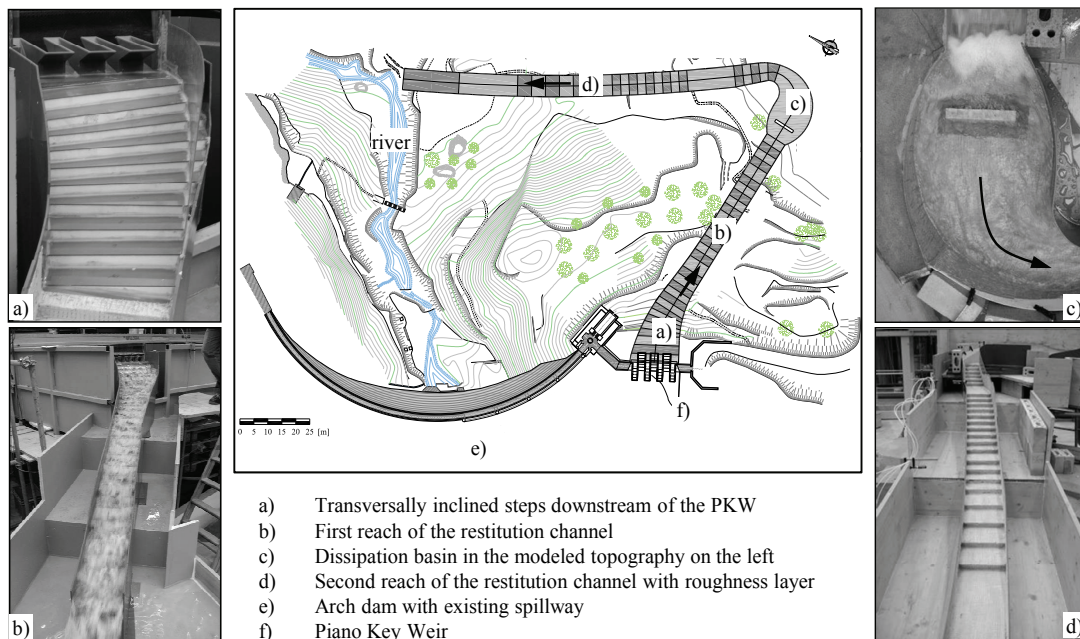e)   Arch dam with existing spillway
f)   Piano Key Weir

Figure 5. Plan view of the final design of Piano Key Weir and the restitution channel.

To simulate the roughness of uncoated excavated rock, with estimated irregularities of 5 to 10 cm on the prototype, a rough cement layer was added at the corresponding surfaces of the channel. Strips of approximately 2 mm high and 10 mm wide were applied (Fig. 5d).

For the final configuration the water levels were measured again, with and without roughness layer. The outflow velocities at the end of the two channel reaches were measured by micropropeller in a 10 mm regular grid. The integrated average velocity of the subcritical flow made it possible to estimate the kinetic as well as the total head, and finally the corresponding energy dissipation.

3.3 *Results*

The optimization of the first reach of the channel led, concerning the upper part, to a particular design of transversally inclined concrete steps and a side wall to avoid overflow. The second reach of the restitution channel is smoothly curved and has longer steps than the first one. At the outlet of the basin, lateral guide walls allow for uniform flow distribution. At the outlet of the second reach, the flow is guided securely towards the axis of the natural river.

Reservoir (Piano Key Weir)

Discharge [m³/s]	20*	20+	40*	40+	60*	60+	80*	80+
Upstream head level	1667.3		1667.4		1667.6		1667.8	

Downstream first reach

Discharge [m³/s]	20*	20+	40*	40+	60*	60+	80*	80+
Water level [m]	0.5	0.8	1.0	1.1	1.3	1.5	1.3	1.6
Velocity [m/s]	4.9	3.3	5.5	4.9	6.0	5.5	8.2	6.6
Kinetic head [m]	1.2	0.6	1.5	1.2	1.9	1.5	3.4	2.2
Total head [el. m]	1637	1636	1638	1637	1638	1638	1640	1639

