Spillway discharge capacity upgrade at Gloriettes dam

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In many countries dams are found to have insufficient flood discharge capacity with respect to updated design floods. Upgradig of spillway discharge capacity has therefore become a significant issue for operators of hydropower plants. The Gloriettes concrete arch dam in the French Pyrenees, operated by EDF, shows a deficit of 80 m^3/s for the new design flood of 150 m^3/s. Therefore a complementary spillway on the right bank is to be implemented. The type of labyrinth weir known as the Piano Key Weir (PKW) was selected.

For the Gloriettes dams upgrading scheme, two different shapes of PKW as well as the tailrace channel were tested and optimized by physical modelling (scale 1:30) at the Laboratory of Hydraulic Constructions (LCH) at EPFL in Switzerland, taking into account the geometrical and hydraulic boundary conditions.

As hydrological data records increase and new methods are developed for flood discharge estimations, and also there are higher requirements from the society regarding safety issues, a large number of existing dams are requiring spillway rehabilitation, to improve their flood discharge capacity. As in many other countries, several dams in France have been found to have insufficient discharge capaicty, with respect to updated design floods. Spillway capacity upgrades have therefore become a significant issue for operators of hydropower schemes.

1. Gloriettes dam

The Gloriettes dam, a concrete arch structure on the Gave d'Estaubé river, operated by Electricité de France (EDF), is in the French Pyrenees. It was built between 1949 and 1951. The initial flood discharge system consists of four free-overflow sluices on the dam crest at el. 1667. Its capacity is about 70 m^3/s at the maximum operating level at el. 1667.8. For the new design flood with a 1000 year return period, a peak discharge of 150 m^3/s was defined. To compensate for the deficit of 80 m^3/s, a complementary spillway on the right bank is to be constructed. The type of labyrinth weir, known as the Piano Key Weir (PKW) has been selected. Two different shapes with weir crests at the same level as the existing spillway have been designed and evaluated by physical model tests.

The existence of a geotechnically unstable zone downstream of the new PKW site, as well as the requirements for environmental integration, do 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 is to produce continuous and maximal energy dissipation, by avoiding overflows.

Fig. 1. Downstream views of Gloriettes dam showing the existing spillway on the left, and the right dam abutment and construction site of the new spillway on the right.

The initial design of the tailrace channel involved two straight reaches with a sloped bottom, interconnected by a 120º curve. The preliminary evaluation was based on an analysis of the water level, flow velocity and energy dissipation. The energy head was computed by applying the Bernoulli equation on steep slopes and the energy dissipation was based on the classical Colebrook-White approach. The original design allowed for only a low level of energy dissipation, leading to high flow velocities (~ 25 m/s). The cavitation index (σ) at the bottom was calculated using the standard cavitation equation. The threshold value $\sigma < 0.2$ (cavitation risk) was exceeded in several reaches.

On the other hand, it was shown by using the empirical Knapp formula [Hager *et al*., 2009 *et. al*, 2009[7]], that the curved part of the channel with its small radius generates an internal/external water level 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 (Fig. 3).

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. The results relating to the hydraulic aspects of the weir system are discussed in Leite *et al*. [2009[1]] and Bieri *et al*. [2009[2]]. These tests led to a maximum discharge of 80 m^3/s in the restitution channel for the design flood of 150 m^3/s.

The hydraulic conditions in the river at the confluence were computed by a numerical 1D-Simulation (HEC-RAS, version 3.1.3). The downstream reach of the restitution channel as well as a part of the natural river Gave d'Estaubé, which discharges the overflow from the existing spillway, were modelled. The simple analysis confirmed the subcritical flow in the channel and river. The existing morphology of the river creates a hydraulic jump on the penultimate step of the channel, and also lateral overflow in the confluence area. Several morphological improvements of the river and the downstream reach of the channel, including local widening and dredging, have been proposed and added to the numerical model. However, as a result of the preliminary numerical simulations, the locally limited influence of the confluence on the outflow in the restitution channel was not taken into consideration for the physical modelling.

