

# Comparison of hydraulic design equations for A-type Piano Key weirs

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## Introduction

Piano Key weirs (PKWs) are a hydraulically attractive alternative to linear overflow weirs, increasing the unit discharge at the unregulated spillway inlet for similar heads and spillway widths. This advantage, allowing for operation of dam reservoirs on elevated supply levels thus providing an increased retention volume, is a result of the non-linear nature of PKWs being folded back and forth to make repeating cycles or keys. As PKWs are relatively novel structures, few practical design criteria are available, so that often physical model tests of prototypes are conducted.

Nevertheless, first comprehensive and systematic model test series were performed, which provided general design equations. They include the relevant and frequent geometrical dimensions and hydraulic parameters. The paper summarizes recently published design equations relating to the head-discharge ratio of PKWs, and compares the latter with the rating curve of a standard crest profile.

## 1 Background

The Piano Key weir (PKW) represents a particular type, or development, of the Labyrinth weir. Differently to Labyrinth weirs, the structural footprint of PKWs is relatively short in streamwise direction allowing for the installation on top of existing gravity dams [1]. Additionally, the inclined key bottom (instead of a horizontal-vertical arrangement as at Labyrinth weirs) improves slightly the hydraulic efficiency for relatively small discharges [2, 3 and 4]. The construction cost is typically relatively moderate, despite of the complex geometry, as a consequence of the repetitive cycle character allowing for serial-production using pre-fabricated formwork. Thus, PKWs became a frequently applied structure in recent years, particularly built in France (see an example in Fig. 1) and presently under construction in Vietnam [5].

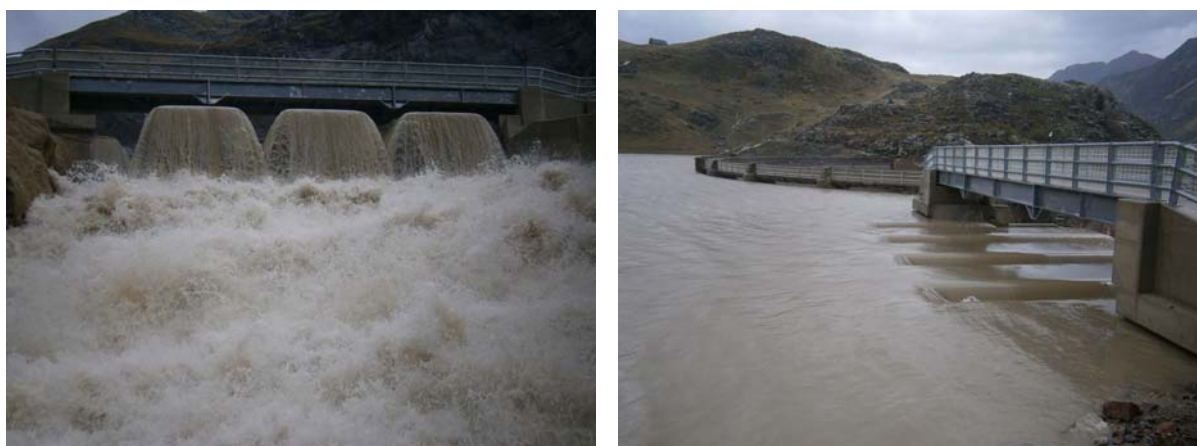


Fig. 1. PKW at Gloriettes Dam in France during operation on October 19, 2012, with a discharge of almost  $50 \text{ m}^3/\text{s}$   
(Photos EDF, Frederic Laugier)

Lempérière et al. [6] and Schleiss [7] presented historical reviews on the evolution from Labyrinth weirs to PKWs. Schleiss concluded his review in 2011 with the remark that “generalized performance curves are not yet available”. Although considerable efforts were made to understand the hydraulic flow behaviour at PKWs, few practical design criteria were available supporting the design engineer to reliably and generally describe the hydraulic efficiency of a PKW. Tests performed on scale models as well as numerical simulations contributed to the increase of knowledge. Nevertheless, most of the prototype applications were model-tested prior to construction, to assure a reliable and efficient operation of the structures (see two examples in Fig. 2).

