Workshop on Nonlinear Weirs: From theory to Practice

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François Lempérière, Hydrocoop, France
Prof. **Ahmed Ouamane**, Biskra University, Algeria
Greg Paxson, Schnabel Eng., USA
Prof. **Blake P. Tullis**, UWRL, Utah State University, USA
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Workshop overview

- Introduction and Overview
- Hydraulics of Labyrinth and PK Weirs
- **Q/A – 5’**
- Design Overview
- **Q/A – Coffee break - 30’**
- Vietnamese experience
- Algerian experience
- **Q/A – 5’**
- French experience
- US experience
- Conclusions and perspectives
- **Q/A – As much as needed!**

1. Introduction

Types of non-linear weirs

Radial Weirs

Box-Inlet Drop Spillways

Fuse Gates

Labyrinth Weirs

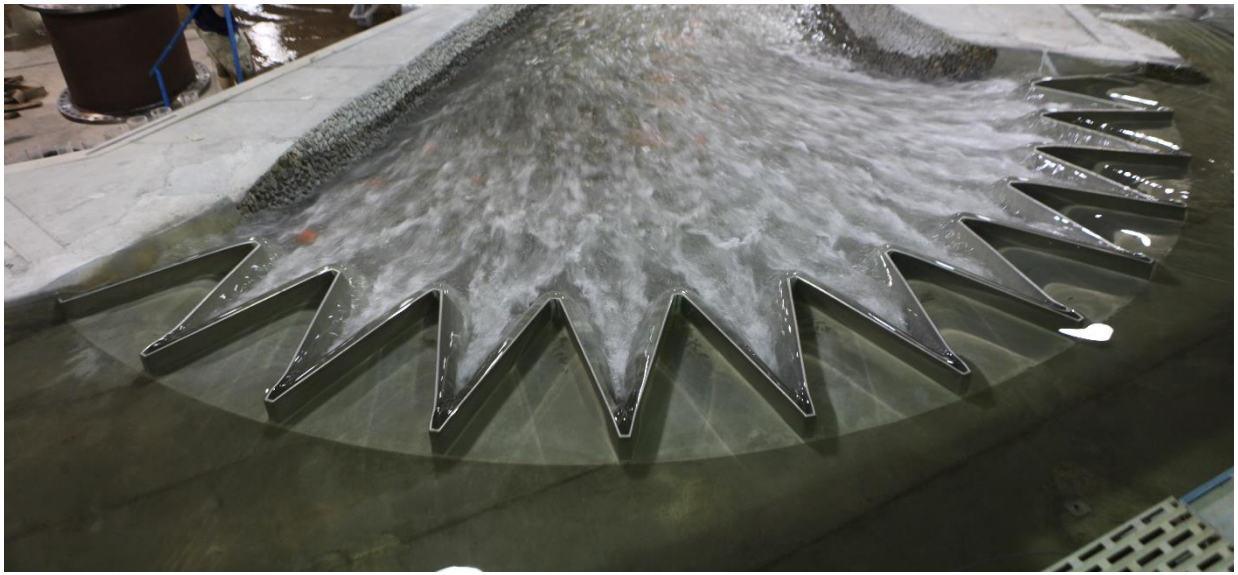
PK Weirs

1. Introduction

Labyrinth Weir

- Increase spillway crest length by ‘folding’ the crest in plan
- Reduced footprint – flexible geometry, inherently stable
- High hydraulic capacity

Linville Lake Dam – USA
Designed for 693 m³/s
with a head of 2.8 m
(Courtesy of Schnabel)



Lake Isabella Dam Physical Model Study (Courtesy of UWRL / USACE)

1. Introduction

Piano Key weir (PKW) = recent evolution of traditional labyrinth weir (Lempérière & Ouamane, 2003)

- Decrease of sidewalls height
- Reduced footprint → can be placed on the crest of gravity dams
- High hydraulic capacity

Malarce dam PKW – France
Designed for 13.8 m³/s/m with a head of 1.50m (Courtesy of EDF-CIH)

Van Phong dam – Vietnam – Gates + PKW – 320m long - 8,700m³/s (Courtesy of VNCOLD)

1. Introduction

Piano Key weir (PKW) = an effective solution to increase the discharge capacity of existing dams in limited space environment

Upgrade of the gated Raviège dam spillway with a PKW - France

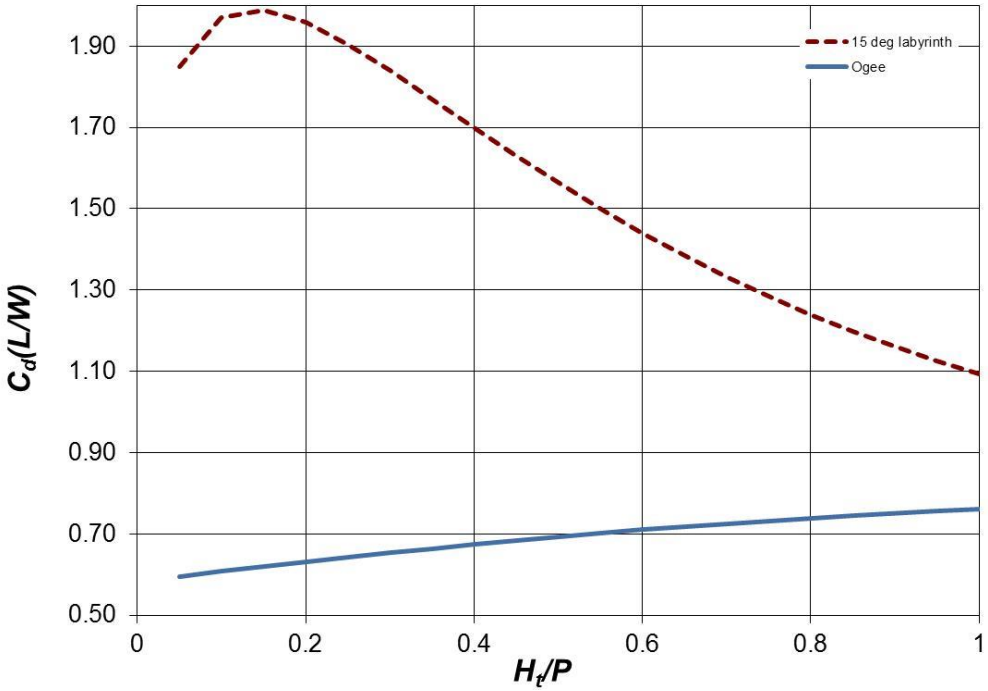
1. Introduction

Labyrinth and Piano Key Weirs

↗ Discharging crest length
Discharge for a given spillway width

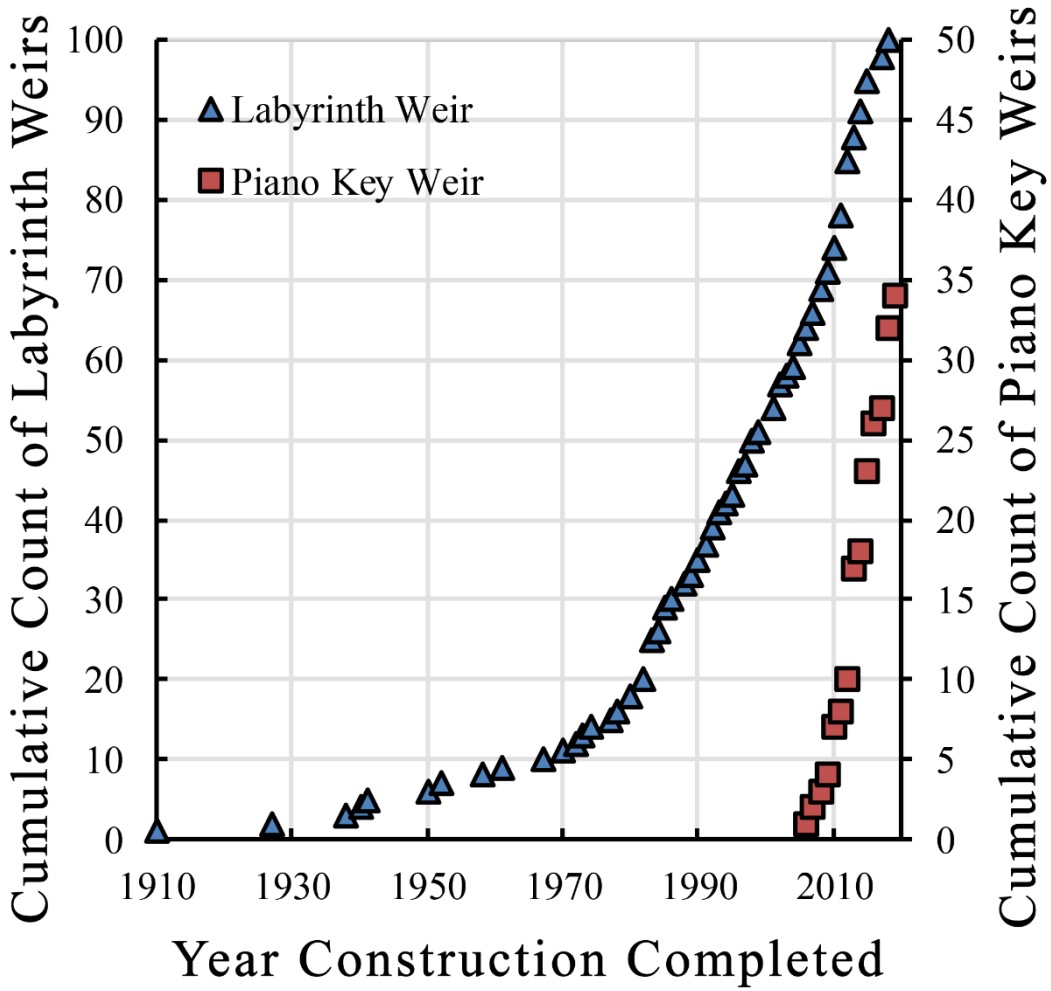
BUT

↘ Discharge capacity
with increasing head



1. Introduction

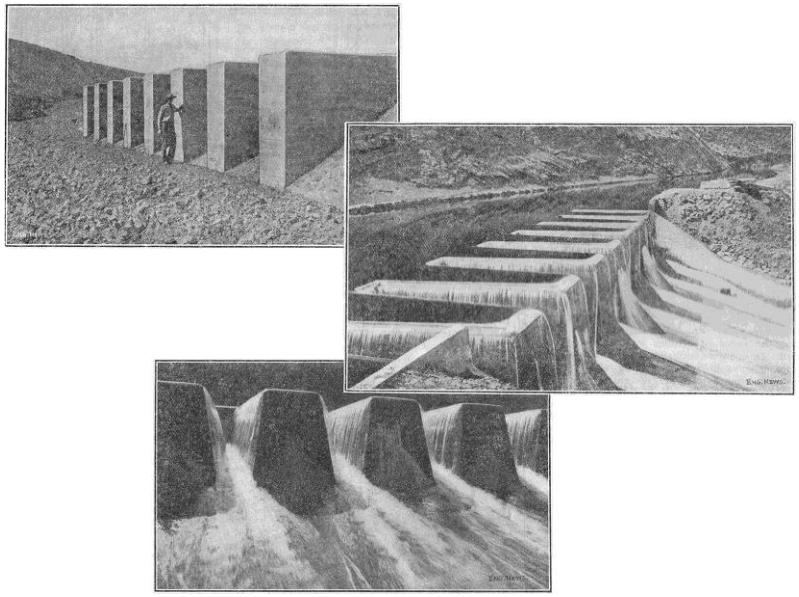
Labyrinth and Piano Key Weirs development

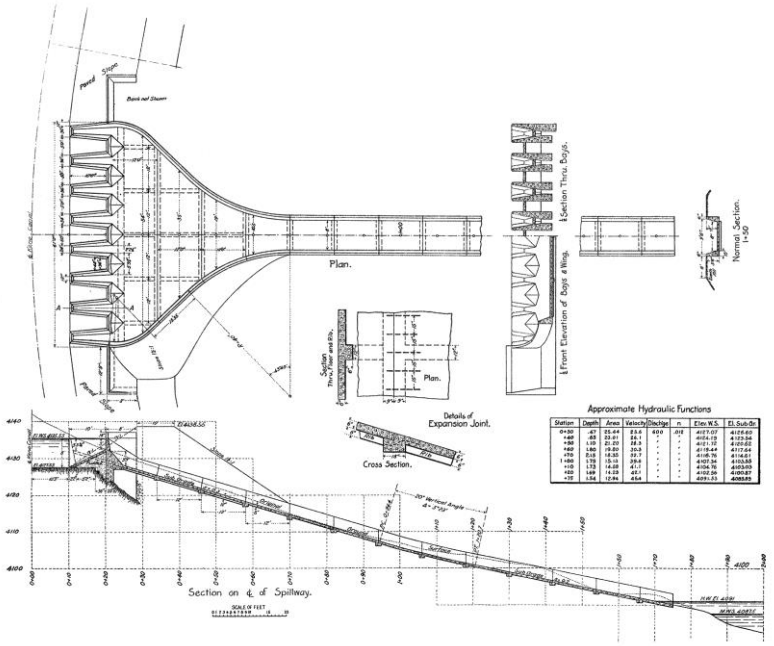


Crookston et al., 2019

2. Labyrinth research timeline + literature

History of Labyrinth Weirs

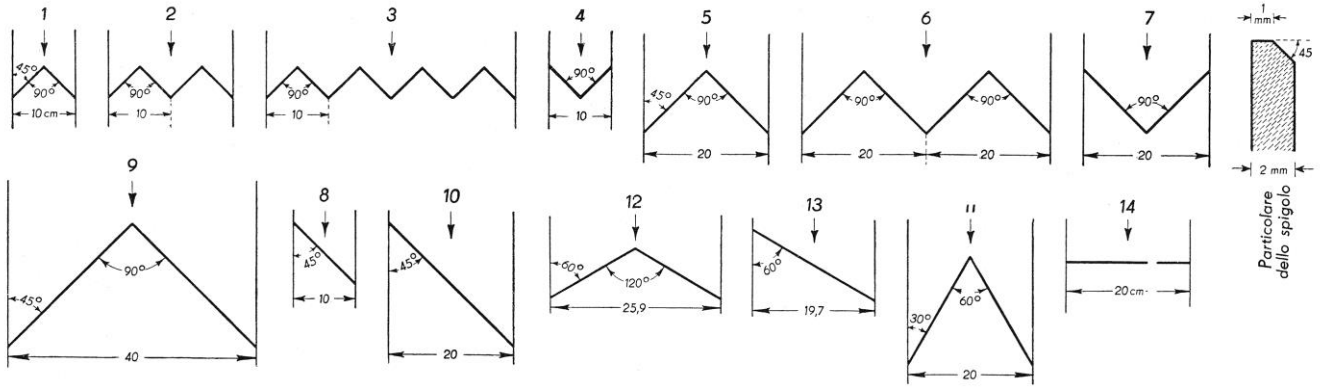


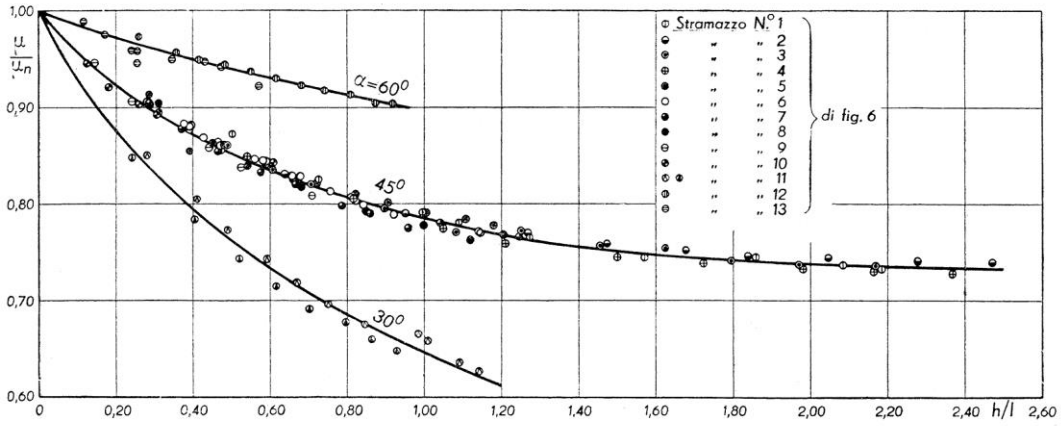


2. Labyrinth research timeline + literature

History of Labyrinth Weirs

- The first Labyrinth Weir hydraulic study was conducted by the Italian Bruno Gentilini (Gentilini, 1941) at Politecnico di Milano (Polytechnic School, Milan) under Giulio De Marchi.