Stilling basin with hydraulic jump

1635.0

Downstream second reach

Discharge [m³/s]	20*	20+	40*	40+	60*	60+	80*	80+
Water level [m]	0.4	0.6	0.7	0.9	0.9	1.1	1.0	1.3
Velocity [m/s]	6.6	4.4	7.7	6.0	8.8	7.1	10.4	8.2
Kinetic head [m]	2.2	1.0	3.0	1.9	3.9	2.6	5.5	3.4
Total head [el. m]	1612	1611	1613	1612	1614	1613	1616	1614

1609.5

Efficiency of energy dissipation in the two channel reaches

Discharge [m³/s]	20*	20+	40*	40+	60*	60+	80*	80+
$\Delta H/H_0$ (measured)	95%	97%	94%	95%	92%	94%	89%	92%
$\Delta H/H_0$ (estimated)	93%		88%		85%		83%	

Water level (80 m3/s) — Right river bank — Ground level
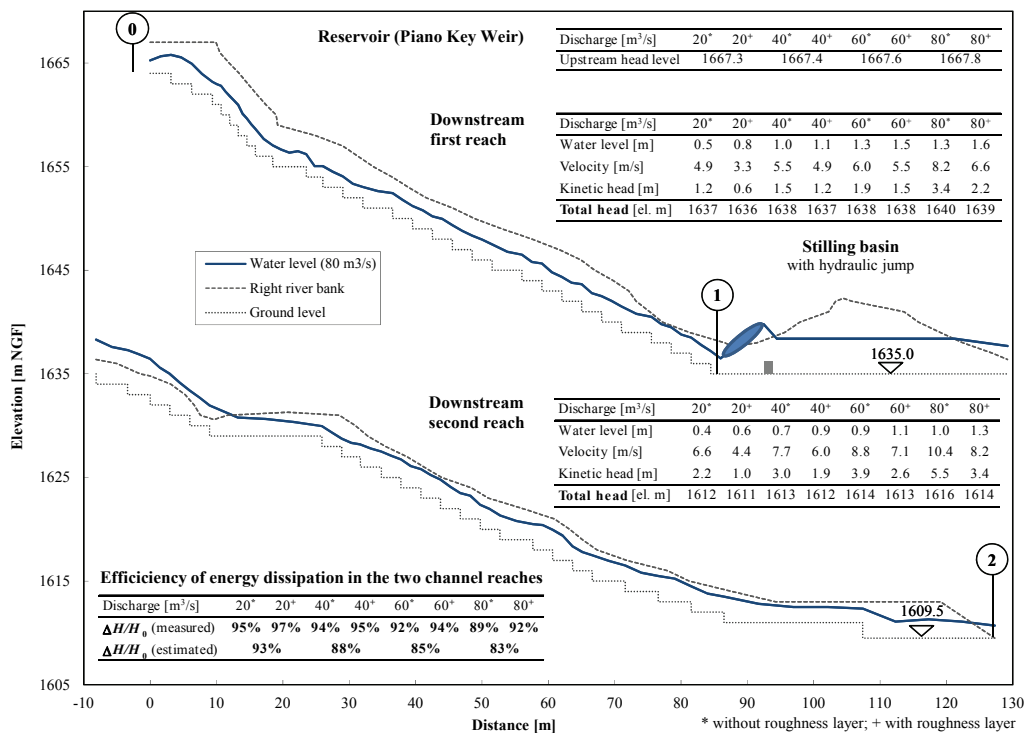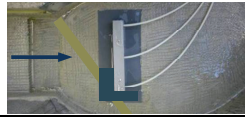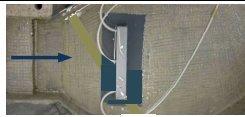
* without roughness layer; + with roughness layer

Figure 6. Velocity and water level measurements with efficiency of energy dissipation in the two channel reaches, without and with roughness layer for four different discharges.