At least four institutions performed in the meantime systemic research model investigations related to PKW hydraulics, including a wide parameter range. These institutions are: Electricité de France (EDF), the Isfahan University of Technology (Iran) [8], the Ecole Polytechnique Fédérale de Lausanne (Switzerland) [9], and the University of Liège (Belgium) [10]. The results of these basic research studies were published. A comparison of the latter is given herein, focusing on the rating curves. Additionally, a general example is discussed, comparing the hydraulic performance of a PKW (using the different design equations so far available) with that of a standard crest profile (ogee) [11] and a board-crested weir [12].

The standard nomenclature related to PKWs as given in Fig. 3 is applied [13], with  $W$  as total transverse width,  $B$  as total streamwise length,  $P$  as vertical height,  $T$  as wall thickness, and  $R$  as parapet wall height. Furthermore, subscript  $i$  refers to the inlet key, i.e. the key that is filled with water for a reservoir surface at the PKW crest elevation, and subscript  $o$  to the outlet key, i.e. the ‘dry’ key for the latter reservoir level. The developed crest length  $L$  is defined as  $L=N(W_i+W_o+2T+2B)$  or alternatively as  $L=W+(2NB)$ , with  $N$  as number of PKW cycles. The discharge is  $Q$ , and the total approach flow head is  $H$ .



Fig. 2. Examples of physical PKW models at LCH; left: Saint-Marc Dam combining a PKW with a steep chute; right: Gage II Dam combining a PKW with a side channel. Both schemes are operated by EDF

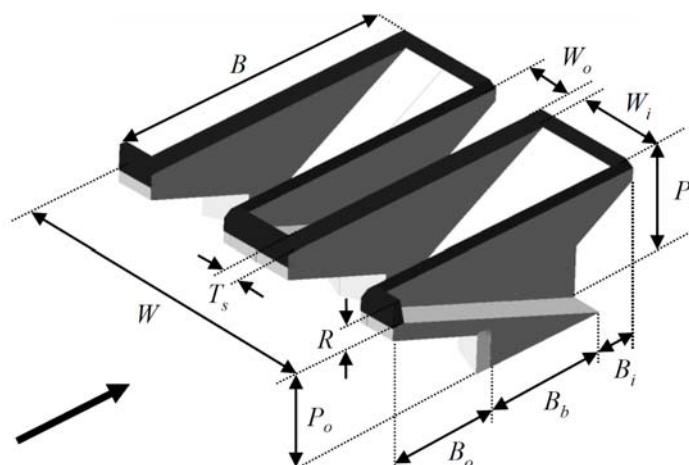


Fig. 3. Standard PKW notation [9, 13]

## 2 PKW rating curves

The head-discharge function (rating curve) of a non-gated weir structure is one of its most important hydraulic features. It ranges between (1) the reservoir full supply level as maximum normal operation level of a reservoir, and (2) the maximum reservoir level during floods and spillway operation under PMF. The latter defines a reservoir volume which is needed for flood release but is not exploitable for water storage, as typically requested for power production or freshwater supply. Or, *vice versa*, the dam must include an additional height to avoid dangerous overtopping at PMF. It is technically possible to reduce that additional height by providing a broad spillway crest generating small specific discharges, for example including a side channels or a stepped spillway. Both types are only efficient and economical under particular conditions. For an increase of the spilling capacity at existing dams, such structures are often difficult to add, so that alternatives are preferred. PKWs include a high performance in terms of their rating curve, i.e. they spill large specific discharges under relatively small heads, if compared with an ogee crest. Before comparing their relative efficiency illustrated on a particular case, the so far published rating curves are presented herein, namely those of Kabiri-Samani and Javaheri [8], Leite Ribeiro et al. [9], and Machiels [10].

All three studies considered A-type PKWs. The experimentally obtained rating curves are subjected to the limits resulting from physical modelling and its related parameter variation, as shown in Table 1. These limits have to be respected when deriving reliable rating curves, as some equations are rather sensitive to them [14]. All studies excluded small heads below some 0.03 to 0.05 m to avoid scale effects.

Table 1. Application limits of the herein considered A-type PKW capacity equations

	$L/W$	$H/P$	$W_i/W_o$	$B/P$	$B_i/B, B_o/B$
Kabiri-Samani and Javaheri [8]	2.5 to 7.0	0.1 to 0.6	0.33 to 1.22	1.0 to 2.5	0.00 to 0.26
Leite Ribeiro et al. [9]	3.0 to 7.0	0.1 to 2.8	0.50 to 2.00	1.5 to 4.6	0.20 to 0.40
Machiels [10]	4.2 to 5.0	0.1 to 5.0	0.50 to 2.00	1.0 to 6.0	0.29 to 0.33

### 2.1 Study of Kabiri-Samani and Javaheri [8]

Kabiri-Samani and Javaheri worked with a 12 m long and 0.4 m wide channel, conducting totally 600 tests resulting in some 3000 data points. The tested PKWs included types A, B, and C, all operated under specific discharges between 25 and 175 l/sm. They investigated the PKW performance under free flow and submerged conditions, i.e. with an effect of the tailwater. Herein, only the free weir flow type is discussed, in order to compare the results later with other design equations.