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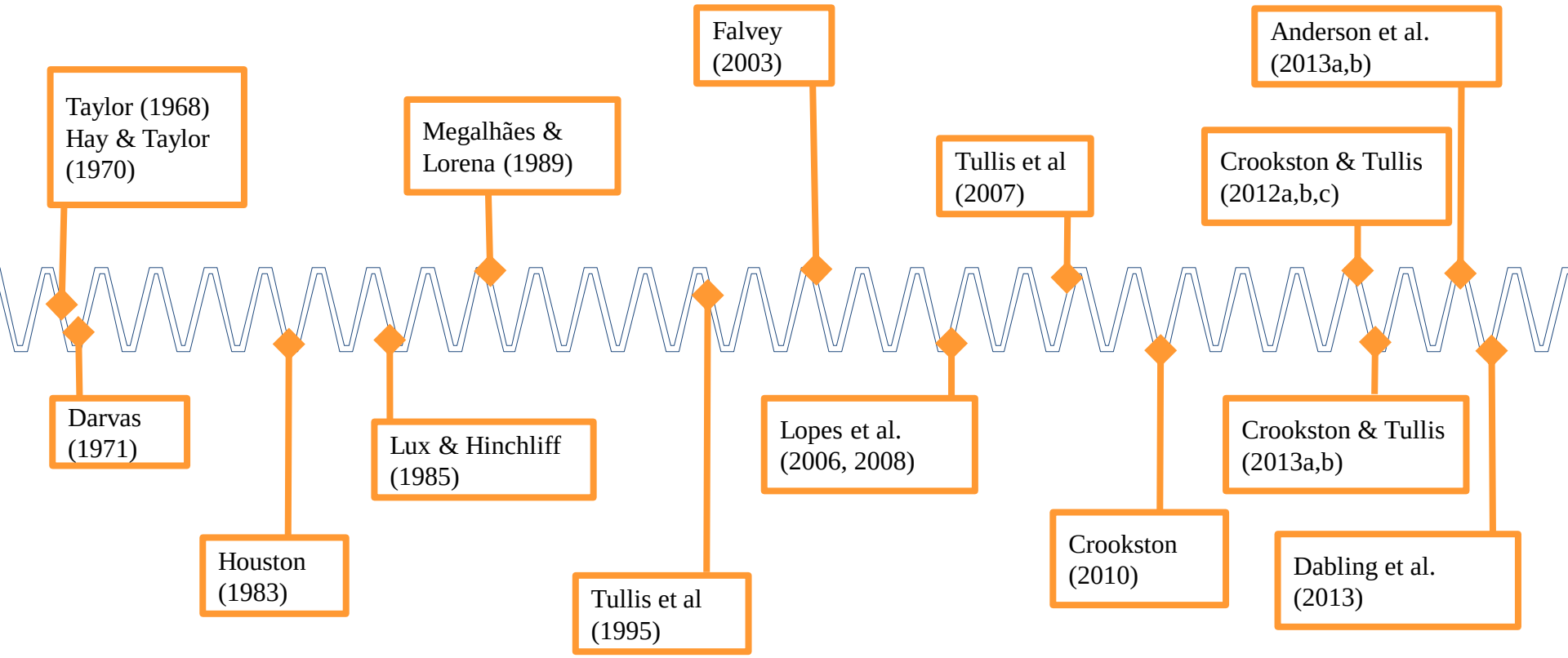

3. Labyrinth prototype examples

Constructed Labyrinth Spillways

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Constructed Labyrinth Spillways

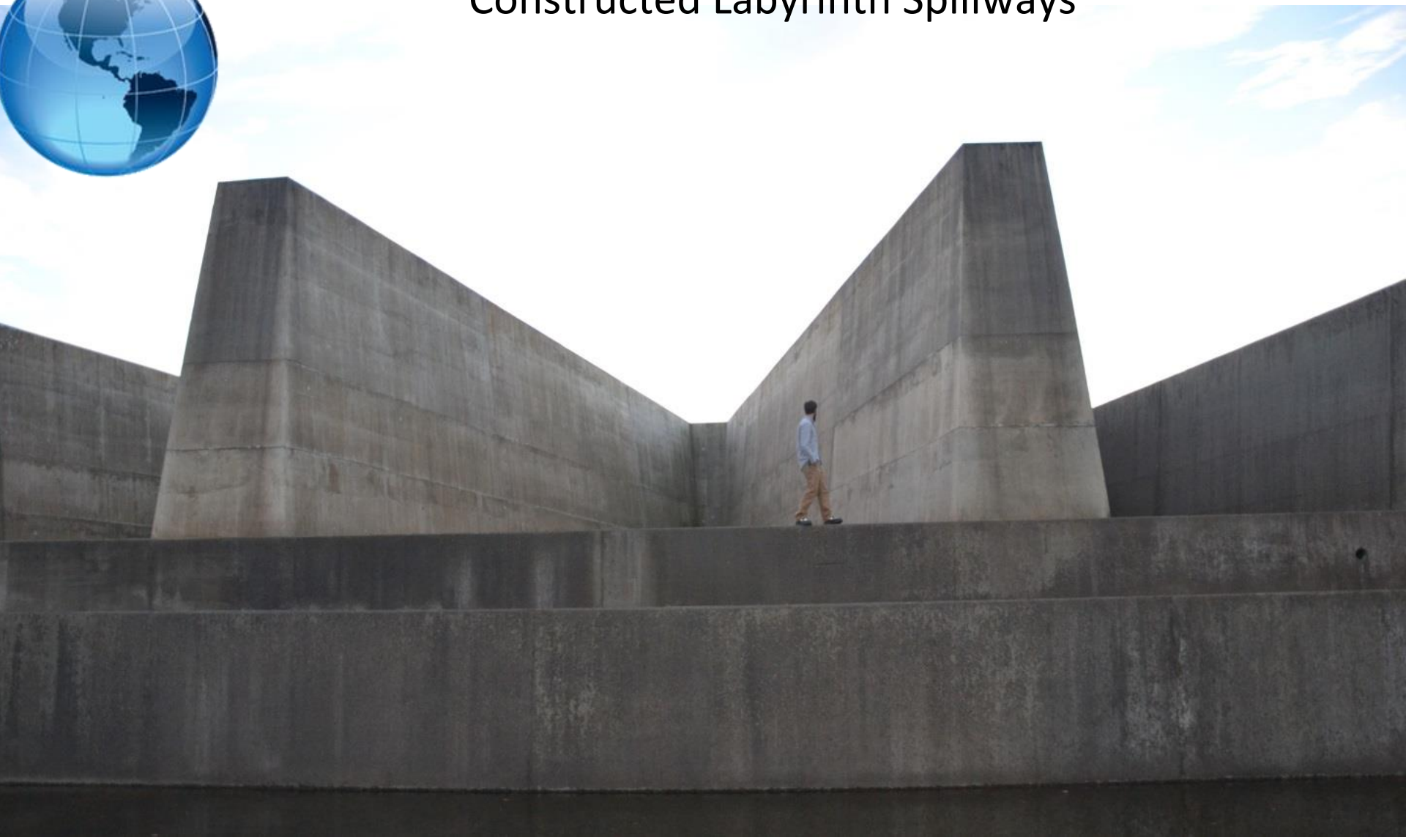
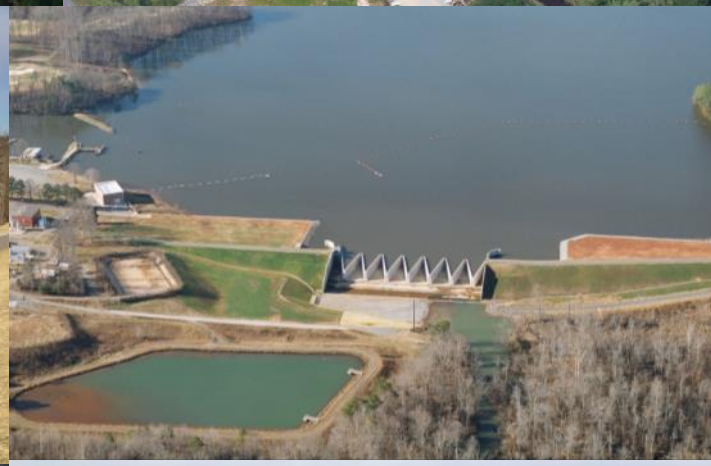
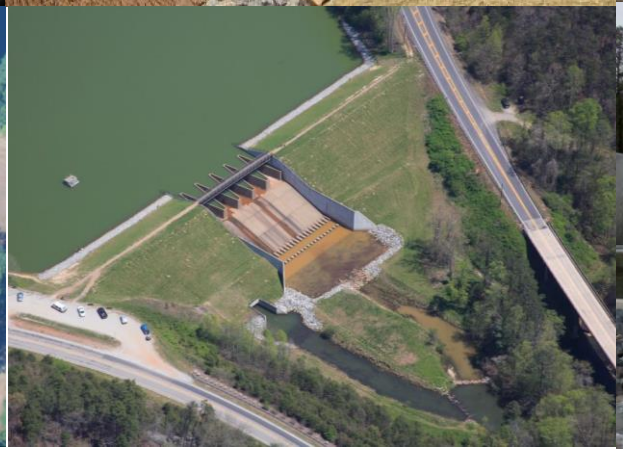


Photo courtesy Freese & Nichols

Photo courtesy Aurecon (Dawid Van Wyk)





4. PKW research timeline + literature

1999-2002 - Early works by Hydrocoop in collaboration with Electricité de France (France), Biskra University (Algeria) and Roorkee University (India) (Lempérière & Ouamane, 2003)

2003 - First publication about PKW by Lempérière and Ouamane

2006 - First prototype PKW (Goulours dam – EDF – France) (Laugier, 2007)

From 2006, massive research works and publications

2011 - 1st Int. Work. on Labyrinth and Piano Key Weirs (Liège, Belgium)

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2020 – 30+ PKWs in operation or under construction all over the world
> 20 journal papers; 3 books and > 100 conference papers

5. PKW Prototype examples – PKW main advantages

⇒ **Freeflow** spillway : **Reliability** during floods

- Almost no risk of operation malfunctionning
- No mechanical equipment
- No maintainance
- No need for human operator
- Normal time : Self Control of reservoir level

⇒ Freeflow spillway - Self Control of reservoir level out of flood operations

⇒ Freeflow spillway – Efficient against debris

⇒ Hydraulics efficiency :

3 to 4 times more efficient than Ogee crest at small head

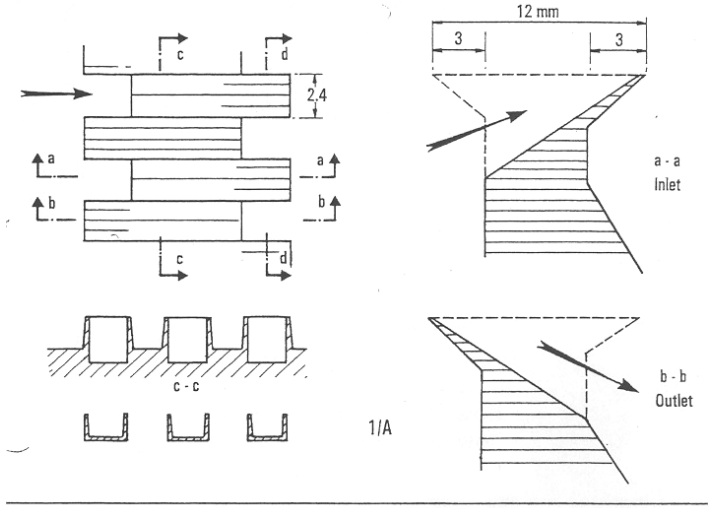
5. PKW Prototype examples – PKW main advantages

⇒ **Reduced footprint**

- Can be installed on the top of most concrete dams
- Gravity AND arch dams

⇒ **Easy** and **cheap** to design and built

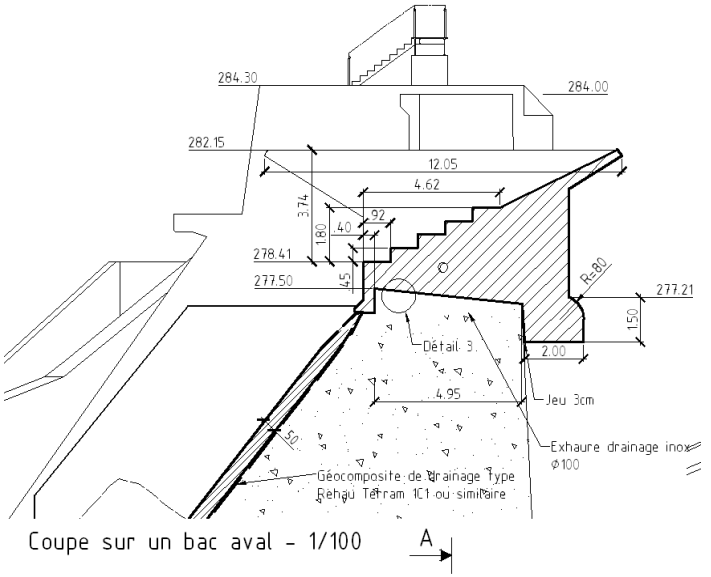
- Triangles and rectangles !
- Project **cost** under control
- No maintainance cost
- No operation cost
- Cheaper than gates



5. PKW Prototype examples – PKW main advantages

⇒ **Structural reliability**

- Hyperstatic structures
  - ✓ Reinforcement ok
- Self-stable structures
  - ✓ Few anchors required for external stabilty



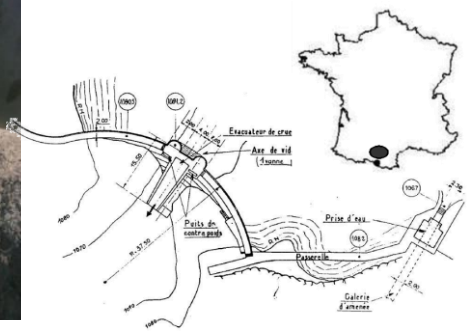
St Marc dam project

-40 m high concrete gravity dam
-Rehabilitation works in 2008
-Adaption to Swelling concrete
=> NO ANCHOR
-Design head : 1.35 m
-Design flow rate : 140 m^3/s
-Total cost : 1.8 M€
-PKW : 30% of the total cost (important energy dissipater)

5. PKW Prototype examples – First PKW at EDF (and worldwide) :

⇒ **Goulours (France, Pyrénées)**

- Design phase : 2003 – 2005
- Construction : 2006



Goulours dam project

- 20 m high concrete gravity arch dam
- Rehabilitation works in 2006
- Design head : 0,95 m
- Design flow rate : 70 m^3/s
- Total cost : **0,4 M€**
- PKW : 30% of the total cost

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Goulours dam project

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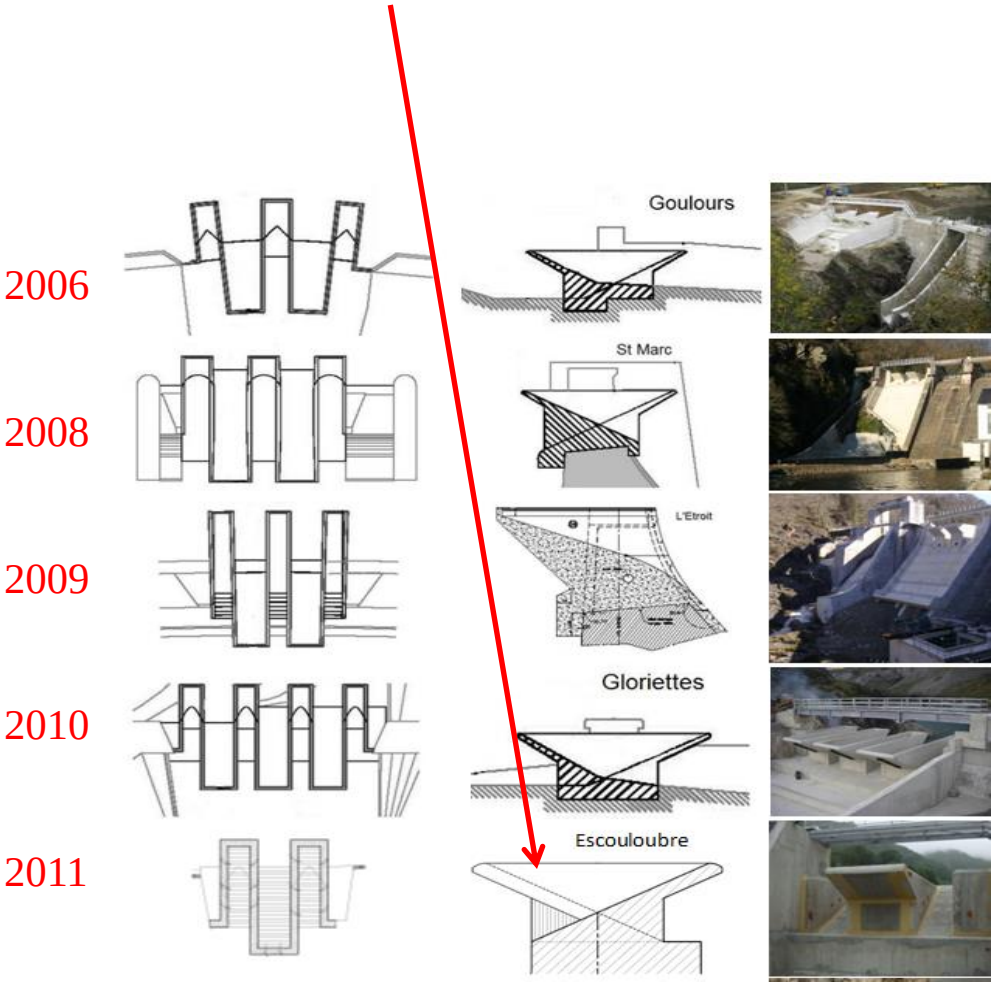
- Total cost : **0,4 M€**
- PKW : 30% of the total cost

