CHAPTER 6 – PIANO KEY WEIRS SPILLWAYS

1. GENERAL DESCRIPTION

The recently developed Piano Key weir (PKW) spillway, which is an innovative structure that can convey very high specific discharges, is a variation of the traditional labyrinth weirs, firstly devised to circumvent some drawbacks of the latter (Barcouda et al., 2006). Using a rectangular layout and ramped floors which create overhanging or cantilevered apexes, the PKW is structurally simple and efficient, can be placed on existing or new gravity dam crest sections and multiplies significantly the discharge capacity compared to a standard weir of similar width (Ouamane and Lempérière, 2003).

Figure 6.1 View of the PKW spillway of Gloriettes Dam in France during construction

(Photo courtesy of EDF)

Figure 6.2 View of the PKW spillway of Malarce Dam in France during spillage

(Photo courtesy of EDF)

The succession of inclined apexes turned alternatively in upstream and in downstream direction gives the name Piano Key weir. Compared to a rectangular labyrinth weir with a common crest layout (in plan form), the PKW has the main advantage that it can be more easily installed at sites featuring limited foundation space (e.g., crest of a gravity dam). In addition, the ramped floors reduce the vertical walls height and thus the volume of reinforcing steel required in concrete. These are the reasons why PKW spillways are an efficient and economical solution for the increase of the flood releasing capacity at existing dams.

PKW has been first proposed by Hydrocoop in collaboration with the Hydraulic Laboratory of Electricité de France (France), the Roorkee University (India) and the Biskra University (Algeria) (Ouamane and Lempérière, 2003). Since its invention, several works have been carried out all over the world to understand its hydraulic behaviour, optimize its design and objectify its advantages and drawbacks (see for instance Blanc and Lempérière, 2001; Barcouda et al., 2006; Ouamane and Lempérière, 2006; Truong Chi et al., 2006; Machiels et al., 2011a & 2014; Leite Ribeiro et al., 2012a & b; Machiels, 2012; Anderson and Tullis, 2012 & 2013). Studies are still ongoing, for instance with a trapezoidal layout (Cicéro et al., 2013a).

The first PKW was installed by EDF in 2006 at Goulours dam in France (Laugier, 2007). Since then PKW have been used to increase the flood discharge capacity of five other EDF dams, namely St. Marc (2008), Etroit (2009), Gloriettes (2010), Malarce (2012) and Charmine (2014) or as new overflow structure (Escouloubre (2011)). Lessons learned from the design of these first PK weir spillways can be found in Vermeulen et al. (2011) and Laugier et al. (2013) or Laugier et al. (2009). Other PKW are presently in operation in Vietnam (Ho Ta Khanh et al., 2011a & 2012), Sri Lanka (Jayatillake H. and Perera K., 2013), Switzerland (Eichenberger P., 2013) and Scotland (Ackers et al., 2013). New PKW are under study or construction in Vietnam (Ho Ta Khanh et al., 2011a & 2012), France (Dugué et al., 2011; Erpicum et al., 2011b; Loisel et al., 2013; Bail et al., 2013), Algeria (Erpicum et al., 2012), South Africa (Botha A. et al., 2013) or India (Das Singhal & Sharma, 2011). These works are part of the rehabilitation of existing dams (to increase discharge capacity) or new projects, with PKW built in the river (diversion weir), on the top of a gravity dam, or on a reservoir bank.

Most of the information available so far on PKW has been published in two books (Erpicum et al, 2011a and 2013a), edited following two specialized workshop held in Belgium (2011) and France (2013).

2. GEOMETRY AND TYPES

The PKW geometry may appear as complex. It involves indeed a large set of parameters. In order to unify the notations, a nomenclature specific to the structure has been developed (Pralong et al., 2011a).

Following this nomenclature, the "PKW-unit" can be defined as the basic structure of the weir. It is made of two side walls, an inlet and two half-outlet keys (Figure 5.15). The main geometric parameters of the structure are the heights of the inlet and outlet keys P_i and P_o, their widths W_i and W_o, the unit width W_u, the number of PKW-units N_u, the longitudinal crest length B_h, the lengths B_o and B_i of the up- and downstream overhangs, the base length B_b and the wall thickness T_s. Subscripts *i, o* and *s* refer respectively to the inlet key, the outlet key and the side wall. W_u is equal to $W_i + W_o + 2T_s$ and the total width W of the weir is equal to $N_u W_u$. The PKW-unit developed crest length L_u of a PKW-unit is equal to $W_u + 2B_h$ and the total developed crest length L of the weir is equal to $N_u L_u$. Parapet walls (vertical extensions of the crest) may be added to the weir. Their height is referred to as P_p.