The Poleni equation of an ogee crest served as base to derive the rating curve, defined as

$$Q_{PKW} = \frac{2}{3} C_d W \sqrt{2gH}^{1.5} \quad (1)$$

with  $g$  as gravitational acceleration and  $C_d$  as discharge coefficient. Based on a dimensional analysis, they derived the PKW discharge coefficient  $C_d$  as a function of the relevant geometrical and hydraulic parameters as

$$C_d = \left[ 0.212 \left( \frac{H}{P} \right)^{-0.675} \left( \frac{L}{W} \right)^{0.377} \left( \frac{W_i}{W_o} \right)^{0.426} \left( \frac{B}{P} \right)^{0.306} \exp \left( 1.504 \frac{B_o}{B} + 0.093 \frac{B_i}{B} \right) \right] + 0.606 \quad (2)$$

The coefficient of determination between their model data and Eq. (1) combined with Eq. (2) is reported as  $R^2=0.99$ .

### 2.2 Study of Leite Ribeiro et al. [9]

Leite Ribeiro et al. performed systematic model tests in a large channel, reduced to an effective test reach of 3 m length and 0.5 m width. In total, 380 tests were conducted, considering 49 different PKW geometries of type A. The specific model discharge varied between 26 and 440 l/sm. Uniquely free overflow without submergence from the tailwater was tested.

The rating curve is expressed relatively, i.e. normalizing the PKW discharge  $Q_{PKW}$  with the reference discharge of a linear sharp-crested weir. The latter is expressed as  $Q_s=0.42W(2gH^3)^{0.5}$  [15], so that the discharge increase ratio  $r$  follows as

$$r = \frac{Q_{PKW}}{Q_s} = \frac{Q_{PKW}}{0.42W\sqrt{2gH}^{1.5}} \quad (3)$$

The values of  $r$  were found to be a function of primary and secondary parameters. The primary parameters with a dominant effect on the discharge capacity of a PKW are the developed length  $L$ , the total transverse width  $W$ , the vertical height  $P_i$ , and the head  $H$ . The secondary parameters influence the discharge capacity in the order of few per cents, including the ratio of inlet to outlet key width  $W_i/W_o$ , the ratio of inlet to outlet height  $P_i/P_o$ , the relative overhang length  $(B_i+B_o)/B$ , and the relative parapet wall height  $R_o/P_o$ . The influence of the secondary parameters was expressed with individual correction factors, namely  $w$ ,  $p$ ,  $b$ , and  $a$ .

The data analysis indicated that

$$r = 1 + 0.24 \left( \frac{(L-W)P_i}{WH} \right)^{0.9} \cdot (wpba) \quad (4)$$

Here,  $r=1$  for  $L=W$  similar to sharp-crested weirs or for small  $P_i$  combined with large  $H$ , so that the structure of the PKW is negligible as compared to the large head. It is assumed that the term  $wpba$  is equal to 1 for both extreme scenarios as discussed before. Even the latter pragmatic approach, i.e. to set  $wpba=1$ , results in a reliable prediction of  $r$  with  $R^2=0.96$  if taking into account all 380 tests. This again indicates that the primary parameters are dominant and to optimize during the design process.

Nevertheless, the secondary parameters can be taken into account to further increase the precision of Eq. (4), so that  $R^2=0.98$ . The values of the individual correction factors are maximally between 0.92 and 1.20 for the geometries tested by Leite Ribeiro et al. The individual correction factors are defined as given below, namely for the ratio between inlet and outlet key width as

$$w = \left( \frac{W_i}{W_o} \right)^{0.05} \quad (5)$$

for the ratio between outlet and inlet key height as

$$p = \left( \frac{P_o}{P_i} \right)^{0.25} \quad (6)$$

for the ratio between key overhangs to the base length as

$$b = \left( 0.3 + \frac{B_o + B_i}{B} \right)^{-0.50} \quad (7)$$

and for the relative parapet wall height as

$$a = 1 + \left( \frac{R_o}{P_o} \right)^2 \quad (8)$$

Furthermore, the study indicated that sectional models using exclusively 1.5 PKW cycles provide reliable discharges per cycle [16]. This result was derived from a systematic variation of the cycle number in the model and facilitates the set-up of similar research facilities. The physical model is of channel approach flow type, ignoring the effects of a reservoir type approach flow. To compensate this simplification, additional case study model tests including reservoirs were analysed to estimate the latter effect.