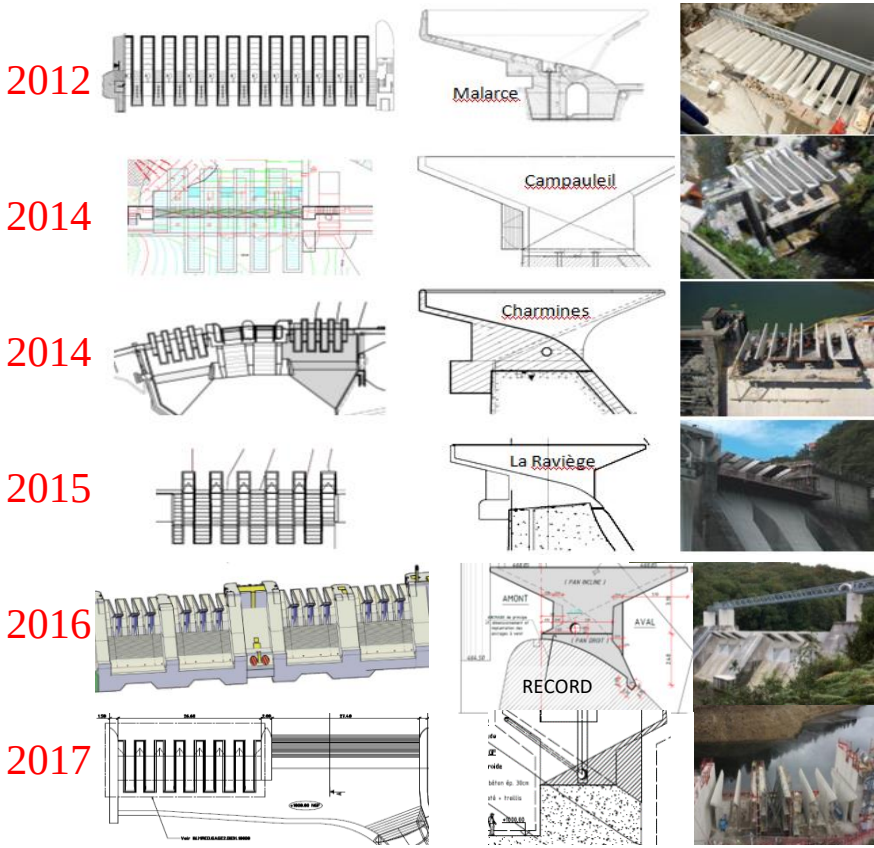
5. PKW Prototype examples – Why PKW at EDF ?

After Goulours => 10 other PKW designed and built by EDF from 2006 to 2018 :

⇒ 10 for flood control

⇒ 1 unit for plant operation (balance basin)





5. PKW Prototype examples – Van Phong dam (Vietnam)

New dam

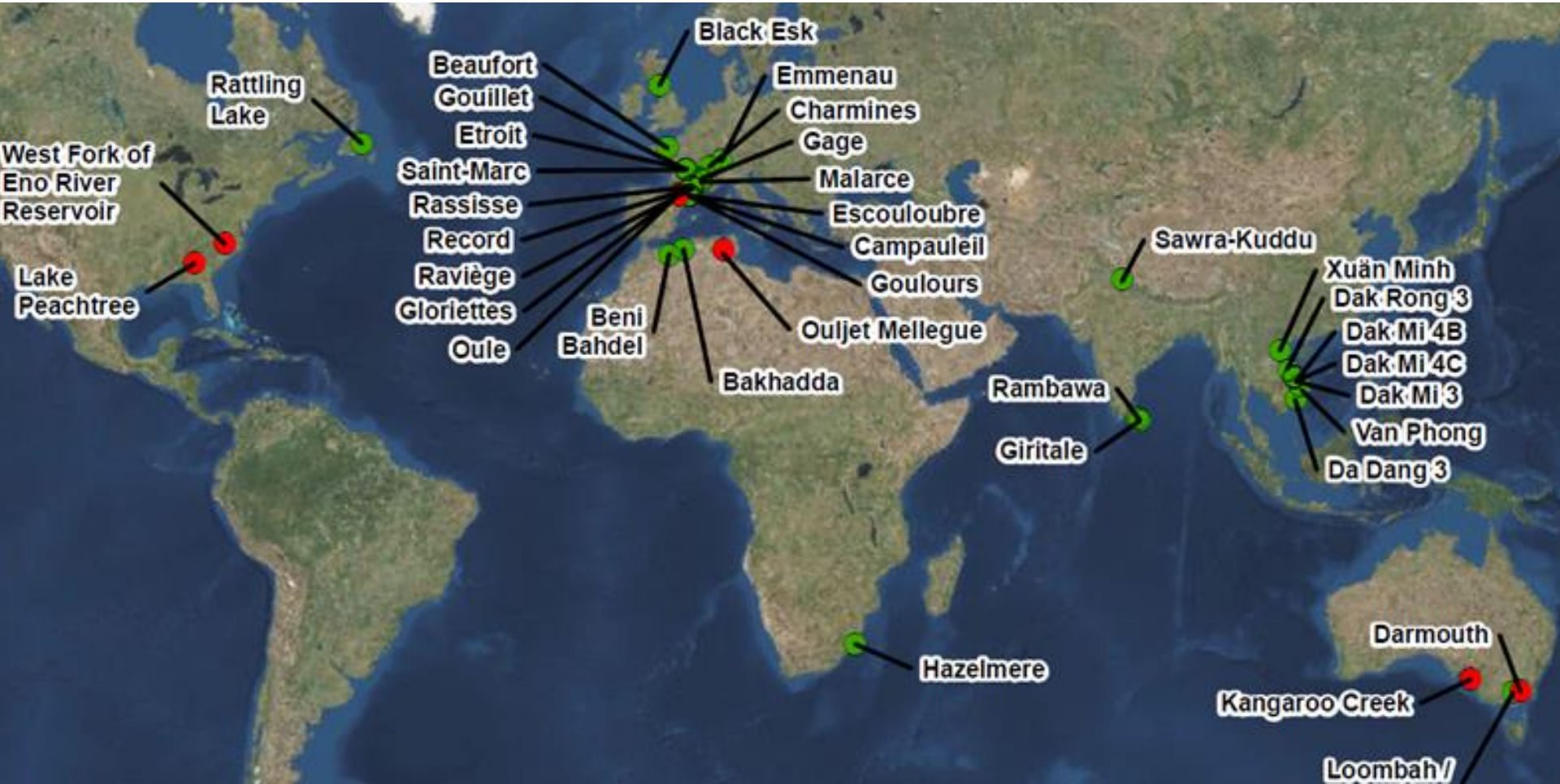
Run-off river scheme

Combination of gates and PKW

Van Phong dam – Vietnam – Gates + PKW – 320m long - 8,700m³/s (Courtesy of VNCOLD)

5. PKW Prototype examples – PKW World Register

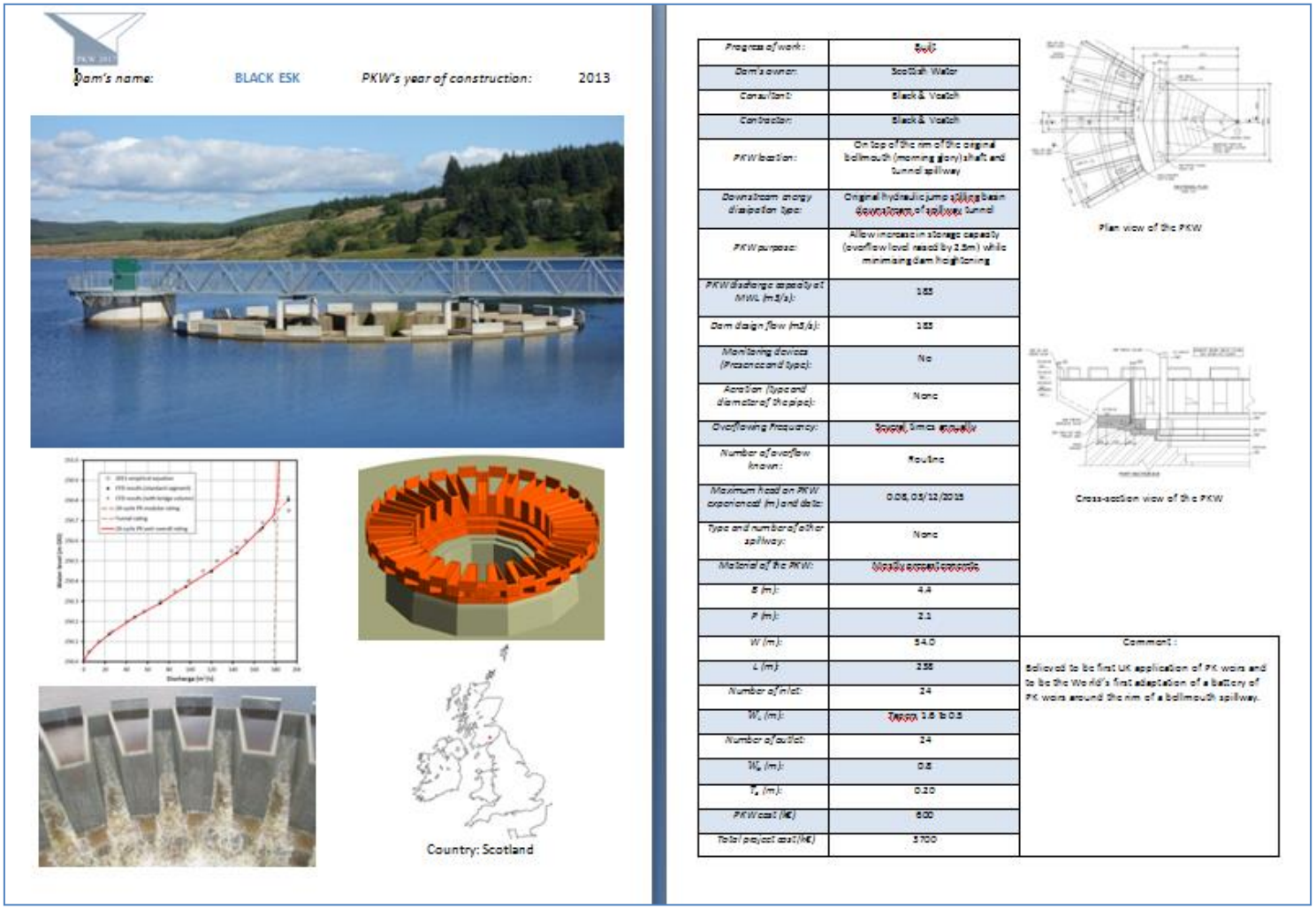
- 1st issue for PKW 2017 in Vietnam
- Published in Hydropower and Dams, issue 6 – 2017
- https://www.uee.uliege.be/cms/c_5026433/en/world-register-of-piano-key-weirs-prototypes



5. PKW Prototype examples – PKW World Register

- Aim and objectives:

  Gathering together all PKW projects built (mostly) or under development
- Forms send to Owners and Engineers



Dam's name: BLACK ESK PKW's year of construction: 2013

Country: Scotland

Progress of work:	Built
Dam's owner:	Scottish Water
Consultant:	Black & Veatch
Contractor:	Black & Veatch
PKW location:	On top of the rim of the original bellmouth (morning glory) shaft and tunnel spillway
Downstream energy dissipation type:	Original hydraulic jump stilling basin downstream of spillway tunnel
PKW purpose:	Allow increase in storage capacity (overflow level raised by 2.5m) while minimising dam heightening
PKW discharge capacity at MWL (m3/s):	185
Dam design flow (m3/s):	185
Monitoring devices (Presence and type):	No
Aeration (type and diameter of the pipe):	None
Overflowing frequency:	Several times annually
Number of overflow known:	Routine
Maximum head on PKW experienced (m) and date:	0.08, 05/12/2015
Type and number of other spillway:	None
Material of the PKW:	Visually exposed concrete
B (m):	4.4
P (m):	2.1
W (m):	54.0
L (m):	258
Number of inlet:	24
W_i (m):	Tapers 3.8 to 0.5
Number of outlet:	24
W_o (m):	0.5
T_o (m):	0.20
PKW cost (M€)	600
Total project cost (M€)	5700

Plan view of the PKW

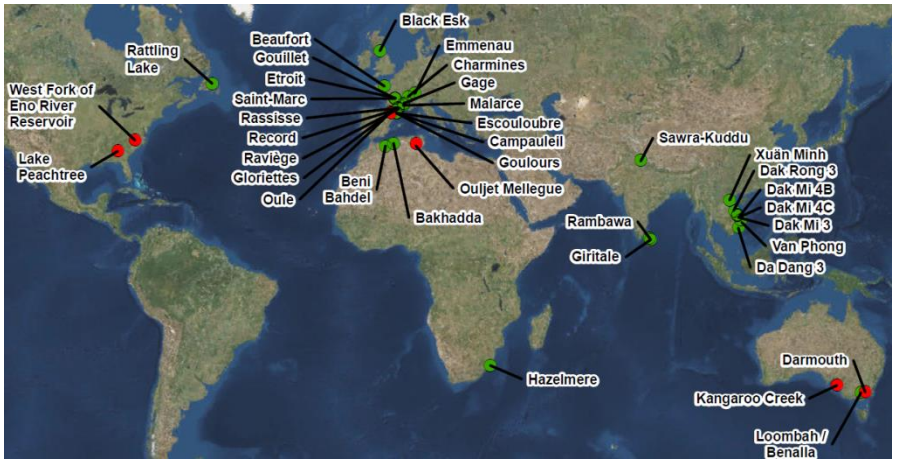
Cross-section view of the PKW

Comment :

Believed to be first UK application of PK weirs and to be the World's first adaptation of a battery of PK weirs around the rim of a bellmouth spillway.