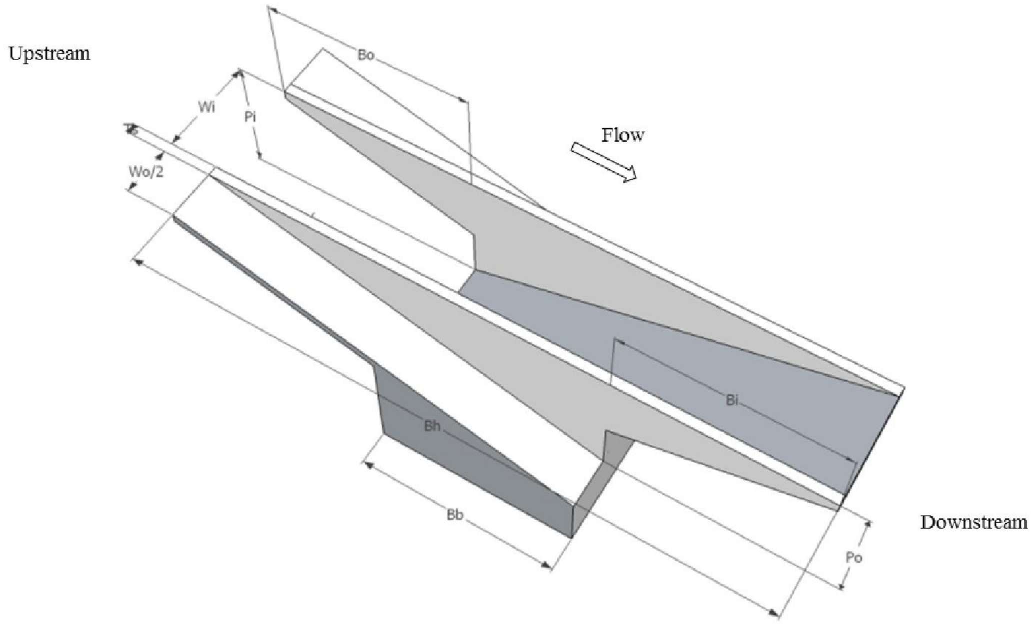

Figure 6.3 Sketch of a PKW unit and main notations (Erpicum et al. 2013b)

Depending on which PKW apexes have overhangs (upstream and/or downstream), PKW have been classified in 4 types (Truong et al., 2006): type-A with symmetric overhangs, type-B with a single upstream overhang, type-C with a single downstream overhang and type-D without overhang (Figure 5.16).

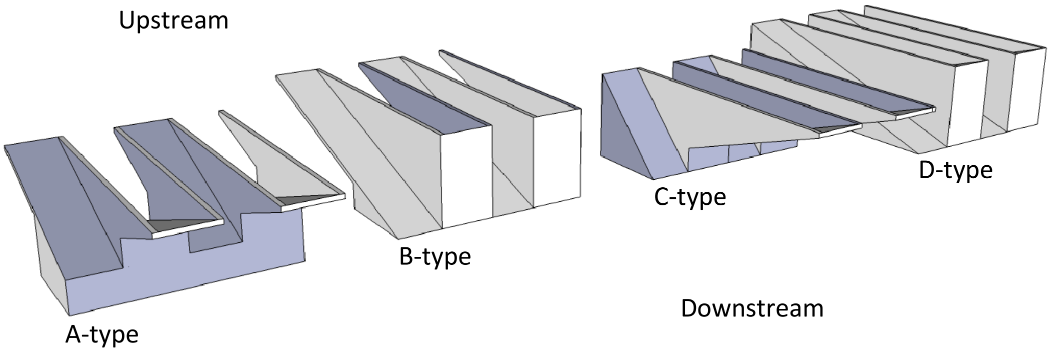

Figure 6.4 Types of PK weir (adapted from Erpicum et al. 2013b)

3. DISCHARGE CAPACITY

The PKW is a free surface weir and its discharge Q_P is thus proportional to the upstream head H as

$$Q_P = \propto \sqrt{2gH^3} \quad (3.1)$$

As summarized by Leite Ribeiro et al (2012a), two approaches may be chosen to derive the proportion factor, which represents the effect of the crest length and shape.