### 2.3 Study of Machiels [10]

Machiels conducted systematic model tests in a 7.2 m long and 1.2 m wide flume. For each parameter set investigated, a 0.3 m wide PKW model including 2.5 cycles was inserted into the channel, charged with specific discharges between 13 and 400 l/sm. Again, the proposed design equation is only valid for free overflow conditions.

Starting with an analytical approach, Machiels gives the specific PKW discharge as the sum of the specific discharges on the downstream (subscript  $d$ ), the upstream (subscript  $u$ ) and the side (subscript  $s$ ) crests, so that

$$q = \frac{Q_{PKW}}{W} = q_u \frac{W_o}{W_u} + q_d \frac{W_i}{W_u} + q_s \frac{2B}{W_u} K_{w_i} K_{w_o} \quad (9)$$

With  $W_u$  as cycle width, defined as  $W_u = W_i + W_o + 2T$ . The specific discharges are given as

$$q_u = 0.374 \left( 1 + \frac{1}{1000H + 1.6} \right) \left( 1 + 0.5 \left( \frac{H}{H + P_T} \right)^2 \right) \sqrt{2gH^3} \quad (10)$$

$$q_d = 0.445 \left( 1 + \frac{1}{1000H + 1.6} \right) \left( 1 + 0.5 \left( \frac{H}{H + P} \right)^2 \right) \sqrt{2gH^3} \quad (11)$$

$$q_s = 0.41 \left( 1 + \frac{1}{833H + 1.6} \right) \left( 1 + 0.5 \left( \frac{0.833H}{0.833H + P_e} \right)^2 \right) \left( \frac{P_e^\alpha + \beta}{(0.833H + P_e)^\alpha + \beta} \right) \sqrt{2gH^3} \quad (12)$$

Here,  $P_T = P + P_d$ , with  $P_d$  as dam height below the PKW. Furthermore,  $P_e$  is the mean weir height along the side wall, given as

$$P_e = P_T \frac{B_o}{B} + \frac{P}{2} \left( 1 - \frac{B_o}{B} \right) \quad (13)$$

The parameters  $\alpha$  and  $\beta$  characterize the influence of the inlet key slope on the side crest discharge efficiency. They are

$$\alpha = \frac{0.7}{S_i^2} - \frac{3.58}{S_i} + 7.55 \quad (14)$$

$$\beta = 0.029 \exp \left( -\frac{1.446}{S_i} \right) \quad (15)$$

with  $S_i$  as slope of the inlet key, given as  $S_i = P/(B - B_o)$ . Furthermore,  $K_{wi}$  describes the influence of the flow velocity variation along the side crest on its discharge efficiency, as

$$K_{wi} = 1 - \frac{\gamma}{\gamma + W_i^2} \quad (16)$$

$$\gamma = -0.0038 \frac{W_i}{W_o} + 0.0055 \quad (17)$$

The parameter  $K_{wo}$  takes into account the side crest length decrease induced by the outlet key flow and the side nappe interference. It depends on  $H/W_o$  as

$$K_{wo} = 1 \quad \text{for } H/W_o \leq L_1 \quad (18)$$

$$K_{wo} = \frac{2}{(L_2 - L_1)^3} \left( \frac{H}{W_o} \right)^3 - \frac{3(L_2 + L_1)}{(L_2 - L_1)^3} \left( \frac{H}{W_o} \right)^2 + \frac{6L_2L_1}{(L_2 - L_1)^3} \left( \frac{H}{W_o} \right) + \frac{L_2^2(L_2 - 3L_1)}{(L_2 - L_1)^3} \quad (19)$$

$$K_{wo} = 0 \quad \text{for } H/W_o \geq L_2 \quad (20)$$

Finally, the limits  $L_1$  and  $L_2$  for the use of Eqs. (18) to (20) include the slope of the outlet key, given as  $S_o = P/(B - B_i)$ , and are defined as

$$L_1 = -0.788 S_o^{-1.88} + 5 \quad (21)$$

$$L_2 = 0.236 S_o^{-1.94} + 5 \quad (22)$$

Machiels presents a hydraulic analysis including a very detailed description of the flow phenomena, water surfaces, pressures, streamlines and flow velocities. Furthermore, he adds technical-economic considerations regarding the dimensions of a PKW.