The velocities measured at the outlet of the two channel reaches make it possible to estimate the energy dissipation, expressed as the ratio between head loss and total head (Fig. 6). As expected the efficiency decreases with increasing discharge. The preliminary computation shows good agreement with the experimental results for the estimation of head losses for low discharges, related to nappe flow conditions. For skimming flow conditions, the discrepancy increases with the discharge. The irregular step disposition as well as the channel curvature may be causes of these differences. For the rough layer, the measured velocities at the channel outlets are lower, and as a result the head losses and efficiency are higher. To take into account uncertainties about air entrainment, a safety factor of 1.3 was applied. The optimized design of the stilling basin allows for the required 120º change in direction. It consists of an excavated round pool, enforced by lateral walls on the left and downstream parts. To stabilize the water depth in the basin between 4 and 5 m for the design flood, the outlet must be 6.2 m wide and made of concrete. The maximum jump height is about 9 m. The sill is located 5 m downstream of the upper chan-

nel outlet, and the preliminary defined height $s = 1.6$ m was confirmed by the tests. The round shape of the basin avoids lateral recirculation zones, but reduces the flow section. For that reason, the sill width b_s has been shortened to 7.5 m, corresponding to the channel width. The measured pressures on the front part of the sill are about 5.3 m water column (Tab. 1) and slightly asymmetrical, which can be explained by the asymmetrical shape of the basin. Standard deviations between 1.3 and 1.9 m indicate highly varying flow with a risk of negative values. The measures also show that the back of the sill is about 50 per cent less loaded.

Table 1. Static and dynamic pressures on front and back parts of the sill for the design flood of 80 m³/s.

Water column [m]		Static pressure	Dynamic pressure				Test configuration
		μ	μ	max	min	σ	
Front part	Left	5.1	5.2	13.7	-2.3	1.9	
	Center	5.1	5.4	13.2	0.5	1.3	
	Right	5.4	5.4	12.4	-0.6	1.6	
Back part	Left	2.1	2.2	8.1	-2.1	0.8	
	Center	2.3	2.4	5.2	-0.7	0.8	
	Right	2.1	2.3	6.4	-1.2	0.8	

4 STRUCTURAL ASPECTS AND REALISATION

The Gloriettes dam is located at high altitude and in a wild part of the French Pyrenees. The dam site is inaccessible throughout the whole winter because the road is covered by snow and crosses several avalanche areas. Even in summer, the dam site is a long way from all utilities. It takes 1.5 h to drive to the next concrete factory, for example. Since the construction could only be carried out in summer (from the beginning of May until the end of October), it was decided to split the project into two stages:
− **Stage 1**, during summer 2009, consisted of rock blasting and excavation for the second reach of the channel (Fig. 7a), the stilling basin (Fig. 7d) and the lower part of the first reach, as well as constructing the side walls of the stilling basin and the channel reaches.
− **Stage 2**, during summer 2010, consisted of rock blasting and digging the upper part of the first channel reach with the side walls (Fig. 7c), sawing the concrete wall and constructing the PKW (Fig. 7b), which was realized in less than four months.
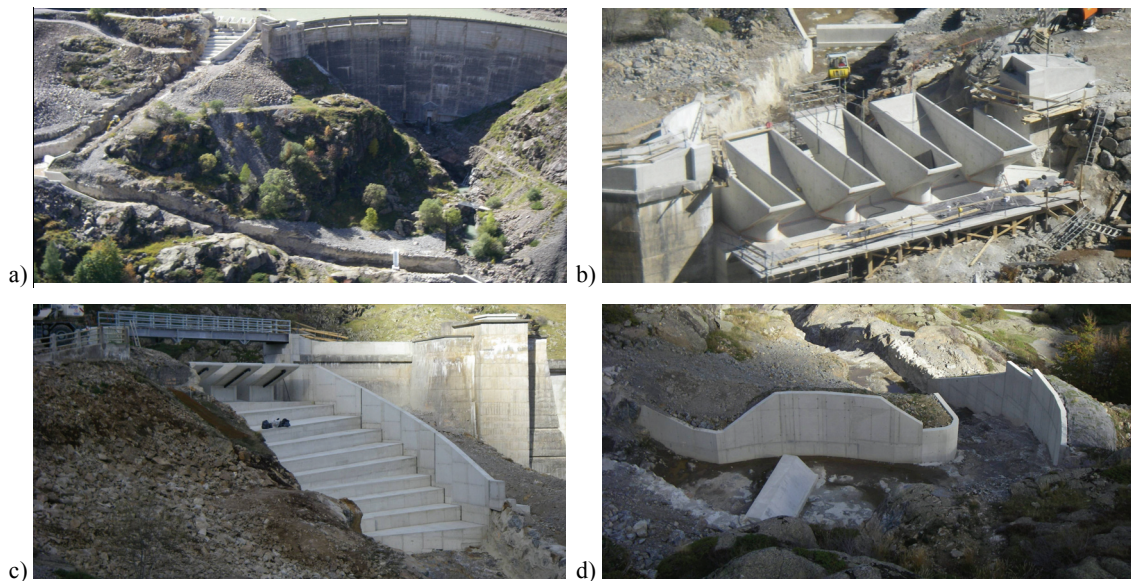
Figure 7. Gloriettes dam area: overall view of the final project (a), upstream view of PKW (b), inclined steps downstream of the PKW (c) and stilling basin with lateral walls and sill (d).