Referring to the developed crest length L, the discharge coefficient $C_{P,L}$ is closely related to the crest shape. Eq. 3.1 writes as (Leite Ribeiro et al., 2012b)

$$Q_P = C_{P,L} L \sqrt{2gH^3} \tag{3.2}$$

In this approach, L varies with the head as the effective crest length decreases with increasing heads because of local submergence on the upstream apex for instance. $C_{P,L}$ also varies with the head as it includes both frontal and side weirs effects.

Referring to the width of the weir W, Eq. 3.1 writes (Ouamane & Lempérière, 2006; Machiels et al., 2011a)

$$Q_P = C_{P,W} W \sqrt{2gH^3} \tag{3.3}$$

with a discharge coefficient $C_{P,W}$ accounting for both crest shape and developed length effects.

Whatever the approach used to model the PKW discharge, it is of common use to look at its discharge capacity by comparison with the one of a standard linear weir of same width. The discharge increase ratio r is defined by Leite Ribeiro et al (2012a & b) as

$$r = \frac{Q_P}{Q_S} \tag{3.4}$$

Considering Eq. 3.3, Eq. 3.4 writes

$$r = \frac{C_{P,W}}{C_S} \tag{5.7}$$

where C_S is the discharge coefficient of the standard linear weir.

Whatever its geometry, a PKW is much more efficient than an ogee crested weir of same width, especially for low heads (Figure 6.5). This explains why most of the existing PKW have been designed for a maximum H/P ratio lower than 1 (Pfister et al., 2012). This high increase in efficiency is due to the developed length of the crest which equals several times the weir width while the discharge coefficient is close to the one of a sharp crested weir. Regarding a traditional labyrinth weir with the same cycle shape in plan view (same crest print), a PKW is around 10% more efficient for a head H equal to its height P_i, as shown in a study performed by Anderson and Tullis (2012).