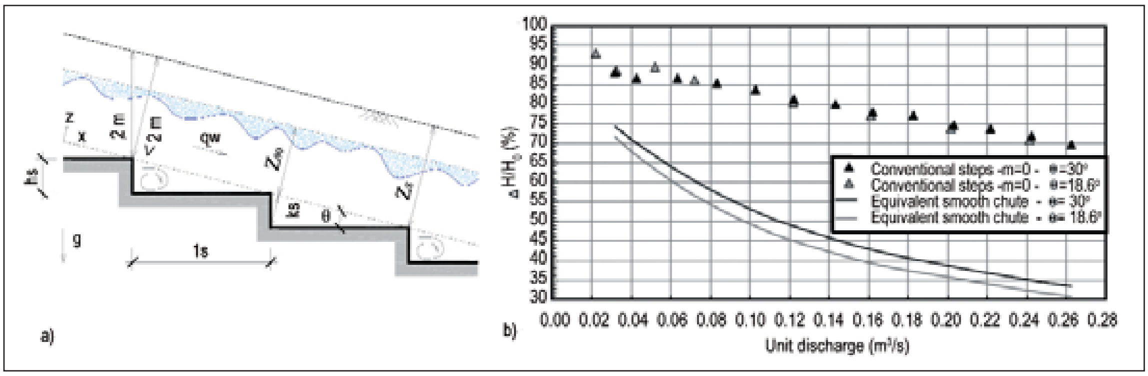


Fig. 2. Longitudinal profile of the channel with skimming flow (a) and relative energy loss ΔH/H0 (b) for conventional stepped (without macro-roughness elements m=0) and smooth chutes for two different chute slopes θ [André 2004[5]].

2.2 Stepped channel reaches

To obtain maximum energy dissipation in the two channel reaches, steps were provided along the profile. The preliminary design was based on a recent research project on stepped spillways [André *et al*., 2008[3,4]). Low discharges generate nappe flow, and high discharges create skimming flow conditions. Nappe flow dissipates energy by flow impact on the steps. Skimming flow is less efficient, because of the entrapped flow cells between the steps. When both flow regimes are partly present, the regime is called transition flow. The onset of the flow regimes is defined by (1) for transition flow and (2) for skimming flow, where h_c (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. 2a):

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

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

$$h_c = (q_w^{\;2}/g)^{1/3} \qquad \text{... (3)}$$

The dissipated energy, ΔH, for different ratios of step height and length is generally between 85 and 95 per cent of the available head H_0. The head loss ΔH (Fig. 2b) 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 is 33.33. The corresponding discharge scale factors ($\lambda^{5/2}$) are 1.134 and 6.415, respectively. For a discharge of 80 m³/s and a channel width of 7.5 m, the regime is skimming flow and energy dissipation about 90 per cent for both step heights.

Two step heights, 1 m and 2 m, with variable step length were analysed. In both cases, the main design criteria for the steps composition was avoiding overflow and minimizing the excavation volume, which led to variable step lengths. By knowing the chute slopes θ of the two channel reaches, normal heights of the steps k_s (4) and Froude numbers for steep slope F_θ (5) could be defined and led to the depth of uniform flow mixture $Z_{90,u}$ (6). This flow depth corresponds to an air concentration of 90 per cent:

$$k_s = h_s \cdot \cos\theta \qquad \text{... (4)}$$

$$F_\theta = q_w / \sqrt{g \cdot k_s^{\;3} \cdot \cos\theta} \qquad \text{... (5)}$$

$$Z_{90,u} / k_s = 0.58 \cdot F_\theta^{\;0.6\cos\theta} \qquad \text{... (6)}$$

For 1 m and 2 m high steps, $Z_{90,u}$ 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 base 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 siginficant excavation volume, the 2 m high steps were no acceptable.

2.3 Stilling basin

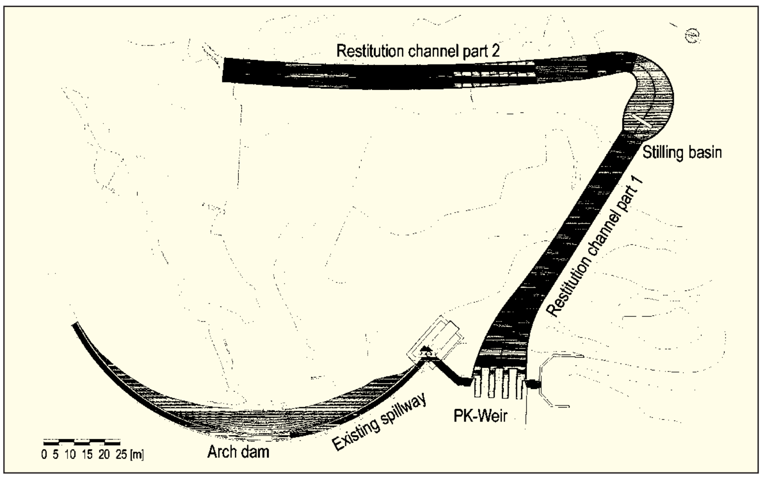
The stilling basin has to be able to dissipate the residual energy of the flow at the downstream end of the upper reach of the restitution channel. It has to be integrated in the existing topography so as to provide critical flow conditions at the entrance of the second channel reach. To ensure satisfactory performance of the sudden expansion of the stilling basin, a central sill has proved to be effective [Bremen, 1990[6]]. 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 (see 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 eq. (6). By using the Bélanger formula, the conjugated downstream flow depth h_2 is then calculated. For the design flood of 80 m³/s, h_1 is 1.1 m, v_1 9.7 m/s, Fr_1 3.0 and as a result h_2 4.1 m.