5. PKW Prototype examples – PKW World Register

- Overview of PKW projects worldwide



- 27 forms (June, 2020)
- > 30 PKW projects

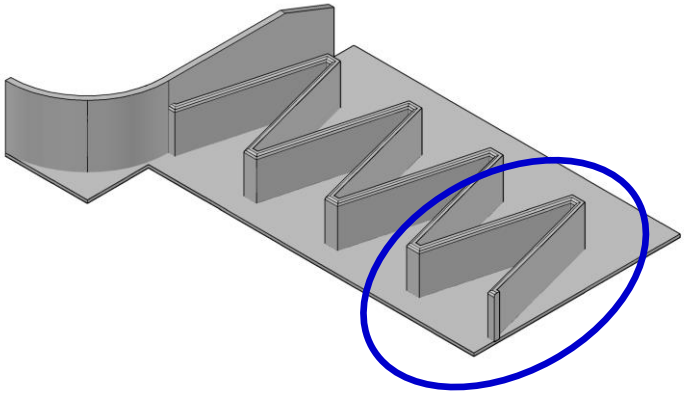
Detailed formsheet avalaible on PKW website :

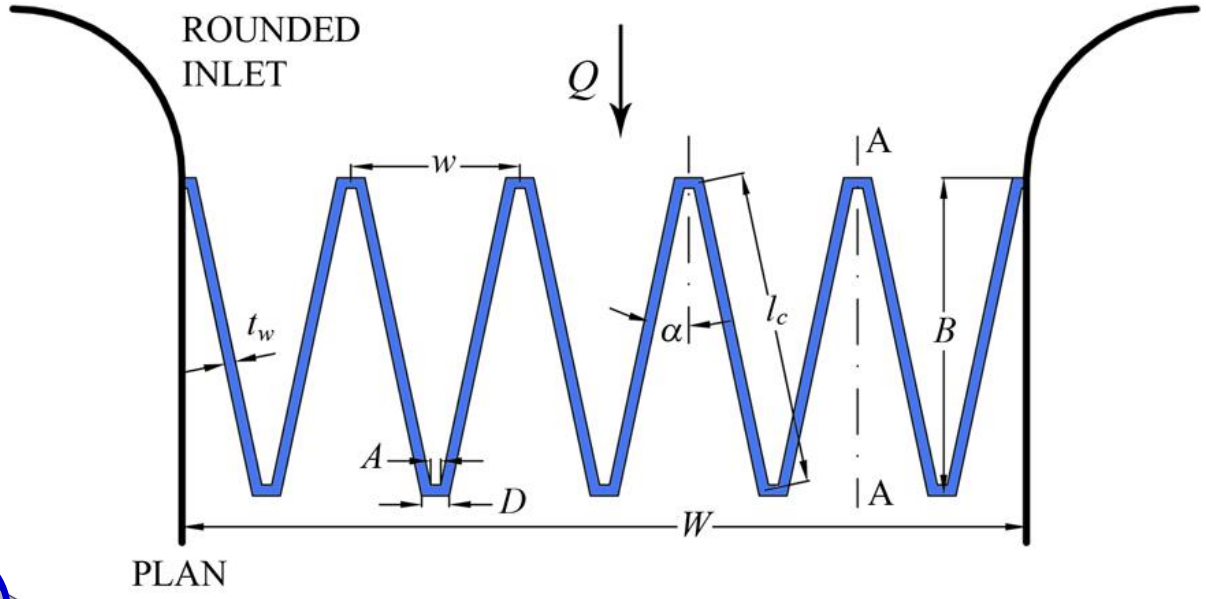
https://www.uee.uliege.be/cms/c_5026433/en/world-register-of-piano-key-weirs-prototypes

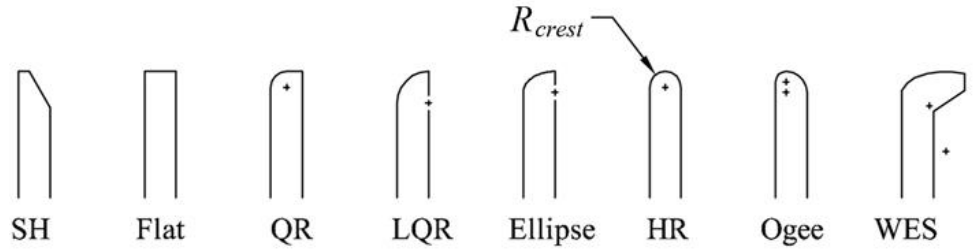
Dam name	Country	PKW date of construction	Comment
Bakkhada	Algeria	1938	No project detailed form. Labyrinth weir. PKW "ancestor" close to type A
Beni Bahdel	Algeria	1940	No project detailed form. Labyrinth weir. PKW ancestor close to type B
Goulours	France	2006	
Saint-Marc	France	2008	
Etroit	France	2009	
Gloriettes	France	2010	
Rattling Lake	Canada	2011	No project detailed form
Escouloubre	France	2011	
Gouillet	France	2011	
Malarce	France	2012	
Beaufort	France	2013	
Black Esk	United Kingdom	2013	
Dak MI 4B	Vietnam	2013	
Dak Rong 3	Vietnam	2013	
Giritale	Sri Lanka	2013	
Loombah	Australia	2013	
Sawra Kuddu	India	2013	
Emma	Switzerland	2013	No project detailed form
Campauleil	France	2014	
Charmines	France	2015	
Rambawa Tank	Sri Lanka	2015	
Rassisse	France	2015	
Raviege	France	2015	
Van Phong	Vietnam	2015	
Da Dang 3	Vietnam	2016	
Dak Mi 3	Vietnam	2016	
Record	France	2016	
Xuan Minh	Vietnam	2016	
Gage	France	2017	
Hazelmere	South Africa	2017	No project detailed form
Oule	France	In progress	
Ouljet Mellegue	Algeria	In progress	

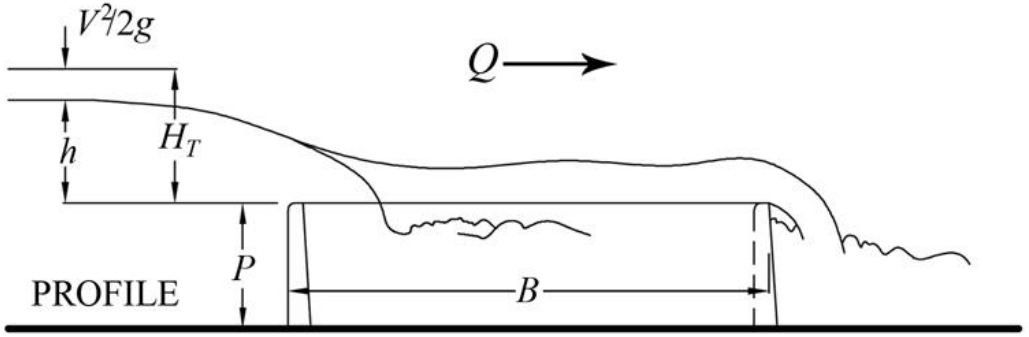
6. Labyrinth and PK Weirs hydraulic design and performance

Labyrinth weir terminology









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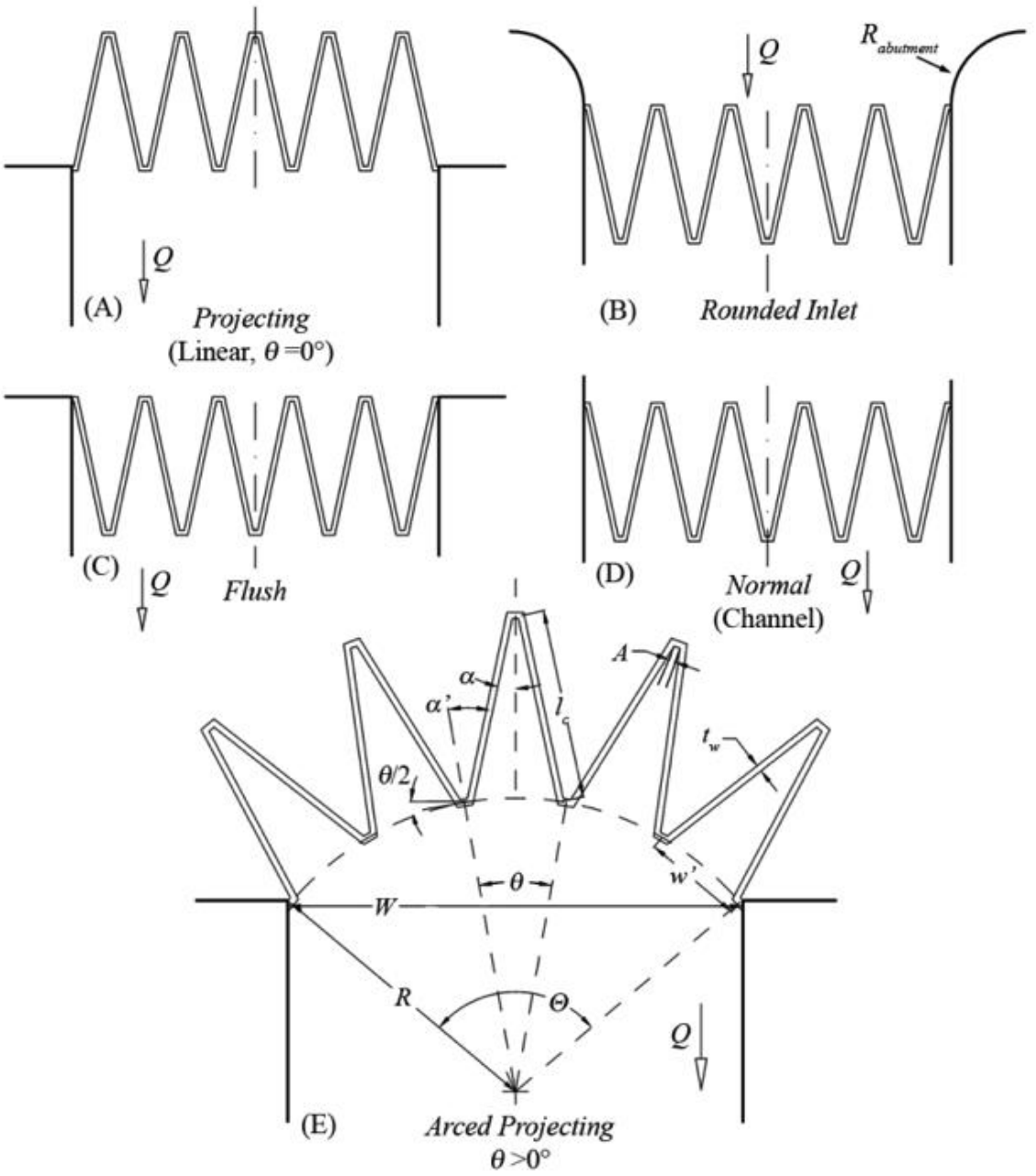
Labyrinth weir terminology



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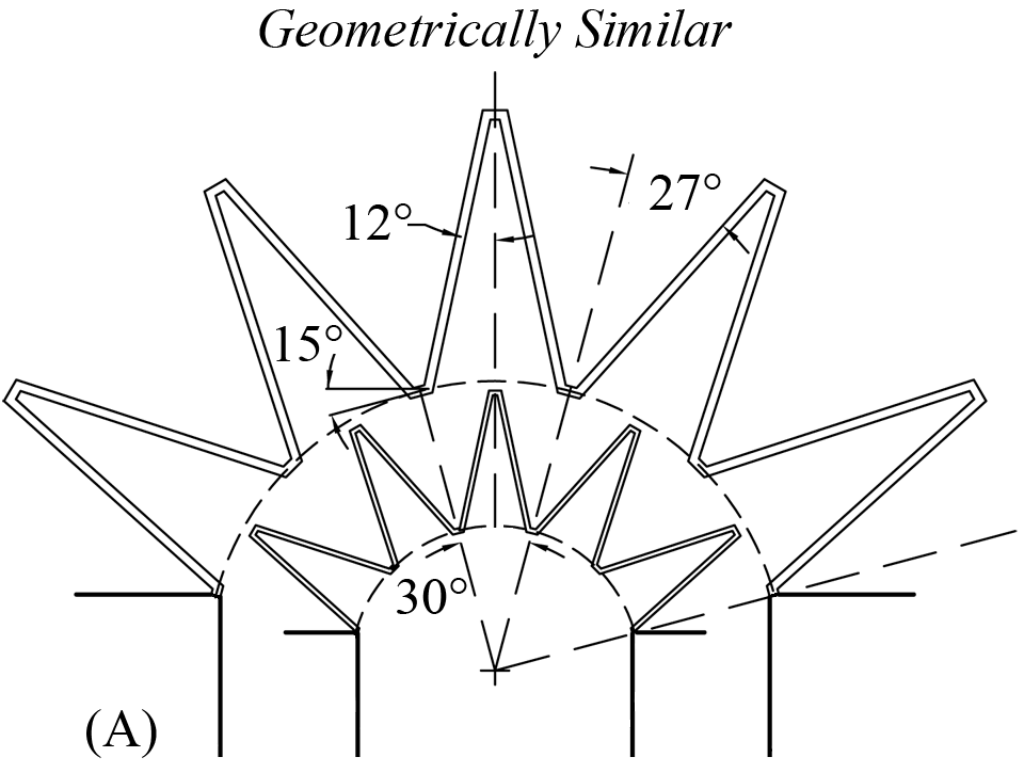