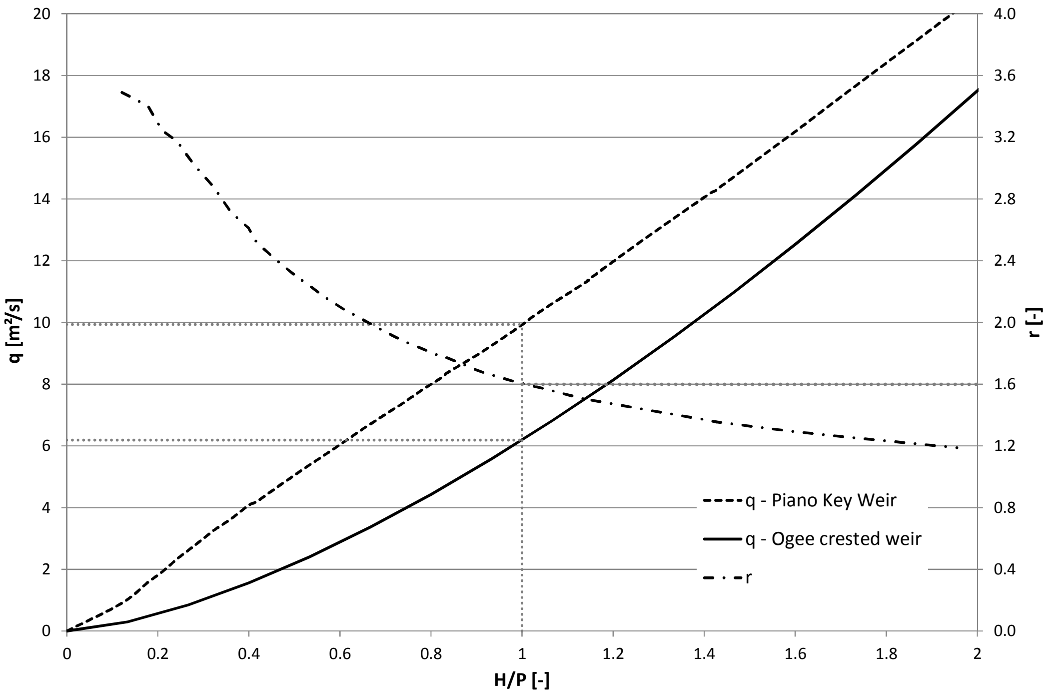

Figure 6.5 Discharge per unit width *W* of a PKW compared to an ogee crested weir – PKW: $P=P_i=P_o=2$ m and $L/W=5$; Ogee crested weir: design head=2 m, $C_S=.494$ (Erpicum et al. 2013b)

4. MAIN GEOMETRICAL PARAMETERS

The PKW discharge efficiency results from the cumulative effects of (i) three differents types of overflow (linear weir flow over the inlet key apex, linear weir flow over the outlet key apex, and lateral weir flow over the sidewalls), (ii) the developed length of the crest and (iii) the upstream head. Clinging, leaping or springing nappes may happen, depending upon the geometrical parameters of the structure and the head (Machiels et al., 2011a). The greater the head, the smaller the PKW's efficiency increase compared to a linear weir, according to the progressive reduction of the effective developed length and the saturation of the outlet.

Ouamane & Lempérière (2006) and Leite Ribeiro et al (2012a) showed that the crest length magnification ratio L/W is the main parameter controlling the discharge capacity. A value of 5 seems to be a reasonable compromise between weir efficiency and structure complexity (Lempérière, 2009; Lempérière et al, 2011), while L/W ratio of existing PKW ranges from 4 to 8 (Pfister et al, 2012). In a further step, Machiels et al (2011a and 2012b) identify the key height P, the key width ratio W_i/W_o and the overhangs positions ratio B_o/B_i as the main geometric parameters influencing the PKW hydraulic efficiency for a given L/W ratio.

These studies showed that it is of major importance to increase all the geometrical ratios which increase the inlet cross section, as this last one can be seen as the "engine" of the PKW. Increasing the inlet cross section decreases the flow velocity along the lateral crest, and thus increases its efficiency. Maximum value of the parameters is reached when the release capacity of the outlet key is affected. Indeed, the outlet key can be seen as the “brake” of the PKW. Too small of an outlet key cross section and slope increase the free surface level over the lateral crest elevation (local submergence) and thus significantly limit the weir efficiency.

A PKW design with a height ratio P/W_u equal to 1.3, a key widths ratio W_i/W_o equal to 1.25 and an overhangs lengths ratio B_o/B_i equal to 3 was found by Machiels (2012) to provide the highest discharge capacity when the L/W ratio is equal to 5. This finding is consistent with the findings of Leite Ribeiro et al (2012a), Anderson & Tullis (2013) or Lempérière et al (2011).

However, the Machiels’ study also highlights the importance of the technical and economic criteria in the definition of an optimal PKW design. A high PKW (P/W_u = 1.3) is more effective from a hydraulic point of view and should thus be considered for new dam projects, shorter PKW designs ($P/W_u \approx 0.5$), though less hydraulically efficient, would be more practical for dam rehabilitation projects. For the later, W_i/W_o and B_i/B_o ratios equal to 1 are relevant (Erpicum et al, 2104).

Each PKW’s design will be a compromise of the aforementioned parameters in order to find an appropriate answer to the specific project’s features. In any case, for a given available width, the PKW has (i) a great efficiency at low relative heads (H/P_i), and (ii) can provide a discharge capacity 2 to 5 times greater than an ogee crest using the same width.

5. TAILWATER SUBMERGENCE

Weir tailwater submergence occurs when the elevation of the downstream water surface exceeds the crest elevation and increases the upstream water elevation for a give discharge. For most applications, PKW submergence would only be a consideration for in-channel or in –river structures and not for top-of-dam applications.

Similarly to labyrinth weirs, PKW tailwater submergence has been studied mainly using physical modeling in channel configurations (Belaabed and Ouamane, 2011; Cicéro and Delisle, 2013b; Dabling and Tullis, 2012). It appears that the PKW behaviour regarding submergence is very sensitive to the geometry and type (Figure 6.6), and deserves particular care to be predicted.