According to Bremen [1990[6]], the length of the basin x_j is defined by eq. (7). The calculated value of x_j of about 16 m was increased here by 5 m, to guide the water in the curved part of the basin towards the outlet:

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

Fig. 3. Plan view of the final design of the restitution channel.



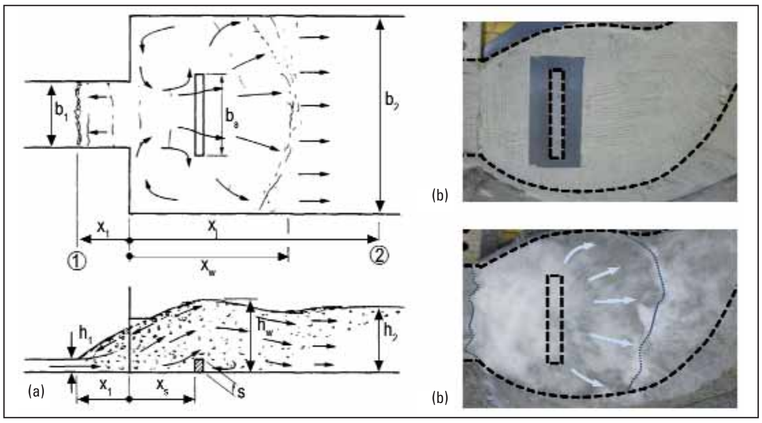


Fig. 4. Notation for sill-controlled jumps in sudden expansion [Bremen 1990[7]] (a); final design of the modelled basin, empty (b); and, with overall flow pattern (c).

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 to be twice the value of 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[7]]. 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 xs can be defined by eq. (8) as 6.5 m:

$$X_p = x_s / ((b_2 - b_1)/2) \quad \text{... (8)}$$

The sill height s is defined by eq. (9) and equals 1.6 m:

$$s = \frac{x_s / x_r + 0.0116 \cdot Fr_1 - 0.225}{0.155 - 0.008 \cdot Fr_1} \quad \text{... (9)}$$

The sill width b_s can be calculated with eq. (10) as 9.4 m:

$$b_S = b_1 \cdot (1 + 0.25 \cdot (b_2 / b_1 - 1)) \quad \text{... (10)}$$

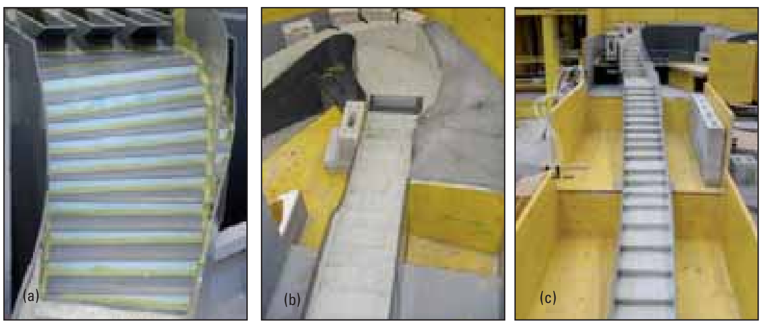
For reasons of environmental integration, the smoothly rounded side walls of the basin (Fig. 4b) are the only difference between the initial model configuration and the theoretical case. Flow patterns were therefore quite similar (see Fig. 4c).

3. Physical model tests

3.1 Experimental setup and device

Considering the hydraulic characteristics of the existing spillways, the new PKW and the restitution channel of Gloriettes dam, an overall geometrical scale of 1:30 was considered as appropriate to avoid scale effects. The Froude similarity was applied, which means that the ratio between forces of inertia and gravity is maintained.