### 3. Comparison

The rating curves of the three above presented studies are compared herein, based on a virtual prototype. The latter has to be defined in accordance with the limits of the studies as given in Table 1, as far as possible. The following prototype situation is used, including

- A gravity dam with a  $W=100$  m wide chute spillway on its downstream face (e.g. an RCC dam), a dam height of  $P_d=30$  m below the PKW foundation, free overfall conditions at the weir without tailwater submergence, and ignoring the effect of the distal weir ends.
- A design discharge of  $Q_D=2'500$  m<sup>3</sup>/s, according to a specific discharge of 25 m<sup>3</sup>/s (as frequently applied on stepped spillways [17]).
- A symmetrical A-type PKW, mounted at the dam crest as unregulated control structure, with  $B=8.00$  m as total streamwise length,  $P=P_i=P_o=5.00$  m as vertical height,  $T=0.35$  m as wall thickness,  $R=0$  m (without parapet walls),  $W_i=1.80$  m as inlet key width,  $W_o=1.50$  m as outlet key width, and  $B_i=B_o=2.00$  m as overhang lengths.

The following characteristics follow from the above chosen PKW geometry: developed crest length  $L=500$  m, cycle width  $W_u=W_i+W_o+2T=4.00$  m, number of cycles  $N=W/W_u=25$ ,  $L/W=5.00$ ,  $B/P=1.60$ ,  $W_i/W_o=1.20$ ,  $B_i/B_o=0.25$ , and  $S_i=S_o=0.83$ . To derive the overflow capacity, water heads between 0 to 6 m were considered.

The resulting rating curves are shown in Fig. 4a for the three herein presented studies, also respecting their application limits in terms of  $H/P$ . Furthermore, the rating curve of a standard crest profile (ogee) is provided, computed following Vischer and Hager [11] considering a design head of  $H_D=5.00$  m for  $Q_D$ . Finally, the rating curve of a broad-crested profile is included in Fig. 4a, based on Hager and Schwalt [12]. All PKW curves start at typically  $H/P=0.1$  what is identical with  $H=0.50$  m due to the lower model limitation to avoid scale effects. The error would probably be small if applying the equations also in the reach  $H<0.50$  m, as the effects of water surface tension and water viscosity occurs for a head of some  $H<0.3T$  on prototype. For a large head of  $H=5.00$  m, the predictions of Leite Ribeiro et al. and Machiels result in  $Q=4'040$  m<sup>3</sup>/s and  $Q=3'590$  m<sup>3</sup>/s, respectively. For the design discharge of  $Q_D=2'500$  m<sup>3</sup>/s, the equation of Leite Ribeiro et al. gives  $H=3.12$  m and Machiels gives  $H=3.38$  m. Interesting is the comparison of the PKW discharge capacity with that of the ogee. For the latter, the head  $H=5.00$  m for  $Q_D=2'500$  m<sup>3</sup>/s according to its design criterion. The flow head of the ogee is thus between 150 and 160% of that of the PKW. Or, in other words, the absolute head difference is around 1.62 and 1.88 m. Accordingly, the dam height might be reduced by this height difference, resulting in lower construction cost, or the height difference is available to generate additional storage volume.

Figure 4b shows the discharge ratio  $\eta=Q_{PKW}/Q_{Ogee}$  in function of the head  $H$ . As visible, the efficiency of PKWs is particularly high for small heads. Then, the developed length  $L$  is fully active, similar to a very long linear sharp crested weir. For a head of  $H=1.00$  m, a PKW spills around 4 to 5 times more discharge than an ogee, dependent on the chosen equations to derive the PKW discharge. Note that the ratio  $L/W=5.00$  herein, a value being close to the aforementioned capacity increase. For large discharges, the flow interacts in the wedges and corners of the PKW crest (in plan view), so that the capacity decreases and finally approaches that of an ogee. For heads  $H>3.50$  m a discharge ratio of  $\eta<2$  occurs. Even though these heads are hydraulically still efficient on PKWs, they might be uninteresting from an economical point of view as compared to the construction cost of an ogee crest.