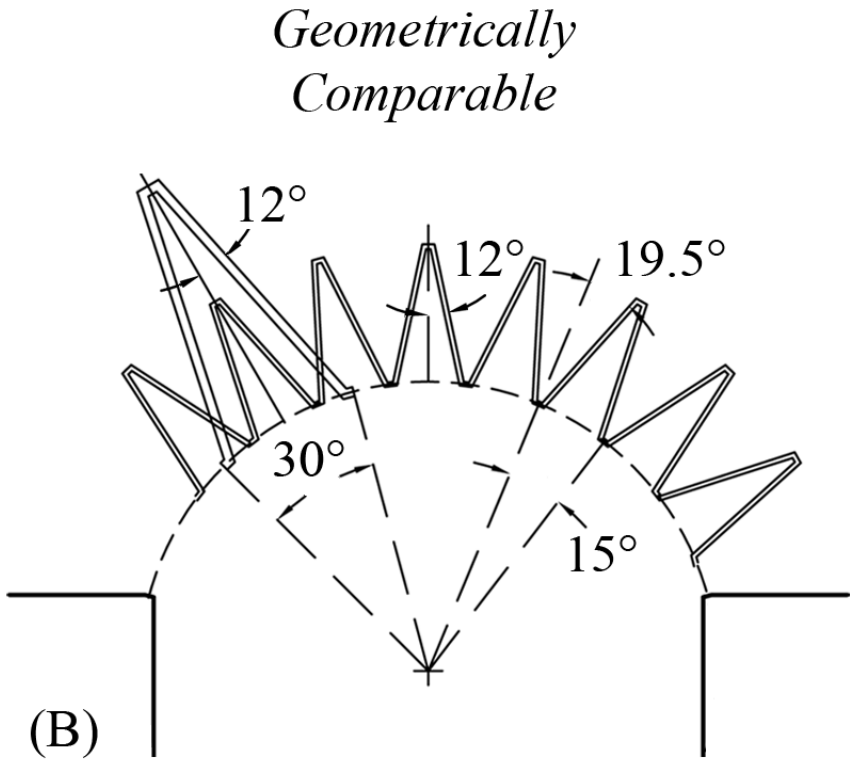

6. Labyrinth and PK Weirs hydraulic design and performance

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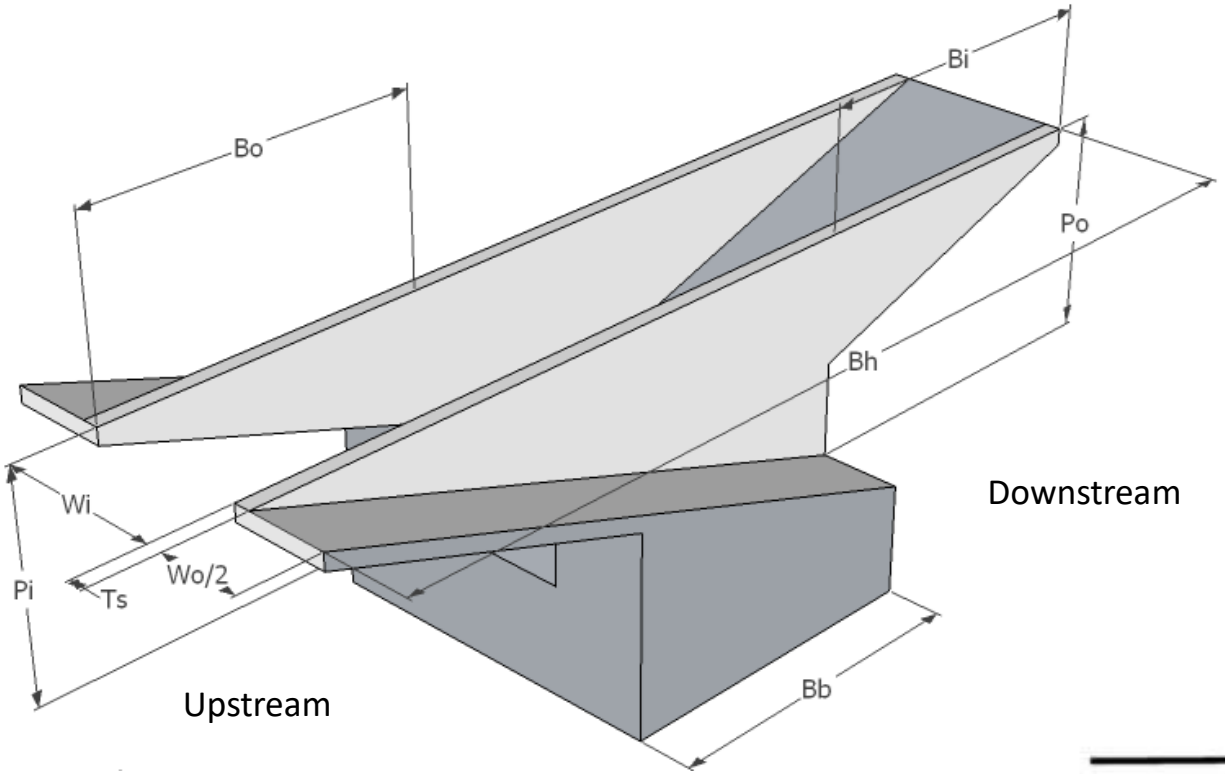
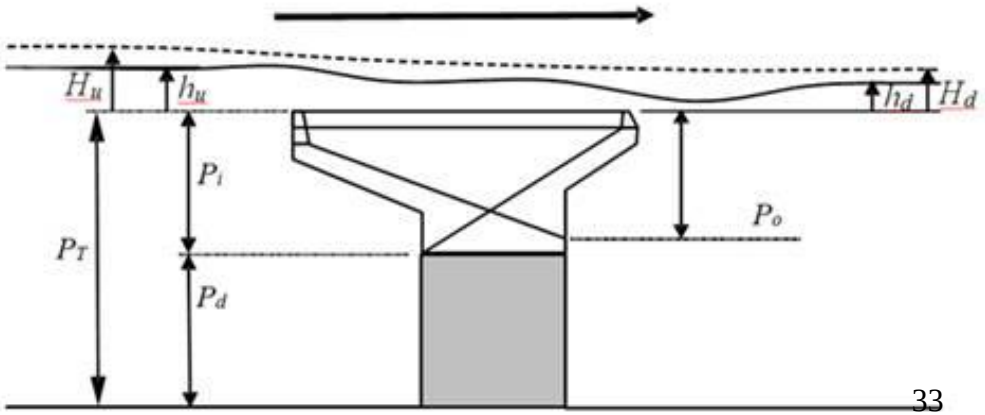
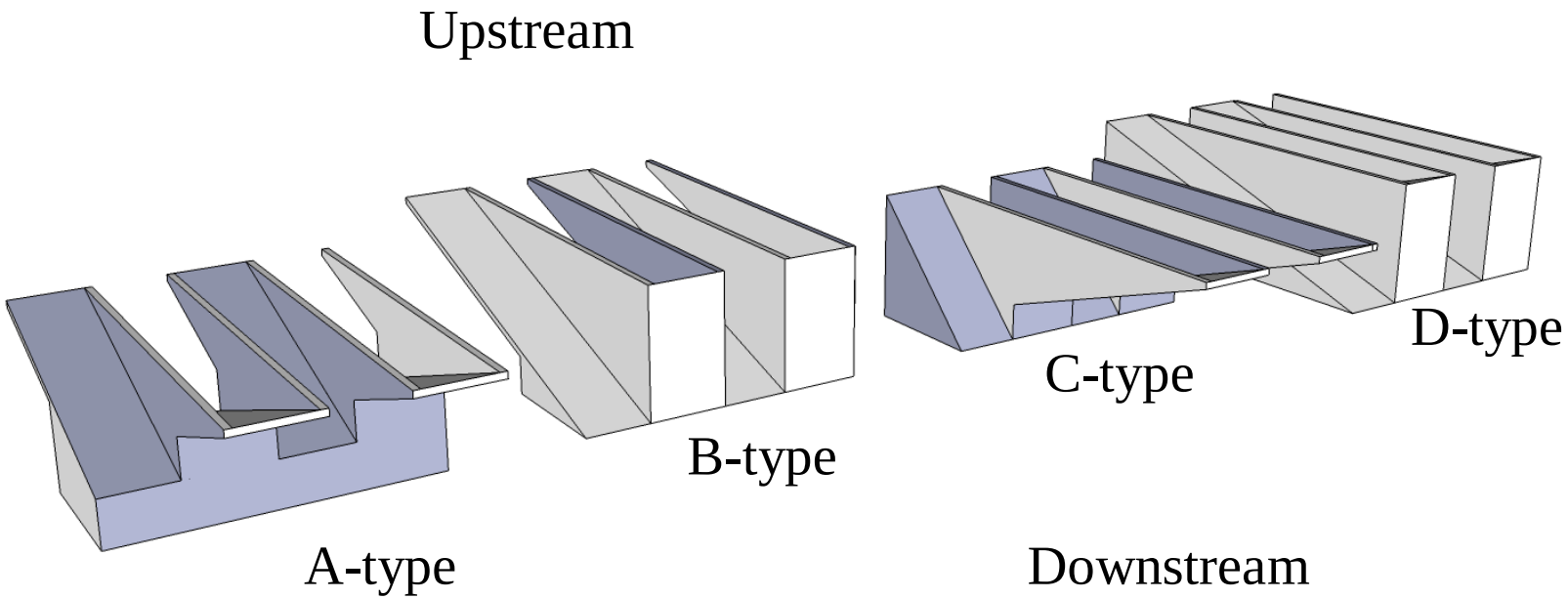
Large set of parameters → specific nomenclature published in 2011 (Pralong et al., 2011)





6. Labyrinth and PK Weirs hydraulic design and performance

4 types of PKW (Truong et al., 2006 and Lempérière et al, 2011)



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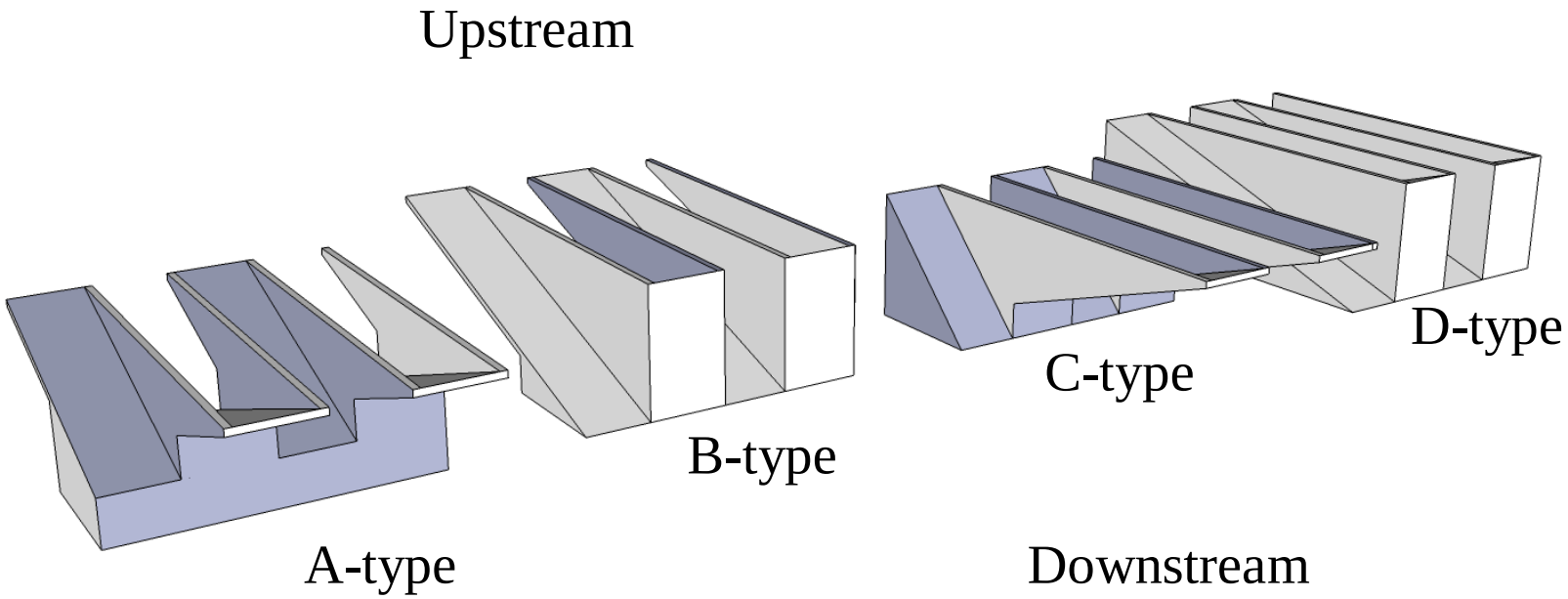

Modification of the crestprint

rectangular → trapezoidal (Cicéro et al., 2013 and Mehboudi et al., 2016)

→ Going closer to an optimized labyrinth weir crest print (Crookston and Tullis, 2013)

6. Labyrinth and PK Weirs hydraulic design and performance

Labyrinth Weir Design

Hydraulic Design and Analysis of Labyrinth Weirs. I: Discharge Relationships

B. M. Crookston, A.M.ASCE[1]; and B. P. Tullis, M.ASCE[2]

Abstract: A method is presented for the hydraulic design and analysis of labyrinth weirs based upon the experimental results of physical modeling. Discharge coefficient data for labyrinth weirs with quarter-round and half-round crest shapes are presented for sidewall angles ranging from 6 to 35°. Cycle efficiency is also introduced as a design aid, which compares the hydraulic performance of different cycle geometries. Geometric parameters that affect flow performance are discussed. The predictive accuracy of the design method is evaluated through comparisons to previously published labyrinth weir head-discharge data. The companion paper examines nappe behaviors that affect flow performance and presents hydraulic design considerations specific to nappe characteristics. **DOI: 10.1061/(ASCE)IR.1943-4774.0000558.** © *2013 American Society of Civil Engineers.*

CE Database subject headings: Weirs; Water discharge; Coefficients; Hydraulics; Design; Irrigation.

Author keywords: Labyrinth weir design; Discharge coefficients; Weir crest shape; Cycle efficiency; Local submergence.

Introduction

A labyrinth weir is a linear weir that is folded in plan-view to increase the crest length for a given channel or spillway width (Fig. 1). There are an infinite number of possible labyrinth weir configurations and design variations; however, labyrinth cycles are typically placed in a linear fashion (i.e., upstream apexes align at a common channel cross section; Fig. 1), have a sidewall angle (α) less than 30°, and are oriented towards the approaching flow.

A labyrinth weir is able to pass large discharges at relatively low heads compared to traditional linear weir structures of equal width. As a result of their hydraulic performance and geometric versatility, labyrinth weirs have been placed in streams, canals, rivers, ponds, and reservoirs as headwater control structures, energy dissipaters, flow aerators, and spillways. Labyrinth weirs are well suited for spillway rehabilitation where aging infrastructure, dam safety concerns, freeboard limitations, and revised and larger probable maximum flows have required increased spillway capacity. Recently constructed examples are: Lake Brazos spillway in Texas (Vasquez et al. 2007) and Lake Townsend spillway in Greensboro, North Carolina (Tullis and Crookston 2008).

Flow Characteristics

The geometry of a labyrinth weir produces complex three-dimensional flow patterns. At very low heads, it behaves similar to a linear weir ($\alpha = 90°$, oriented normally to the approach flow direction) of equivalent length. However, as the head increases, labyrinth weir discharge efficiency, as quantified by the discharge coefficient value, begins to decline as nappe collision and local submergence regions develop (Crookston and Tullis 2012c).

Previous Studies

Labyrinth weir head-discharge relationships have been described by various empirical equations. These relationships vary based on different definitions of the discharge coefficient, the characteristic weir length, and the upstream driving head (e.g., the inclusion of the velocity head component $V^2/2g$, described in the following). In the present study, a standard form of the weir equation, Eq. (1), was selected with the centerline length of the crest (L_c) as the characteristic weir length:

$$Q = \frac{2}{3} C_{d(\alpha°)} L_c \sqrt{2g} H_T^{3/2} \tag{1}$$

where Q = labyrinth weir discharge; $C_{d(\alpha°)}$ = dimensionless discharge coefficient; $L_c = N(2l_c + A + D)$ where N = number of cycles, l_c = centerline length of the sidewall, A = inside apex length, and D = outside apex length; g = acceleration constant of gravity; and H_T = total upstream head (unsubmerged) measured relative to the crest elevation [$H_T = V_2/2g + h$ (V is the average cross-sectional velocity at the gauging location, and h is the piezometric head upstream of the weir)].

Several earlier labyrinth weir studies resulted in published design methods; a selection is presented and discussed. Hay and Taylor (1970) presented parameter guidelines, based upon research by Taylor (1968), for sharp-crested triangular and trapezoidal labyrinth weirs. Discharge rating curves for $h/P < 0.6$ were presented in terms of a labyrinth-to-linear weir discharge ratio (based on a common channel width, W, and h), requiring discharge information for a linear weir ($\alpha = 90°$) of equivalent weir height (P), wall thickness (t_w), and crest shape. The Bureau of Reclamation (USBR) conducted model studies to aid in the design of Ute Dam (Houston 1982). Discrepancies between their experimental results and the recommendations by Hay and Taylor (1970) were attributed to different definitions of upstream head [h, Hay and Taylor (1970);

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Hydraulic Design and Analysis of Labyrinth Weirs. II: Nappe Aeration, Instability, and Vibration

B. M. Crookston, A.M.ASCE[1]; and B. P. Tullis, M.ASCE[2]

Abstract: Nappe behavior should be considered in the design of labyrinth weirs to ensure hydraulic optimization and to account for potential vibrations, pressure fluctuations, noise, and flow surging. Information regarding nappe aeration conditions (clinging, aerated, partially aerated, and drowned), nappe instability, and nappe vibrations for trapezoidal labyrinth weirs on a horizontal apron with quarter- and half-round crests ($6° \leq$ sidewall angle $\leq 35°$) is presented. Corresponding headwater ratio ranges and hydraulic behaviors associated with nappe aeration conditions are documented and discussed to aid in labyrinth weirs design, including design options directly related to nappe behavior (e.g., crest shape, crest roughness, vents, nappe breakers, notches, and staged cycles). The effects of artificial nappe aeration (a vented nappe) on nappe behavior and discharge capacity are also discussed, including recommended placements of nappe breakers. **DOI: 10.1061/(ASCE)IR.1943-4774.0000553.** © *2013 American Society of Civil Engineers.*

CE Database subject headings: Weirs; Vibration; Hydraulics; Design; Stability.

Author keywords: Labyrinth weir; Labyrinth weir nappe aeration; Labyrinth weir nappe pressure fluctuations; Nappe breaker; Labyrinth weir nappe instability; Labyrinth weir nappe noise.

Introduction

Head-Discharge Estimation

A labyrinth weir (Fig. 1) is a type of polygonal overflow structure that has a distinct geometric shape (triangular, trapezoidal, or rectangular cycles in plan-view) and advantageous hydraulic characteristics. Relative to linear weirs, labyrinth weirs have lower operating upstream heads for a given design flood, a feature that can facilitate increased reservoir storage under base-flow conditions through a higher weir crest elevation without compromising flood discharge capacity. Labyrinth weirs are used as primary or auxiliary spillways (new and rehabilitated structures) to increase discharge capacity, regulate water levels (e.g., intake ponds, residential areas, rivers with high max/min flow ratios), or as a cost-effective, passive-control alternative to gated control structures. They are hydraulically most efficient at low heads (Crookston and Tullis 2013); as the upstream total head (H_T) increases, hydraulic efficiency declines and energy dissipation increases. Although labyrinth weir discharge coefficient, $C_{d(\alpha°)}$, values may be less than that of a linear weir [$C_{d(90°)}$], the increase in the crest centerline length, L_c, typically more than compensates, providing an increase in discharge capacity at higher heads relative to linear weirs (Crookston and Tullis 2013).

Discharge coefficients and discharge rating curves for labyrinth weirs have been determined from physical models for over 50 prototype structures [e.g., Avon (Darvas 1971), Dungo (Magalhães and Lorena 1989), Hyrum (Houston 1983), Keddara (Magalhães and Lorena 1989), Lake Brazos (Tullis and Young 2005), Lake Townsend (Tullis and Crookston 2008), Ute (Houston 1982), and Woronora (Darvas 1971)]. As discussed in "Part I" (Crookston and Tullis 2013), labyrinth weir discharge is a function of crest length (L_c), cycle geometry (Fig. 2) and configuration (arced or linear), approach flow conditions [in-channel or reservoir application, Crookston and Tullis (2012a, b)], and hydraulic conditions [H_T, tailwater submergence (Tullis et al. 2007), and local submergence (Crookston and Tullis 2012c)].