Fig. 5. Final configuration of the dissipation channel: Transversally inclined steps downstream of the PKW (a); first channel reach and dissipation basin in the modeled topography on the left (b); and, second channel reach with roughness layer (c).



The model consisted of two connected parts. The first, reproducing the reservoir, the arch dam and the spillways, was placed in a square steel tank. The other was to model the restitution channel, which was reproduced within surrounding walls, allowing extensions in all three dimensions. The channel reaches and the stilling basin were made of PVC. For the tests on the restitution channel, the existing spillways had been trapped. This measure allowed for precise control of the discharge eminating from the PKW, supplied by the network of the laboratory and measured by an electromagnetic flowmeter. The water level in the tank was controlled by two ultrasonic sensors. 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 optimization procedure

For the performance tests and optimization an iterative and systematic procedure was applied. All the tests had been carried out originally with the design flood of 80 m^3/s. The optimized configuration was verified afterwards, with lower discharges of 20 m^3/s, 40 m^3/s 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 transversel flow distribution, the water levels on both borders of the steps were measured. The flow behaviour in the curved part immediately downstream of the PKW required particular attention. Two mean measures for flow equilibration and overflow obviation had been developed and were 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 the height of this structure, but also its shape was part of the optimization procedure, see photo (a) below.

The main function of the stilling basin downstream of the first reach of the channel is to allow for a 120º change of direction in subcritical flow conditions. The easily adaptable test installation allowed for the optimization of several elements of the basin. The bottom level, the surrounding form, the inflow and outflow configurations of the basin, the height and form of the sill as well as the adjoining natural topography were systematically adapted and qualitatively evaluated, see photo (b) below. The static and dynamic pressures on the sill in extreme flood conditions were measured. The energetic spectral density analysis aimed to define structurally the problematic frequencies.

To simulate the roughness of uncoated excavated rock, with estimated irregularities of between 5 and 10 cm on the prototype, a layer of rough cement was added at the corresponding surfaces of the channel and basin. The strips, approximately 2 mm high and 10 mm wide, were applied crosswise. The resulting roughness on the prototype corresponds theoretically to a Strickler coefficient of 35 m$^{1/3}$/s, see photo (c) below.

For the final configuration with and without roughness layers, the water levels were measured again. The outflow velocities at the end of the two channel reaches were measured by micropropeller in a 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.

Table 1: Velocity and water level measurements with efficiency of energy dissipation in the two channel reaches with and without roughness layer for four different discharges								
Discharge (m^3/s)	20		40		60		80	
	*	+	*	+	*	+	*	+
Reservoir (PKW)								
Upstream head level (el. m)	1667.3		1667.4		1667.6		1667.8	
Outlet first reach								
Ground level (el. m)	1635.0		1635.0		1635.0		1635.0	
Water level (m)	0.5	0.8	1.0	1.1	1.3	1.5	1.3	1.6
Measured 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
Downstream head level (el. m)	1636.8	1636.4	1637.5	1637.3	1638.2	1638.0	1639.7	1638.8
Outlet second reach								
Ground level (el. m)	1609.5		1609.5		1609.5		1609.5	
Water level (m)	0.4	0.6	0.7	0.9	0.9	1.1	1.0	1.3
Measured 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
Downstream head level (el. m)	1612.1	1611.1	1613.2	1612.2	1614.3	1613.2	1616.0	1614.2
Total head loss ΔH (m)	55.1	56.2	54.2	55.2	53.2	54.3	51.7	53.5
Efficiency $\Delta H/H_0$ (measured)	95%	97%	94%	95%	92%	94%	89%	92%
Efficiency $\Delta H/H_0$ (estimated)	93%		88%		85%		83%	
* = without roughness layer; + = with roughness layer								

3.3 Results

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

The average velocities at the outlet of the two reaches of the channel make it possible to estimate the efficiency of the energy dissipation, the ratio between head loss and total head (see Table 1). The efficiency decreases with an increasing discharge. This was also predicted by the calculated values, which are slightly lower than the measured ones. The 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 the causes of these differences. For the roughness layer, the measured velocities at the channel outlets are lower, and as a result the head losses and efficiency are higher.