Dabling and Tullis (2012) concluded that for relatively low levels of submergence, the PKW requires less upstream head relative to the labyrinth weir to pass a given discharge. This increase in efficiency was smaller than 6%, and this trend reversed at higher submergence levels.

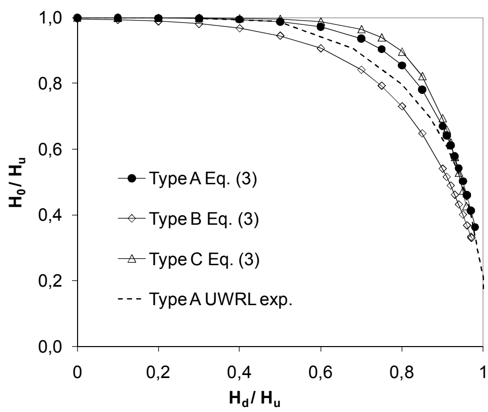

Figure 6.6: Comparison of the sensitivity to submergence of three PKW different types (Cicéro and Delisle, 2013b)

6. FLOATING DEBRIS

Debris is often transported to hydraulic structures during flood events (Pfister et al. 2013b). The accumulation of debris on PKW should be considered in situations where there is the potential for debris to reduce spillway capacity. For example, storm events in forested catchments may result in flows laden with woody debris. Depending upon the spillway hydraulics and site conditions, this type of debris may pass over the weir during floods and require little maintenance. Conversely, conditions may result in large accumulations of woody debris that reduce spillway capacity (-25% may be observed) and raise safety concerns.

A systematic laboratory study conducted by Pfister et al. (2013b) to evaluate the interaction between various piano key weir geometries and woody debris types and sizes indicated that floating debris blockage probability is highly influenced by trunk diameter and upstream head. The effects of debris accumulation on the upstream head varied with the value of the debris-free reference upstream head condition. At lower upstream reference head values, the cumulative debris tests indicated a relative increase of the debris-associated upstream head of approximately 70%; higher upstream reference head values produced upstream head increases limited to approximately 20%.

7. AERATION AND ENERGY DISSIPATION

The observed downstream nappes on existing PKW prototypes, mostly equipped with aeration pipes below the inlets apex, are usually well-aerated (Figures 6.7 and 6.8) and not subjected to clinging or vibration, even for very low heads. However, no measurement of air entrainment in aeration pipes has ever been done on prototypes or adequately sized hydraulic models. On Malarce spillway (France), in 2014 EDF equipped the prototype aeration pipes with air entrainment measurement devices. Data shall be published in the forthcoming years (Pinchard et al., 2013).

Figure 6.7 Flow over the Malarce dam PKW (France) with an upstream head of a few cm (Photo courtesy of EDF)

Figure 6.8 Flow over the Escouloubre PKW (France) with an upstream head of a few cm (Photo courtesy of ULg-HECE)

The possible high specific discharge downstream of a PKW implies that great care has to be provided to the downstream energy dissipation structure design, in particular when the weir is located on the crest of a high dam.

The energy dissipation solutions already designed and build downstream of PKW prototypes cannot be generalized and have all been found in an innovative way using physical modelling (Figures 6.9 and 6.10) (Bieri et al., 2011; Erpicum et al., 2011b; Leite Ribeiro et al., 2011).

Figure 6.9 Low slope stepped spillway channel downstream of the Gloriette dam PKW (Photo courtesy of EDF)

Figure 6.10 Smoth spillway and inclined flip bucket downstream of the Saint Marc dam PKW (Photo courtesy of EDF)

Laboratory tests have been performed with PKW upstream of stepped spillways (Ho Ta Khanh et al., 2011b; Silvestri et al., 2013a & b). They showed that the flow downstream of a PKW is always well aerated, i.e., that the inception point is located immediately at the weir toe (Figure 6.11).

In the scope of the Gloriette dam project, Bieri et al. (2011) showed that PKW combined with stepped chutes may lead to pronounced downstream energy dissipation.