Figure 4a indicates that, for the herein chosen conditions, the rating curves of the three presented PKW studies essentially collapse. In general, the empiric equation of Leite Ribeiro et al. [9] predicts the highest discharge capacity, and the relationship of Machiels [10] gives the lowest values. To further compare the three PKW discharge predictions, an absolute and a relative discharge difference are shown in Fig 4c. There, the left ordinate indicates the absolute discharge difference for a given head, with the data of Machiels [10] as basis. It is visible that the absolute difference is – if subtracting the values following Machiels [10] from those of Leite Ribeiro et al. [9] – in the order of 200 to 300 m<sup>3</sup>/s for heads below  $H<4$  m, and of around 450 m<sup>3</sup>/s for  $H=5$  m. The same trend is recognisable for the subtraction of the Machiels [10] values from those of Kabiri-Samani and Javaheri [8]: Small heads result in small discharge difference, and large heads above 3 m in large absolute discharge differences. In parallel, Fig. 4c shows the relative discharge ratio on its right ordinate, dividing one predicted discharge by another, both for the same head. Again, the values of Machiels are taken as reference. Both ratios – that of the values following Kabiri-Samani and Javaheri [8] divided by these of Machiels [10], and that of Leite Ribeiro et al. [9] divided by Machiels [10] – show that the relative deviation is large for small heads below approximately 1.5 m, and small (i.e. around 10%) for head larger than 1.5 . Herein, the best correlation is achieved for heads of 2 to 3 m.

Figure 4 thus globally shows that PKWs are highly efficient to spill large specific discharges under small heads, particularly if compared to ogee crests. Furthermore, the herein discussed equations to predict the PKW rating curves give similar, but not identical results.