The optimized design of the stilling basin allows for the required 120° 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 at 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 from the upper channel outlet, and the preliminary defined height of 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 has been shortened to 7.5 m, corresponding to the channel. The measured pressures on the front part of the sill are about 5.3 m water column (see Table 2) and slightly asymmetrical, which can be explained by the asymmetrical shape of the basin. Standard deviations of between 1.3 and 1.9 m indicate highly varying flow with a risk of negative values. The back of the sill is about 50 per cent less loaded.

Table 2: Measured static and dynamic pressures on the front and back parts of the sill for the design flood of 80 m^3/s							
w.c. (m)		Static pressure μ	Dynamic pressure μ	max.	min.	σ	Test configuration
	Left	5.1	5.2	13.7	-2.3	1.9	
Front part	Centre	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	
	Centre	2.3	2.4	5.2	-0.7	0.8	
	Right	2.1	2.3	6.4	-1.2	0.8	

Gloriettes dam area during the excavation of the stilling basin and the channel reaches (in July 2009). The line indicates channel axis.

4. Special feature of the construction site, cost and time schedule

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 the utilities. It takes 1.5 h to drive to the nearest concrete factory, for example.

Since the construction work can only be done in summer (from the beginning of June until the end of September), it was decided to split the project into two stages:

- Stage 1, during the summer of 2009, consisted of rock blasting and excavations for the second reach of the channel, the stilling basin, 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 the summer this year, has consisted of rock blasting and digging the upper part of the first channel reach, building the side walls of the upper first reach, sawing the concrete wall (see pictures below) and constructing the PKW.

Upstream (above) and downstream (below) view on the future installation site of the PKW before modification.

Since the arch dam is located very close to the excavation areas, see photo below, rock blasting is strictly controlled by several accelerometers located on the dam and around its foundation. In July 2009, while the stilling basin and the second reach were being blasted, a maximum velocity of 4 mm/s was recorded, with a frequency of 30 Hz.

From a geological point of view, the right bank of the river, where the new spillway and the channel reaches are being 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 (Fig. 9).

The total cost for the spillway upgrading project is about €1.2 million. The main part of the cost relates to the energy dissipation devices: the stepped reaches of the channel and the stilling basin.

5. Conclusions

Piano Key Weirs are compact and adaptive elements for increasing the flood discharge capacity of existing dams. The discharge of water downstream of these structures often requires original and innovative solutions. The Gloriettes dam spillway upgrade project has involved particularly complex boundary conditions, including a change of direction by 120°. The channel configuration, with two stepped reaches and an intermediate stilling basin, makes it possible to guide the water to the natural channel by dissipating energy. The geometrical characteristics have been pre-designed by a theoretical approach and optimized by physical modelling. Uniform flow distribution on the spillway, as well as energy dissipation of more than 90 per cent were finally achieved, minimizing the excavation volume and also meeting requirements to integrate well into the mountainous environment.

The chosen approach, which involved a preliminary design, based on a recently developed theoretical approach and simple numerical computation, followed by systematic tests on a physical model, has been satisfying in terms of both the technical and economic aspects. The iterative test programme, as well as the appropriate experimental equipment made it possible to solve a complex engineering problem, taking into account environmental, structural and economic criteria.

The project is still under construction. The first stage of project was completed in October 2009, including the downstream part of the first reach, the stilling basin and the second reach. The PKW and the upper part of the first reach is ongoing now. ◊

July 2009: Drilling machine, power shovel and hydraulic rock breaker during the excavation of the stilling basin.

October 2009: The expanding stilling basin and its sidewalls are complete.

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October 2009: The downstream part of the first stepped reach has been completed. The upper part of the reach (with the concrete steps) and the PKW was carried out recently.

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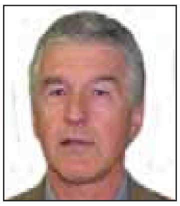

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Jean-Louis Boillat graduated in Civil Engineering from the Swiss Federal Institute of Technology in Zurich (ETHZ), Switzerland, in 1972. After joining the hydraulic laboratory of Ecole Polytechnique Fédérale de Lausanne as a Research Assistant in 1974, he obtained his Doctorate in Technical Sciences, in 1980, on the subject of the influence of turbulence on particle settling velocity. He then worked as an independent engineer on the design of many projects in Switzerland and abroad before returning to EPFL as a lecturer and research project manager, responsible for the hydraulic model studies at LCH. His field activities cover the complementary domains of education, research and services in applied hydraulics.

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