Since the arch dam is located close to the excavation area, rock blasting was strictly monitored

by accelerometers located on the dam and around its foundation. Two blasts produced vibrations with a maximal speed higher than 10 mm/s, but lower than the limit value of 15 mm/s.

From a geological point of view, the right bank of the river, where the new spillway and the channel reaches were built, is formed of gneiss with strong foliation oriented perpendicular to the valley axis and with a high dip. The rock is fractured, ground by the former glacier, but not very degraded. Some geological difficulties were encountered particularly during the construction of the stilling basin: clay faults had to be treated with riprap filled with concrete. The main quantities of material for the construction works are given in Table 2. The total project cost was about €1.5 million. The main part of the cost was related to the energy dissipation devices.

Table 2. Main quantities of material for the construction works.

Excavations	Explosives	Riprap	Concrete	Shotcrete	Reinforcement	Formwork
9000 m^3	2100 kg	800 m^3	1150 m^3	60 m^3	33,000 kg	1300 m^2

5 CONCLUSIONS

Piano Key Weirs are compact and adaptive elements for increasing the flood discharge capacity of existing dams. The evacuation of water downstream of these structures requires original and innovative solutions. The Gloriettes dam spillway upgrade project has involved particularly complex boundary conditions, including a change of direction by 120º of the restitution channel halfway. The channel configuration, with two stepped reaches and an intermediate stilling basin, made it possible to guide the water to the downstream river. Uniform flow distribution on the spillway chute, as well as energy dissipation up to 90 per cent could finally be achieved, minimizing the excavation volume and also meeting integration requirements into the mountainous environment.

The chosen approach, which involved a preliminary design based on a theoretical relations and simple numerical computations followed by systematic tests on a physical model, provided satisfying results in terms of both the technical and economic aspects. Meteorological, geological and environmental issues made the Gloriettes dam rehabilitation project especially ambitious. Nevertheless, the construction works could be completed in the planned time schedule.

REFERENCES

André, S. 2004. High velocity aerated flow on stepped chutes with macro-roughness elements. *Thèse No 2993*, Ecole Polytechnique Fédérale de Lausanne.

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.

Bieri, M., Federspiel, M., Boillat, J.-L., Houdant, B. & Delorme, F. 2010. Spillway discharge capacity upgrade at Gloriettes dam. *International Journal on Hydropower & Dams* 17(5): 88–93.

Boes, R. 2000. Scale effects in modelling two-phase stepped spillway flow. In E. Minor & W. Hager (eds), *Hydraulics of Stepped Spillways* : 53–60. Rotterdam: Balkema.

Boes, R. & Hager, W. 2003. Two-phase flow characteristics of stepped spillways. *J. of Hydraulic Engineering* 129(9): 661–670.

Bremen, R. 1990. Expanding Stilling Basin, *Thèse No 850*, Ecole Polytechnique Fédérale de Lausanne.

Hager, W.H. & Schleiss, A.J. 2009. *Traité de Génie Civil Volume 15 - Constructions hydrauliques, Ecoulements stationnaires*. Lausanne : PPUR - Presses Polytechniques Romandes.

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 weir. *23rd Congress of the Int. Commission on Large Dams; Proc. (Volume III, Q.90-R.43), 25–29 May 2009, Brasilia.*