Labyrinth weirs produce complex flow patterns that influence nappe behavior (e.g., nappe aeration conditions, nappe instability, and nappe vibration), which influence the head-discharge relationship.

Knowledge of labyrinth weir nappe behavior relative to labyrinth weir geometric designs and hydraulic conditions may help avoid undesirable labyrinth weir flow conditions [e.g., nappe vibration, excessive noise, nappe instabilities (also referred to as surging flow), and fluctuating pressure forces on the weir walls]. Current labyrinth weir design information (e.g., Crookston and Tullis 2013; Tullis et al. 1995) does not adequately address nappe instability, the connection between nappe aeration conditions and $C_{d(\alpha°)}$, and nappe vibration.

Nappe Behavior

The jet of water that passes over a weir is referred to as the nappe. Nappe aeration conditions for a variety of linear weirs have been previously investigated and documented (e.g., Kandaswamy and Rouse 1957; Chow 1959). In this study, four different labyrinth weir nappe aeration conditions were observed: clinging, aerated, partially aerated, and drowned. Clinging refers to the nappe adhering to the downstream face of the weir wall at lower values of H_T/P. An aerated nappe condition refers to a condition where

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6. Labyrinth and PK Weirs hydraulic design and performance

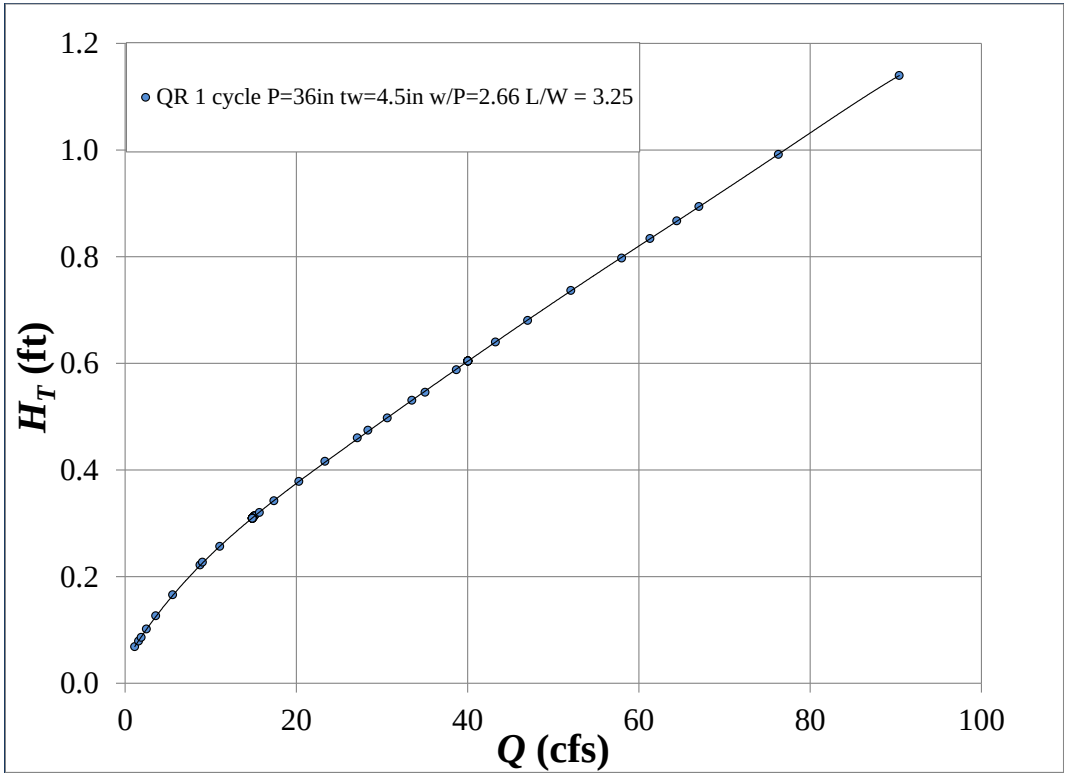
Labyrinth Weir discharge capacity

$$Q = \frac{2}{3} C_{d(\alpha^\circ)} L_c \sqrt{2g} H_T^{3/2}$$

$$Q = \frac{2}{3} C_{d(\alpha^\circ)} W \sqrt{2g} H_T^{3/2}$$

*

$$C_{d(\alpha^\circ)} = f(\alpha, t_w, P, A, \text{crest shape}, H_T, H_d, \text{approach flow}, \text{nappe})$$



6. Labyrinth and PK Weirs hydraulic design and performance

Spreadsheet design method

User Defined

Calculated

Parameter	Symbol	Value	Units	Notes			
Hydraulic Conditions - Input Data							
Design Flow	Q_{design}	1,500.00	(m^3/s)	Input	g =	9.81	(m/s^2)
Design Flow Water Surface Elevation	H	1,680.00	(m)	Input			
Approach Channel Elevation	H_{apron}	1,674.00	(m)	Input			
Crest Elevation	H_{crest}	1,678.00	(m)	Input			
Unsubmerged Total Upstream Head	H_T	2.000	(m)	Input (Piesometric head + Velocity head - losses)			
Downstream Total Head	H_d	0.50	(m)	Input (Piesometric head + Velocity head)			
Labyrinth Weir Geometry - Input Data							
Angle of Side Legs	α	12	(°)	$\alpha \sim 6° - 35°$		0.209	(rad)
Width of labyrinth (Normal to Flow)	W	99.868	(m)	Input or $W = Nw$			
Number of Cycles	N	9	-	$N=W/w$ or Input			
Crest Height	P	4.00	(m)	$P \sim 1.0H_T$			
Thickness of Wall	t_w	0.500	(m)	$t_w \sim P/8$			
Inside Apex Width	A	0.500	(m)	$A \sim t_w$			
Crest Shape	*Crest Shape*	Quarter	-	Quarter or Half			
Nappe Breakers/Vents	-	Breakers		Breakers, Vents, None			
Calculated Data							
Headwater Ratio	H_T/P	0.500	-				
Labyrinth Weir Discharge Crest Coefficient	$C_{d(\alpha)}$	0.429	-	$C_{d(\alpha)} = f(H_T/P, \alpha, Crest\ Shape)$			
Total Centerline Length of Weir	L_c	418.284	(m)	$L_c = 3/2Q_{max}/[(C_{d(\alpha)}H_T^{3/2})(2g)^{1/2}]$			
Actual Length of Sidewall	l_c	22.333	(m)	$l_c = (B-t_w)/cos(\alpha)$			
Length of Apron (Parallel to Flow)	B	22.345	(m)	Input or $B = [L_c/(2N)-(A+D)/2]cos(\alpha)+t_w$			
Cycle Width	w	11.096	(m)	$w = 2l_c sin(\alpha)+A+D$			
Outside Apex Width	D	1.310	(m)	$D = A+2t_w tan(45-\alpha/2)$			
Magnification Ratio	M	4.188	-	$M=L_c/(wN)$			
Cycle Width Ratio	w/P	2.774	-	Normally between 3 and 4			
Relative Thickness Ratio	P/t_w	0.125					
Apex Ratio	A/w	0.045	-	<0.08			
Cycle Efficiency	ε'	1.798	-	$\varepsilon' = C_{d(\alpha)}M$			
Efficacy	ε	2.226		$\varepsilon = C_{d(\alpha)}M/C_{d(90°)}$			
# of Nappe Breakers or Vents	-	9		Breakers on ds Apex, vents on sidewall			
Linear Weir Discharge Coefficient	$C_{d(90°)}$	0.808	-	$C_{d(90°)} = f(H_T/P, t_w, Crest\ Shape)$			
Length of Linear Weir for same Q	-	222.329	(m)	$L = 3/2Q_{design}/[(C_{d(90°)}H_T^{3/2})(2g)^{1/2}]$			
Submergence							
Downstream/Upstream Ratio of Unsubmerged Head	H_d/H_T	0.25	(m)				
Submerged Head Discharge Ratio	H^*/H_T	1.013	-	Piecewise function Tullis et al (2007)			
Submerged Upstream Total Head	H^*	2.025	(m)				
Submergence	S	0.247	-	$S = H_d/H^*$			
Submerged Weir Discharge Coefficient	$C_{d\text{-}sub}$	0.421	-	$C_{d(\alpha)}*(H_T/H^*)^{3/2}$			

6. Labyrinth and PK Weirs hydraulic design and performance

Design method

Parameter	Symbol	Value	Units	Notes			
Hydraulic Conditions - Input Data							
Design Flow	Q_{design}	1,500.00	(m^3/s)	Input	g =	9.81	(m/s^2)
Design Flow Water Surface Elevation	H	1,680.00	(m)	Input			
Approach Channel Elevation	H_{apron}	1,674.00	(m)	Input			
Crest Elevation	H_{crest}	1,678.00	(m)	Input			
Unsubmerged Total Upstream Head	H_T	2.000	(m)	Input (Piesometric head + Velocity head - losses)			
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Thickness of Wall	t_w	0.500	(m)	$t_w \sim P/8$			
Inside Apex Width	A	0.500	(m)	$A \sim t_w$			
Crest Shape	*Crest Shape*	Quarter	-	Quarter or Half			
Nappe Breakers/Vents	-	Breakers		Breakers, Vents, None			

6. Labyrinth and PK Weirs hydraulic design and performance

Design method

Calculated Data				
Headwater Ratio	H_T/P	0.500	-	
Labyrinth Weir Discharge Crest Coefficient	$C_{d(\alpha)}$	0.429	-	$C_{d(\alpha)} = f(H_T/P,\ \alpha,\ Crest\ Shape)$
Total Centerline Length of Weir	L_c	418.284	(m)	$L_c = 3/2 Q_{max}/[(C_{d(\alpha)} H_T^{3/2})(2g)^{1/2}]$
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Magnification Ratio	M	4.188	-	$M=L_c/(wN)$
Cycle Width Ratio	w/P	2.774	-	
Relative Thickness Ratio	P/t_w	0.125		
Apex Ratio	A/w	0.045	-	<0.08
Cycle Efficiency	ε'	1.798	-	$\varepsilon' = C_{d(\alpha)} M$
Efficacy	ε	2.226		$\varepsilon = C_{d(\alpha)} M/C_{d(90°)}$
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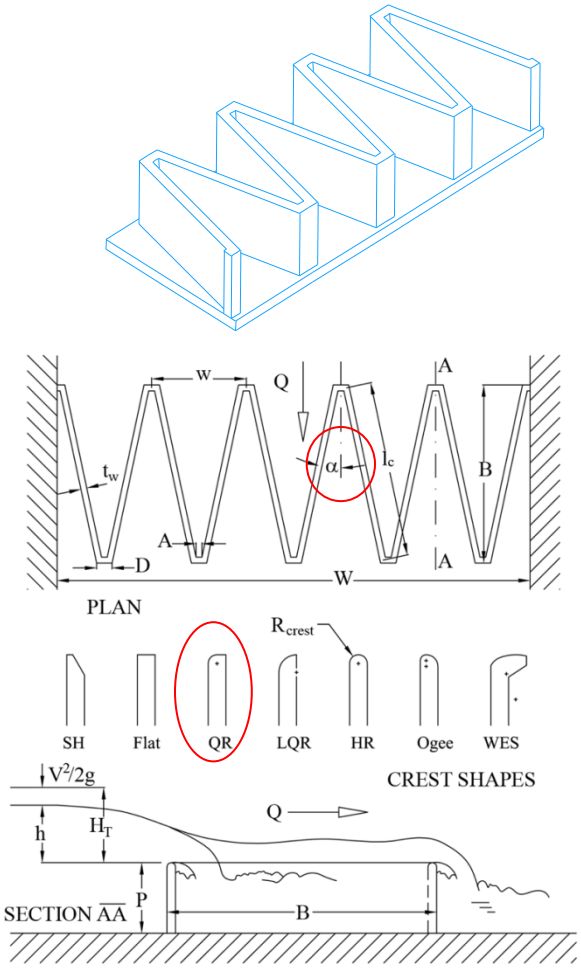
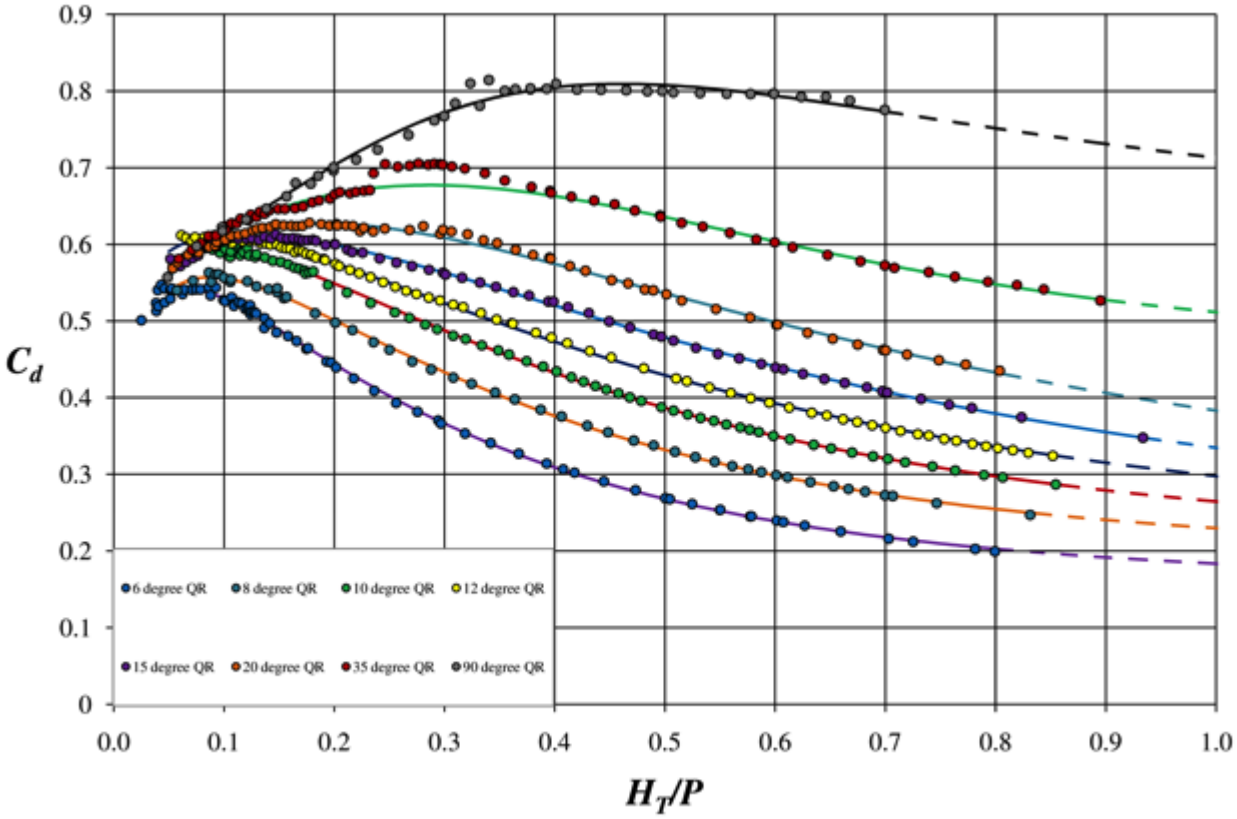
6. Labyrinth and PK Weirs hydraulic design and performance

Discharge coefficient data (QR)