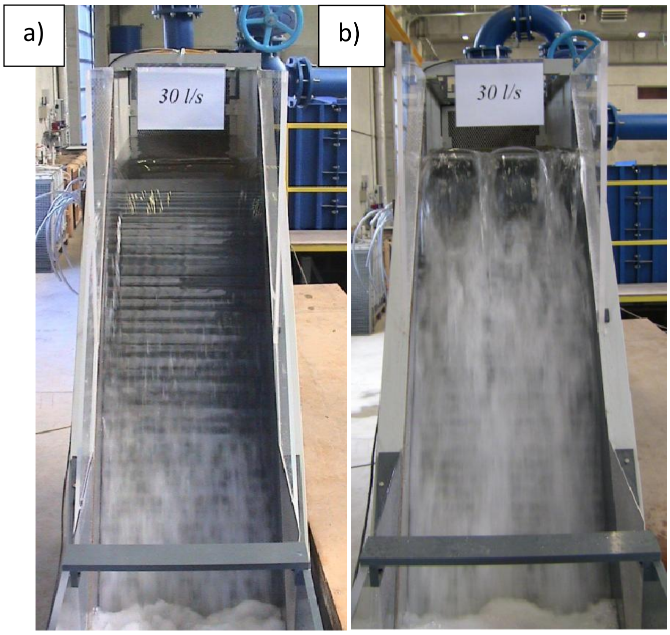

Figure 6.11 Modification in the inception point location along a stepped spillway with a) a standard ogee-crested weir and b) a PKW (q = 0.06 m²/s) (Adpated from Silvestri et al., 2013a)

8. HYDRAULIC DESIGN EQUATIONS FOR A-TYPE PIANO KEY WEIRS AND DESIGN METHOD

Hydrocoop proposed a very simple preliminary method to specify the discharge per unit width q under an upstream head H for an A-type PKW with a maximum height of the labyrinth walls equal to P_m (Lempérière, 2009). This formula (Eqn. 5.8) applies only in the range of parameters specified by the author.

$$q = 4.3\, H\, P_m^{0.5} \tag{5.8}$$

In the meantime comprehensive and systematic model test series have been conducted in several laboratories (Schleiss, 2011). Based on such tests series, three general hydraulic capacity equations for A-type PKWs have been proposed and validated (Kabiri-Samani and Javaheri, 2012; Leite Ribeiro et al., 2012a; Machiel et al., 2014). It is important to notice that these experimentally obtained equations shall be used only within the limits of the parameter range specified by their author, as clearly demonstrated by Pfister et al. (2012). It's worth noticing that these limitations should strictly be respected as small discrepancies may result in significant changes in the PKW discharge coefficient.

Pfister & Schleiss (2013a) compared these three hydraulic design formula for a hypothetical symmetrical A-type PKW, placed on a Roller Compacted Concrete (RCC) gravity dam with a W = 100 m wide chute spillway on its downstream face, a dam height of P_d = 30 m below the PKW foundation. A design discharge of Q_D = 2500 m^3/s was considered, resulting in a specific discharge of 25 m^2/s as commonly used on stepped spillways.

The PKW geometry had a total streamwise length B = 8.00 m, $P = P_i = P_o$ = 5.00 m as vertical height, T_S = 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 result from this PKW geometry: cycle width $W_u = W_i + W_o + 2T_S = 4.00$ m, number of cycles $N = W/W_u = 25$, developed crest length $L = W + (2NB) = 500$ m, $L/W = 5.00$, $B/P = 1.60$, $W_i/W_o = 1.20$, $B_i/B = B_o/B = 0.25$, and $S_i = S_o = 0.83$.

Since the tests of the three hydraulic design formulas presented before were performed with different crest shapes, the results were normalized to a broad-crested weir. The resulting discharge capacity curves are shown in Figure 6.12 within their application limits of *H/P*. As can be seen, the rating curves of the three PKW studies are similar. In general, the empiric equation of Kabiri-Samani and Javaheri (2012) predicts the highest discharge capacity. Additionally, the rating curve of a linear standard crest profile (ogee) is shown in Fig. 5.24, derived from Vischer and Hager (1999) considering a design head of $H_D = 5.00$ m for Q_D.