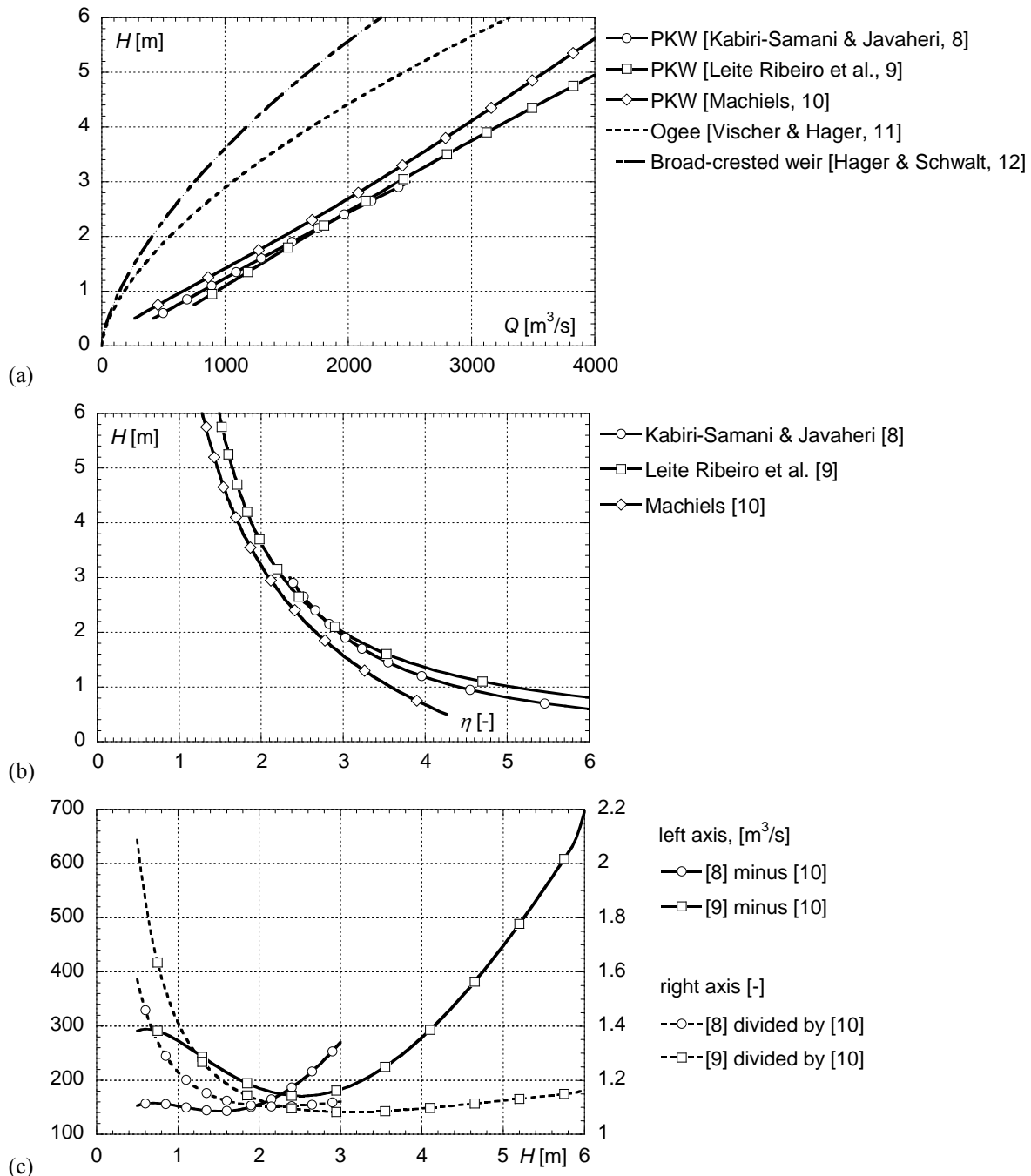


Fig. 4. Comparisons of (a) rating curves for PKW, ogee crest and broad-crested weirs, (b) relative discharge capacity of PKW with ogee as reference (ogee capacity as  $\eta=1$ ), and (c) the three PKW relationships as presented herein

#### 4. Conclusions

Piano Key weirs are a hydraulically efficient weir structure, with a typically higher specific discharge performance than an ogee crest. They thus (1) spill an increased discharge under identical heads and widths, (2) have a reduced width for a given discharge and head, or (3) require a reduce head for a certain weir width and a given discharge. So far, three general design equation related to PKWs were published, which are compared herein. Based on an application example, it was found that the predictions basically give similar discharges for a certain head, but that some discrepancies nevertheless exist. For the design process, a consideration and discussion of several equations is thus recommended.

## References

1. **Lempérière, F. and Ouamane, A.**, “The Piano Keys weir: a new cost-effective solution for spillways”, *International Journal of Hydropower and Dams*, Volume 10, Issue 5, pp. 144-149, 2003.
2. **Laugier, F., Lochu, A., Gille, C., Leite Ribeiro, M. and Boillat, J.-L.**, “Design and construction of a labyrinth PKW spillway at St-Marc Dam, France”, *International Journal of Hydropower and Dams*, Volume 15, Issue 5, pp. 100-107, 2009.
3. **Anderson, R.M. and Tullis, B.**, “Influence of Piano Key Weir geometry on discharge”, Proc. Intl. Conf. *Labyrinth and Piano Key Weirs*, Liège B, 75-80. CRC Press, Boca Raton FL, 2011.
4. **Anderson, R.M. and Tullis, B.**, “Comparison of Piano Key and rectangular Labyrinth Weir hydraulics”, *Journal of Hydraulic Engineering*, Volume 138, Issue 4, pp. 358-361, 2012.
5. **Ho Ta Khan, M., Truong, C.H. and Nguyen, T.H.**, “Main results of the P.K. weir mode tests in Vietnam (2004 to 2010)”, Proc. Intl. Conf. *Labyrinth and Piano Key Weirs*, Liège B, 191-198. CRC Press, Boca Raton FL, 2011.
6. **Lempérière, F., Vigny, J.P. and Ouamane, A.**, “General comments on Labyrinth and Piano Key Weirs: The past and present”, Proc. Intl. Conf. *Labyrinth and Piano Key Weirs*, Liège B, 17-24. CRC Press, Boca Raton FL, 2011.
7. **Schleiss, A.J.**, “From labyrinth to piano key weirs: A historical review”, Proc. Intl. Conf. *Labyrinth and Piano Key Weirs*, Liège B, 3-15. CRC Press, Boca Raton FL, 2011.
8. **Kabiri-Samani, A. and Javaheri, A.**, “Discharge coefficient for free and submerged flow over Piano Key weirs”, *Journal of Hydraulic Research*, Volume 50, Issue 1, pp. 114-120, 2012.
9. **Leite Ribeiro, M., Pfister, M., Schleiss A.J., and Boillat, J.-L.**, “Hydraulic design of A-type Piano Key Weirs”, *Journal of Hydraulic Research*, Volume 50, Issue 4, pp. 400-408, 2012.
10. **Machiels, O.**, “Experimental study of the hydraulic behaviour of Piano Key Weirs”, PhD *Thesis* ULgetd-09252012-224610, University of Liège (B), 2012.
11. **Vischer, D. and Hager, W.H.**, *Dam Hydraulics*. Wiley, Chichester UK, 1999.
12. **Hager, W.H. and Schwalt, M.**, “Broad-crested weir”, *Journal of Irrigation and Drainage Engineering*, Volume 120, Issue 1, pp. 13-26, 1994.
13. **Pralong, J., Vermeulen, J., Blancher, B., Laugier, F., Erpicum, S., Machiels, O., Piroton, M., Boillat, J.L., Leite Ribeiro, M. and Schleiss, A.J.**, “A naming convention for the piano key weirs geometrical parameters. Proc. Intl. Conf. *Labyrinth and Piano Key Weirs*, Liège B, 271-278. CRC Press, Boca Raton FL, 2011.
14. **Pfister, M., Erpicum, S., Machiels, O., Schleiss, A.J. and Piroton, M.** “Discussion to ‘coefficient for free and submerged flow over Piano Key weirs’”, *Journal of Hydraulic Research*, Volume 50, Issue 6, pp. 642-643, 2012.
15. **Hager, W.H. and Schleiss, A.J.**, *Constructions hydrauliques, Ecoulements stationnaires* (Hydraulic structures, steady flow), *Traité de Génie Civil 15*. Presses Polytechniques et Universitaires Romandes, Lausanne, Switzerland, 2009.
16. **Leite Ribeiro, M., Pfister, M., Boillat, J.-L., Schleiss, A.J. and Laugier, F.**, “Piano key weirs as efficient spillway structure”, 24th ICOLD *Congress on Large Dams*, Kyoto (J), Q.94 – R.13, 2012.
17. **Pfister, M., Hager, W.H. and Minor, H.-E.**, “Bottom aeration of stepped spillways”, *Journal of Hydraulic Engineering*, Volume 132, Issue 8, pp. 850-853, 2006.