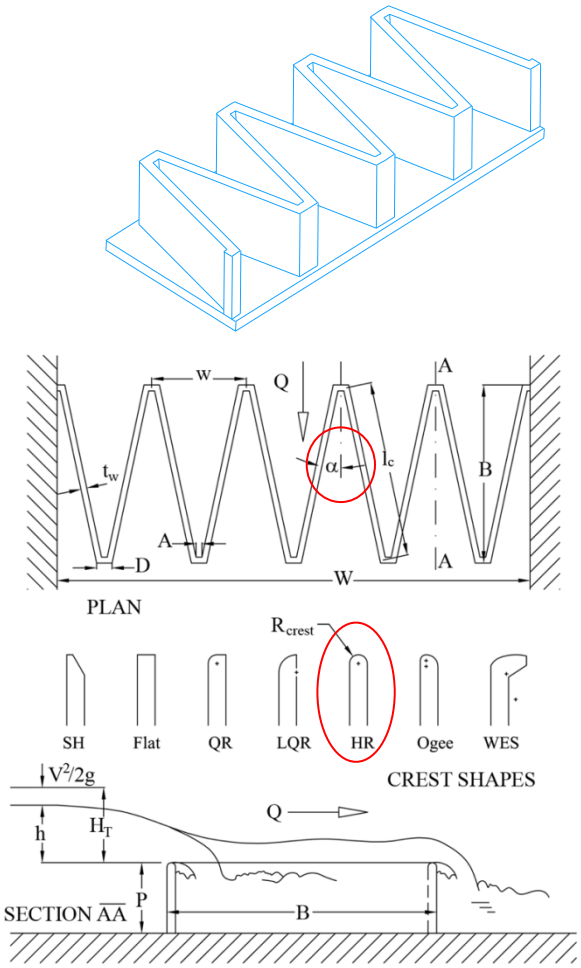


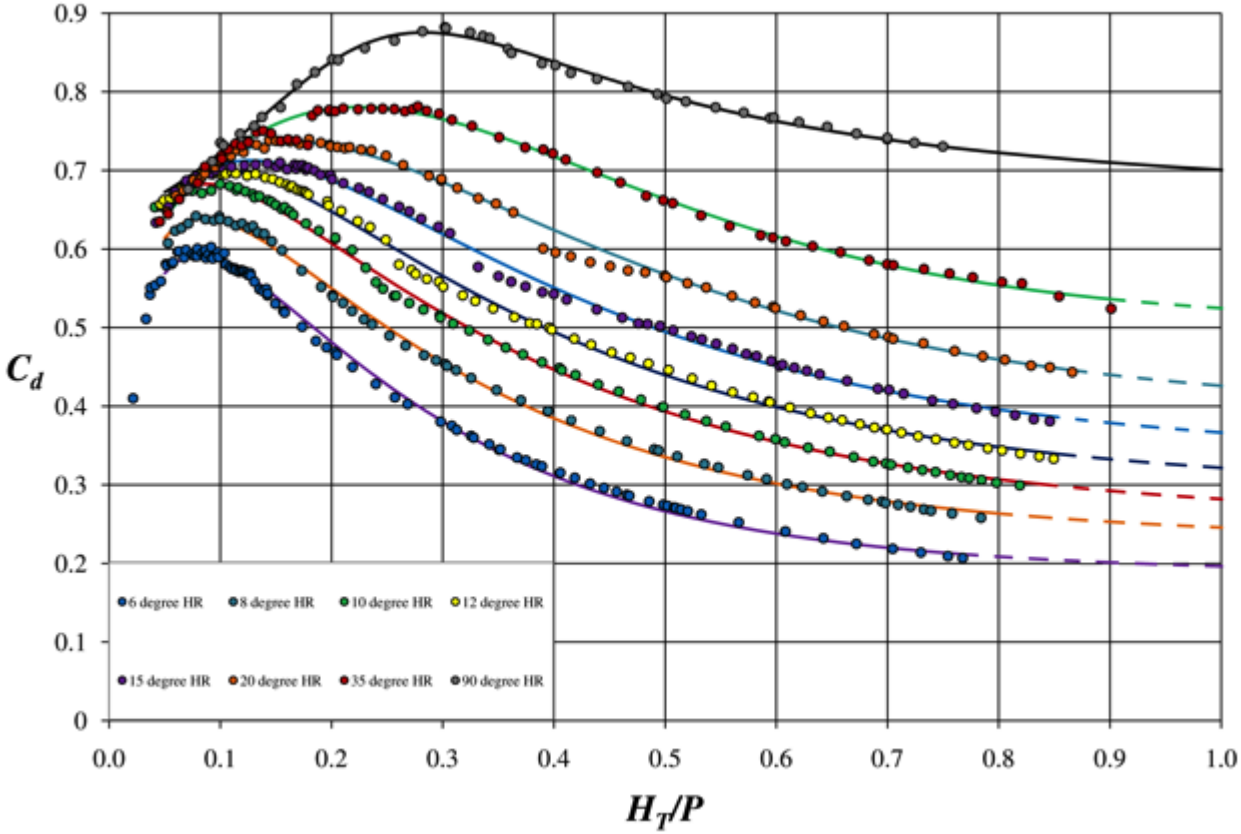


$$Q = \frac{2}{3} C_{d(\alpha^\circ)} L_c \sqrt{2g} H_T^{3/2}$$

6. Labyrinth and PK Weirs hydraulic design and performance

Discharge coefficient data (HR)





$$Q = \frac{2}{3} C_{d(\alpha^\circ)} L_c \sqrt{2g} H_T^{3/2}$$

6. Labyrinth and PK Weirs hydraulic design and performance

PKW = free surface weir

$$Q_P \propto \sqrt{2gH^3}$$

Q = discharge
H = upstream head

Two approaches to derive the proportion factor, which represent the effect of the crest length and shape (Leite Ribeiro et al, 2012)

developed crest length

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

(Leite Ribeiro et al, 2012)

width of the weir

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

(Ouamane & Lempérière, 2006; Machiels et al., 2011)

discharge coefficient

6. Labyrinth and PK Weirs hydraulic design and performance

Comparison with a standard linear weir of same width W (Leite Ribeiro et al, 2012)

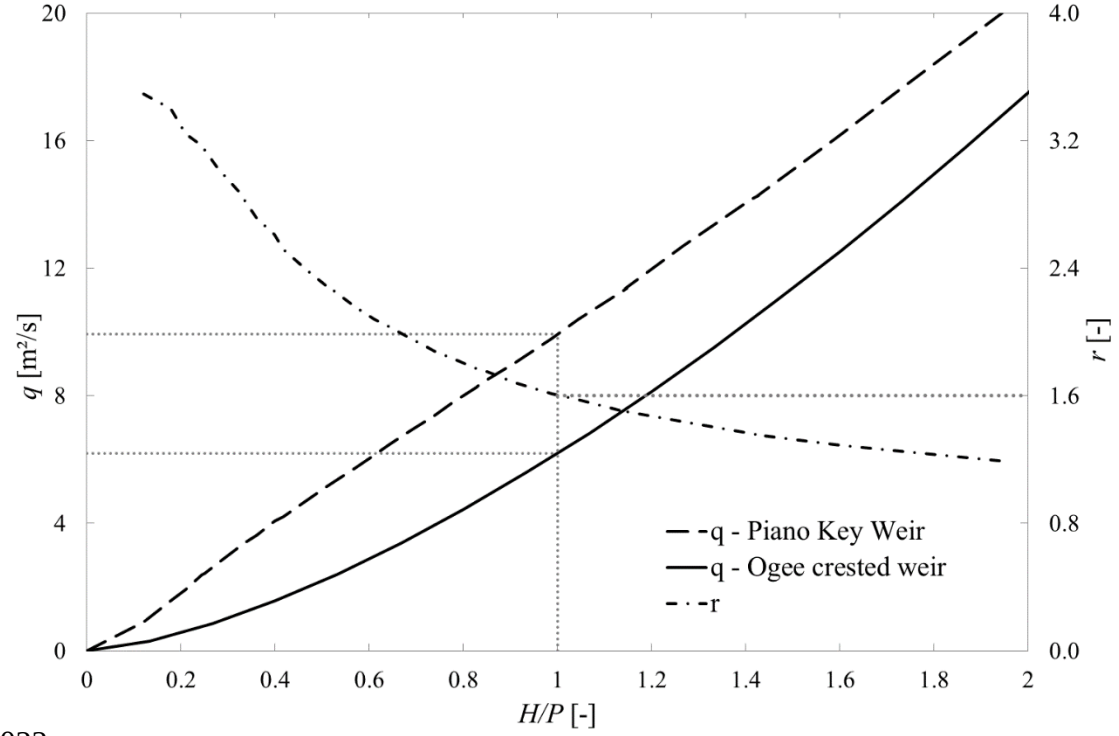
$$r = \frac{Q_P}{Q_S} = \frac{C_{P,W}}{C_S}$$

Discharge coefficient of the PKW (includes influence of crest shape and developped crest length)

Discharge increase ratio

Discharge coefficient of a standard linear weir

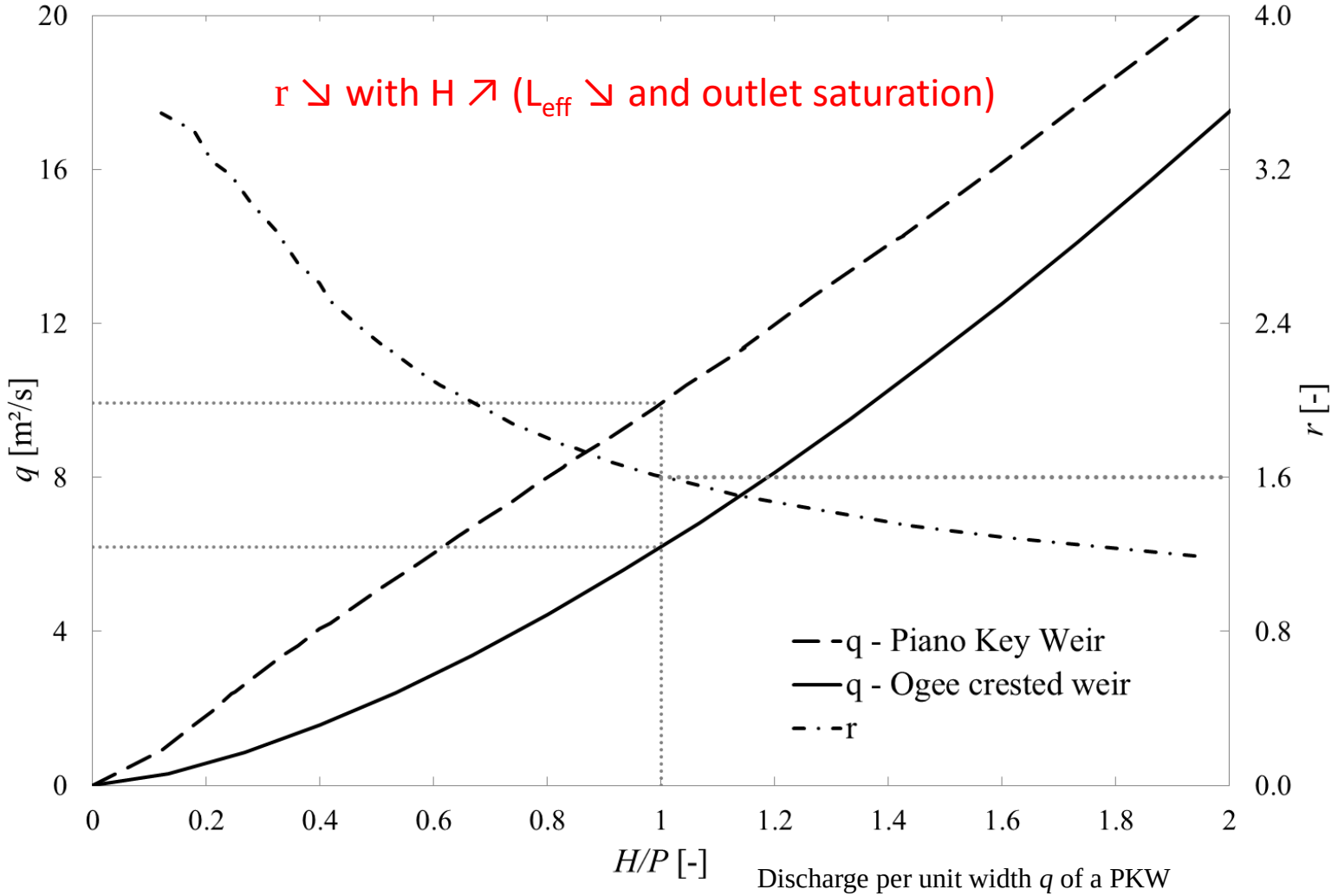
$$C_S = 0.494\left(\frac{H}{H_d}\right)^{0.12}$$



Discharge per unit width q 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, $C_S = 0.494$

6. Labyrinth and PK Weirs hydraulic design and performance

Comparison with a standard linear weir of same width *W*



Discharge per unit width *q* 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 = 0.494

6. Labyrinth and PK Weirs hydraulic design and performance

Main geometrical parameters

- Extensive experimental works (Biskra, Algeria; LNHE, France; LCH, Switzerland; UWRL, USA; HECE, Belgium; Vietnam; …)

- Key geometric parameters …
  - Crest length magnification ratio L/W (Ouamane & Lempérière, 2006; Leite Ribeiro et al., 2012)
  - Weir height P; keys width ratio W_i/W_o, overhangs length ratio B_o/B_i (Machiels et al., 2011 and 2012)

- … and optimal values
  - $L/W \approx 5$ (Lempérière, 2009 and 2011)
  - $P/W_u \approx 1.33$ (Machiels et al., 2014)
  - $W_i/W_o \in [1.25 - 1.6]$ (Leite Ribeiro et al, 2012; Anderson & Tullis, 2013, Machiels et al., 2014)
  - $B_o/B_i > 1$ with $B_i \neq 0$ (Lempérière et al., 2011; Machiels et al., 2014)

→ Hydraulic optimum

  - $P/W_u = 0.5$ (Machiels, 2012)
  - $W_i/W_o = 1$ (Machiels et al., 2014)
  - $B_o/B_i = 1$ (Machiels et al., 2014)

→ Technico-economic optimum

6. Labyrinth and PK Weirs hydraulic design and performance

Main geometrical parameters

- At constant crest length magnification ratio (Erpicum et al., 2014)

→ P is a key geometric parameter
  - To be analyzed depending on absolute upstream head values
  - May increase by a factor 2 to 4 the gain in efficiency

→ W_i/W_o is a secondary parameter
  - May increase by 30% the efficiency

→ B_o/B_i is a secondary parameter
  - May increase by 20% the efficiency

6. Labyrinth and PK Weirs hydraulic design and performance

Main geometrical parameters

- At constant crest length magnification ratio (Erpicum et al., 2014)

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  → W_i/W_o is a secondary parameter
    - May increase by 30% the efficiency

  → B_o/B_i is a secondary parameter
    - May increase by 20% the efficiency

- Lempérière « cost efficient » design (Lempérière, 2009)

$$L/W = 5;\ P/W_u = 0.83;\ W_i/W_o = 1.25;\ B_o/B_i = 1$$

  is close (15%) to the hydraulic optimum (Erpicum et al., 2014)

- Strong influence of the crest shape at low heads (Pfister and Schleiss, 2013)
- Importance of sidewalls thickness (up to 20% influence on discharge capacity) (Laugier et al, 2013)

6. Labyrinth and PK Weirs hydraulic design and performance

Design equations

- Lempérière, 2009
- Kabiri-Samani et al., 2012
- Leite Ribeiro et al., 2012
- Anderson and Tullis, 2103
- Machiels, 2012 ; Machiels et al., 2014

→ Based on comprehensive and systematic model test series
More or less complex
Empirical or process oriented

→ **Only valid within the limits of the parameters range specified by their authors**

(Pfister et al., 2012; Pfister and Schleiss, 2013; Crookston et al., 2016)