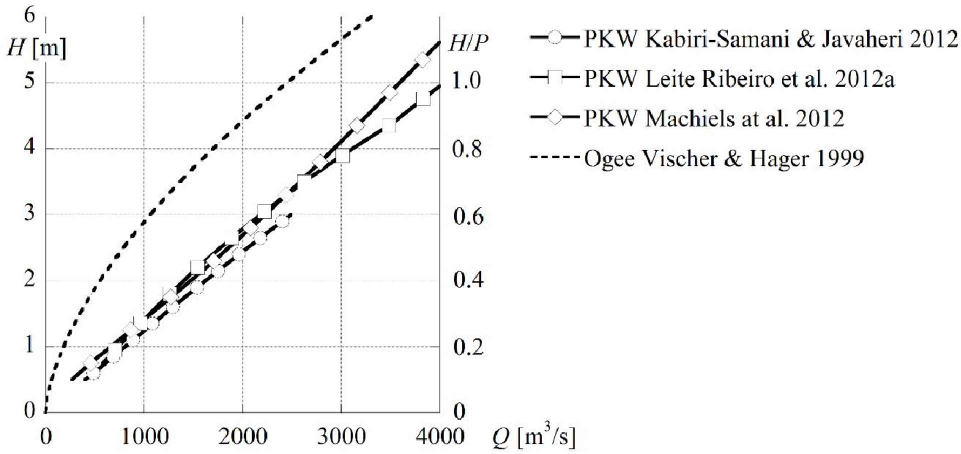

Figure 6.12 Comparisons of the three PKW A-type discharge capacity equations and the rating curve of an ogee crest weir (Pfister and Schleiss, 2013a)

For practical application it may be concluded that the most appropriate PKW hydraulic design formula is dependent upon the application range and the prototype characteristics. The crest shape can serve as a second criterion for the formula selection. If several formulas are within the application range of the prototype, the most conservative result may be considered in order to be on the safe side.

In addition, Machiels et al. (2011b & 2012a) proposed a simple preliminary design method to select a PKW geometry from existing rating curves and specific project constraints (maximum discharge, reservoir level, available width…).

9. PHYSICAL AND NUMERICAL (CFD) MODELLING

The recent development of PKW leads to intensive comparison between 3D numerical modeling and laboratory hydraulic tests on numerous geometries. These works clearly show that the use of (commercial) 3D software is very promising to predict the discharge capacity of a PKW geometry with a good accuracy (+/-10 %) (Pralong et al., 2011b; Lefebvre et al., 2013).

Besides, the freeware WOLF1D PKW (available on http://www.pk-weirs.ulg.ac.be) has been developed by the HECE research group of the University of Liege, on the basis of the large scale physical tests of Machiels (2012). From a PKW geometry, this freeware is able to compute very quickly the flow over half a unit of the weir for a given range of discharge upstream of the structure. The results of the computation are the weir head / discharge curve (+/- 10% accuracy), the water level upstream of the weir depending on the discharge and the distribution of water depth and discharge along both the inlet and the outlet.

Nevertheless, CFD modelling cannot reproduce all the flow characteristics such as instabilities for instance, the effects of complex approach flow conditions or specific abutment geometry as well as the consequences of blocking by floating debris. Furthermore, downstream flow and energy dissipation are critical issues for the overall spillway efficiency which cannot be assessed accurately with numerical models.

Therefore the final geometry of a PKW together with the energy dissipation system should be tested on a physical model, which is the only way to reproduce properly the complete flow features with the required level of precision. It is worthwhile mentioning that all large PKW prototypes have been validated and optimized with physical modelling.

10. STRUCTURAL AND CONSTRUCTION ISSUES

Structural issues highly depend upon the PKW geometry (size and number of overhangs), load cases (ice influence or site seismicity for example), respective unit rates and project features (location at a gravity dam crest or isolated on a bank...).

Existing references have highlighted some specific ratios, such as:

- 50 to 100 kg reinforcement /m^3 concrete
- 60 to 130 m^2 form / m of width

It is common practice to add concrete or anchors in order to reach the required stability factors of safety. Thermal load cases (one face is under the sun, the other one protected by the water) or ice load may easily become sizing ones.

Particular attention is also given to the construction issues, such as pre fabrication (Figure 6.13) or use of steel instead of concrete as on all the prototypes up to date.