## The Authors

**Anton J. Schleiss** graduated in Civil Engineering from the Swiss Federal Institute of Technology (ETH) in Zurich, Switzerland, in 1978. After joining the Laboratory of Hydraulic, Hydrology and Glaciology at ETH as a research associate and senior assistant, he obtained a Doctorate of Technical Sciences on the topic of pressure tunnel design in 1986. After that he worked for 11 years for Electrowatt Engineering Ltd. in Zurich and was involved in the design of many hydropower projects around the world as an expert on hydraulic engineering and underground waterways. Until 1996 he was Head of the Hydraulic Structures Section in the Hydropower Department at Electrowatt. In 1997 he was nominated Full Professor and became Director of the Laboratory of Hydraulic Constructions (LCH) in the Civil Engineering Department of EPFL. The LCH activities comprise education, research and services in the field of both, fundamental and applied hydraulics. The research focuses on the interaction between water, sediment-rock, air and hydraulic structures as well as associated environmental issues and involves both numerical and physical modelling. Prof. Schleiss is also involved as an international expert in several dam and hydropower plant projects all over the world as well as flood protection projects mainly in Switzerland. From 2006 to 2012 he was Director of the Civil Engineering program of EPFL and chairman of the Swiss Committee on Dams (SwissCOLD). In 2006 he obtained the ASCE Karl Emil Hilgard Hydraulic Price as well as the J.C. Stevens Award. He was listed in 2011 among the 20 international personalities that “have made the biggest difference to the sector Water Power & Dam Construction over the last 10 years”. In 2012, he was elected Vice-President of the International Commission on Large Dams (ICOLD) representing the zone Europe.

**Michael Pfister** graduated in Civil Engineering from ETH Zurich, Switzerland, in 2002. He then joined the Laboratory of Hydraulics, Hydrology and Glaciology (VAW) at ETH Zurich as a research- and senior-assistant, and obtained a Doctorate in Sciences in 2007, based on his research on chute aerators. In 2010 he joined the Laboratory of Hydraulic Constructions (LCH) of Ecole Polytechnique Fédérale de Lausanne (EPFL), Switzerland, as a Research and Teaching Associate, continuing his research activity mainly focusing on spillway hydraulics and high-speed two-phase air-water flows.