Fig 6.13 Back Esk Reservoir spillway – PKW made of prefabricated concrete elements (Courtesy of Black and Veatch Ltd)

11. ENGINEERING

So far, PKW have been primarily been designed and constructed as flood-routing spillways in the following three configurations:

- Narrow valley where only a small place is available for installing a new structure;
- Gravity dam, where the PKW - easily implemented on the crest of the dam thanks to its small footprint - allows maximizing the active storage capacity of the reservoir for a given Maximum Water Level. Generally, PKW is combined with surface or bottom outlet gates for the fine tuning of the upstream water level regulation and the flushing of the reservoir;
- Long barrage (gated structure dam) in flat areas. In such a case, the PKW (that may be associated with gates, whose number is then smaller than in a traditional alternative), minimizes the inundated areas during floods by lowering the Maximum Water Level compared to a solution with a linear weir.

The Van Phong barrage in Vietnam, 475 m long and 7 m high on the river bed, with a 15,400 m^3/s design flood, is a good example of the third configuration (Figure 6.14). The initial design included 28 radial gates with 15 m wide and 7.5 m high. The final design, for the same Maximum Water Level, includes only 10 radial gates (and 8 would have been probably enough) in the central part and 60 PKW units with a total length of 302 m on each side. Compared to a labyrinth weir solution, the PKW offer a smaller footprint on the rather deep bedrock, which requires less excavation and concrete volume. The main advantages of the final alternative are: the investment and maintenance cost savings, a higher safety level in operation and a better integration in the environment. Taking into account the recent non-functioning of several gates during floods, many Vietnamese engineers think that a safe solution is now to combine, as much as possible, gates and free overflow crest spillway. In this scope, the PKW is often a good alternative, particularly when the available crest length be limited.

Figure 6.14 The Van Phong barrage in Vietnam (Courtesy of M. Ho Ta Khanh)

A new project with such a configuration is now under design for the Xuân Minh barrage.

12. ONGOING RESEARCH

Ongoing research and developments are orientated towards new developments of the PKW geometry (side wall angle narrowing the inlet key and widening the outlet one; new hybrid configurations using rectangular labyrinth and PKW) and the analysis of downstream flow feature (aeration and energy dissipation).

Research also concerns structural and construction aspects such as the use of steel or combined steel/concrete structures as well as pre-fabrication.

13. PROTOTYPE PIANO KEY WEIR EXAMPLES

The bibliography included herein contains references for many PKW prototypes. A summary of these structures is presented in Table 1.

Table 1: Piano Key Weir Spillways Prototypes

Name	Country	Configuration	Q_{design}	P	H_{design}	Source
			(m^3/s)	(m)	(m)	
Black Esk	Scotland	Bellmouth crest	183	2.10	0.97	Ackers et al. 2013
Campauleil*	France	Dam crest	120	5.35	0.90	Laugier et al. 2013
Charmines	France	Dam crest	300	4.38	1.00	Laugier et al. 2013

Dak Mi 4B	Vietnam	Dam crest	500	3.75	2.00	Ho Ta Khanh et al. 2014
Dak Mi 4C	Vietnam	Dam crest	200	2.50	1.00	Ho Ta Khanh et al. 2014
Dak Rong 3	Vietnam	Dam crest	6550	5.00	3.50	Ho Ta Khanh et al. 2014
Emmenau	Switzerland	River	4.35	1.20	0.45	Eichenberger, 2013
Escouloubre	France	Bank	13	1.80	0.65	Laugier et al. 2013
Etroit	France	Dam crest	82	5.30	0.95	Laugier et al. 2013
Gage*	France	Right bank	455	6.00	1.75	Laugier et al. 2013
Giritale	Sri Lanka	River		2.40	0.45	Jayatillake and Perera. 2013
Gloriettes	France	Right bank	90	3.00	0.80	Laugier et al. 2013
Goulours	France	Right bank	68	3.10	0.95	Laugier et al. 2013
Hazelmere*	South Africa	Dam crest	4300	9.00	3.23	Botha et al. 2031
Loombah	Australia	Bank				
Malarce	France	Dam crest	570	4.40	1.50	Laugier et al. 2013
Raviège*	France	Dam crest	300	4.67	1.40	Laugier et al. 2013
Saint Marc	France	Dam crest	138	4.20	1.35	Laugier et al. 2013
Van Phong	Vietnam	Barrage (River)	8750	5.00	5.20	Ho Ta Khanh et al. 2014

* not yet constructed

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