6. Labyrinth and PK Weirs hydraulic design and performance

Design equations

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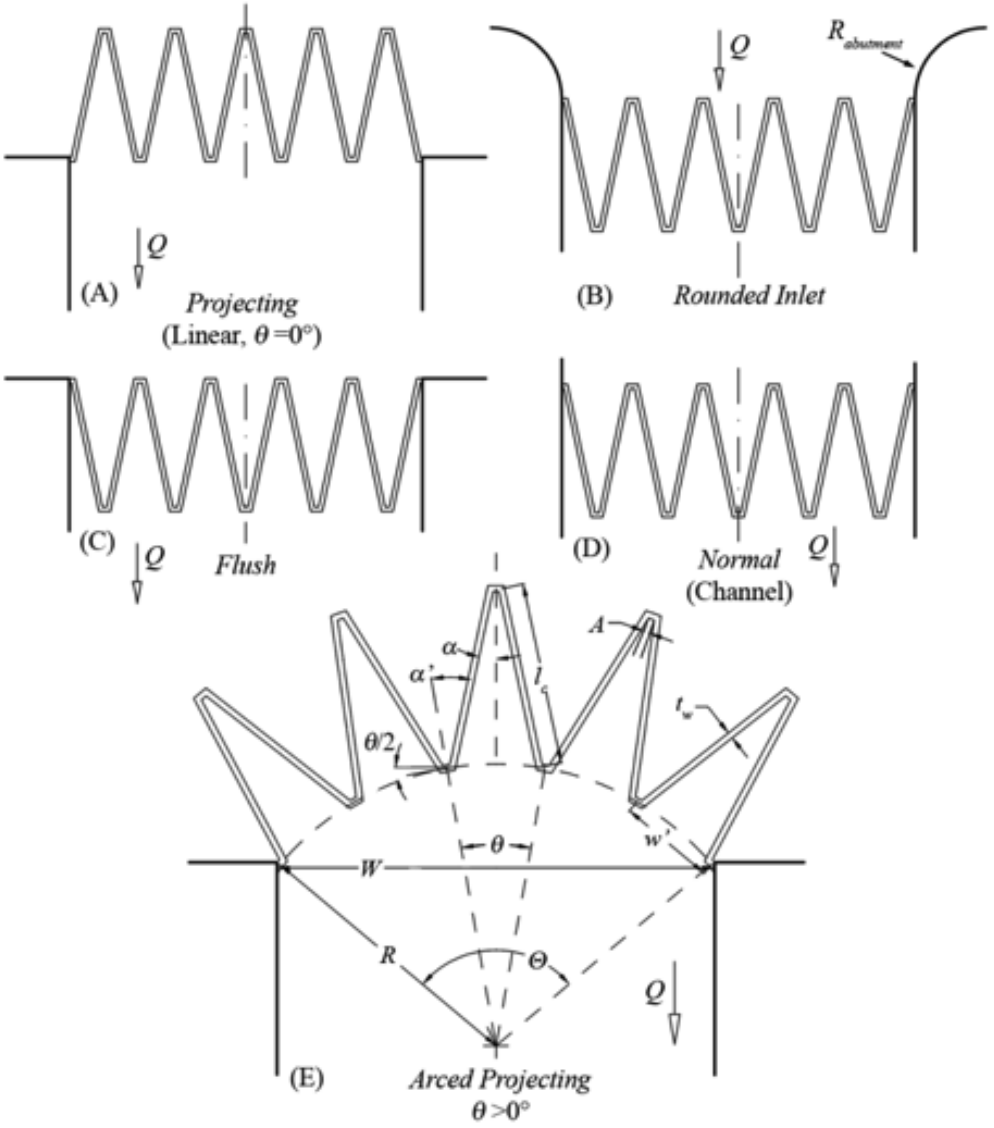
→ Based on comprehensive and systematic model test series
More or less complex
Empirical or process oriented

→ **Only valid within the limits of the parameters range specified by their authors**
(Pfister et al., 2012; Pfister and Schleiss, 2013; Crookston et al., 2016)

→ **It is recommended to consider and discuss the application of several equations in order to assess the uncertainty on their results**

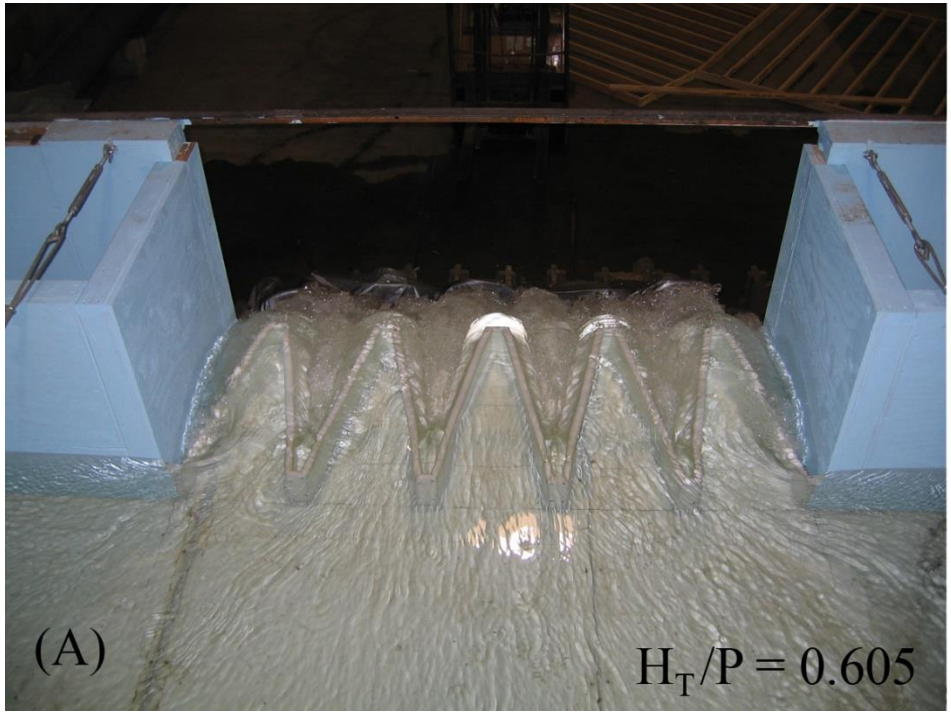
6. Labyrinth and PK Weirs hydraulic design and performance

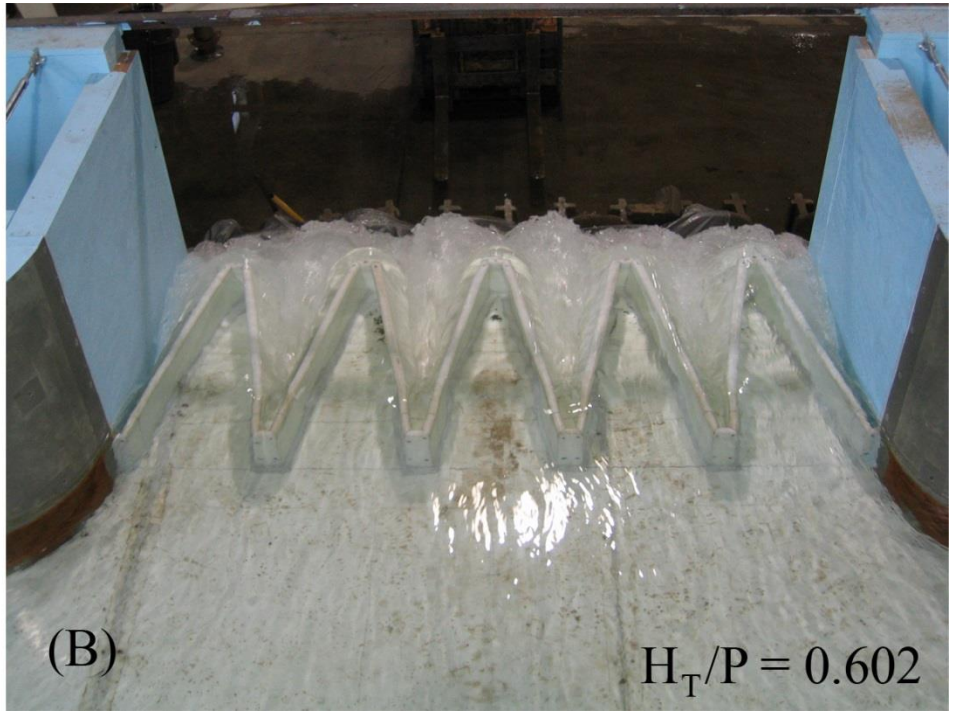
Configurations/Abutments



6. Labyrinth and PK Weirs hydraulic design and performance

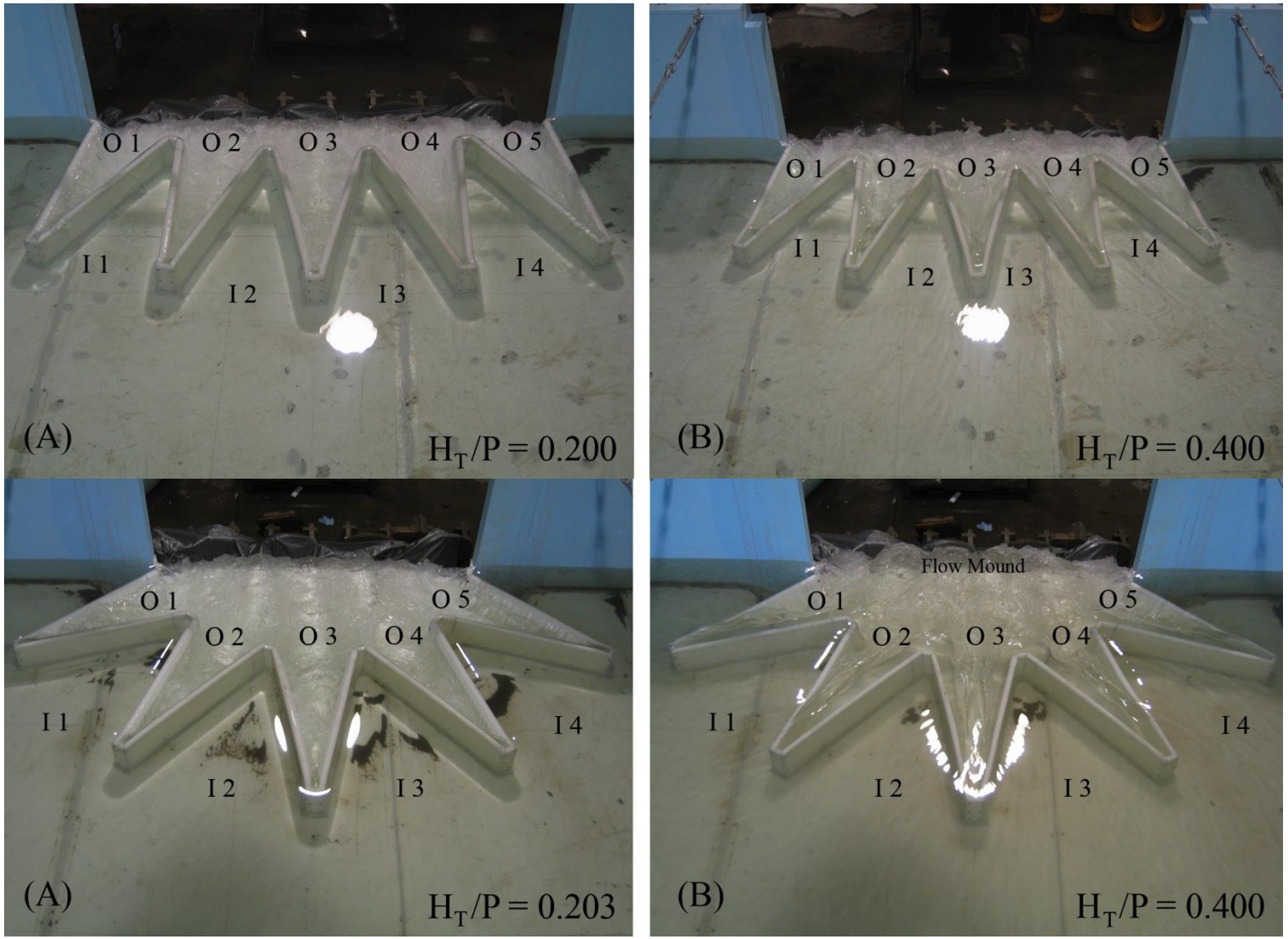
Configurations/Abutments





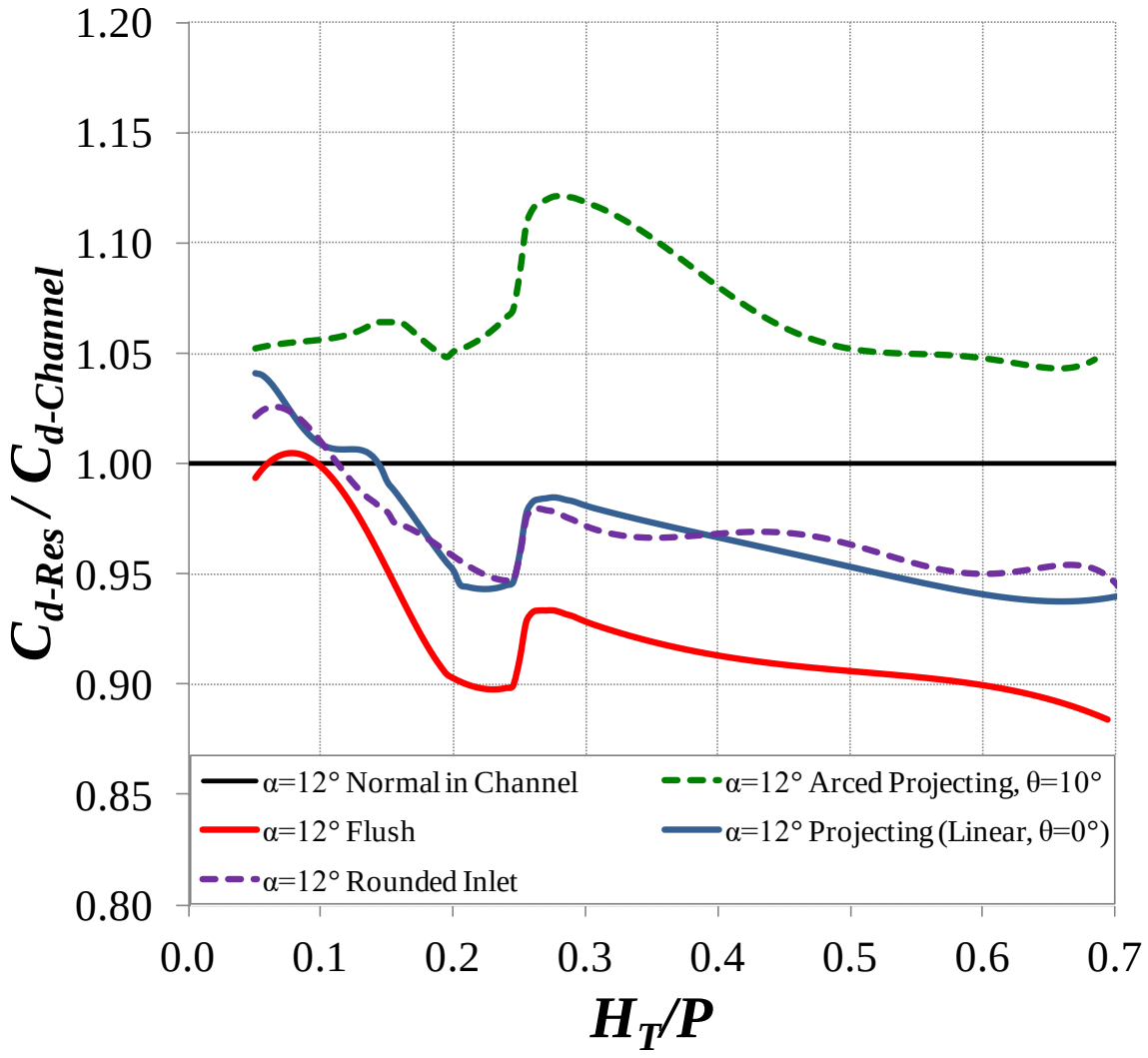
6. Labyrinth and PK Weirs hydraulic design and performance

Arced labyrinth weirs



6. Labyrinth and PK Weirs hydraulic design and performance

Reservoir vs. In-channel



6. Labyrinth and PK Weirs hydraulic design and performance

Labyrinth weir submergence

6. Labyrinth and PK Weirs hydraulic design and performance

Labyrinth weir submergence

Ogee crest weir, Iowa River,
Iowa City (USA)

6. Labyrinth and PK Weirs hydraulic design and performance

Labyrinth weir submergence

Head-Discharge Relationships for Submerged Labyrinth Weirs

B. P. Tullis, M.ASCE[1]; J. C. Young[2]; and M. A. Chandler[3]

Abstract: Low-head labyrinth weir control structures installed on mild sloping channels or where the channel downstream of the weir is constricting and/or heavily vegetated can experience submergence. Weir submergence occurs when the tailwater surpasses the weir crest elevation, causing an increase in the upstream driving head for a given discharge, relative to a free-discharge condition. The most familiar relationship for predicting submerged weir head-discharge relationships is likely that of James R. Villemonte, which he published in 1947 for sharp-crested linear weirs. For lack of a better alternative, Villemonte's relation has also been applied to predicting submerged labyrinth weir performance. A new dimensionless submerged head relationship developed in this study is presented for submerged labyrinth weirs. A similar relationship is also presented for linear sharp-crested weirs. The accuracy of the submerged linear weir relationship was equivalent to Villemonte's and is simpler to solve when working in terms of total upstream head. Relative to Villemonte's relationship applied to labyrinth weirs, the new submerged labyrinth weir relationship reduced the predictive errors from 23 to 3.5% (maximum) and 8.9 to 0.9% (average), relative to the experimental data.

DOI: 10.1061/(ASCE)0733-9429(2007)133:3(248)

CE Database subject headings: Weirs; Control structures; Slopes; Channels; Flow measurement.

Introduction

Weir structures are used in canal, river, and reservoir applications for flow measurement, flow diversion, and/or discharge control, and are typically designed for free-flow conditions, meaning the surface of the tailwater is below the weir crest. Under free-flow conditions, the weir head-discharge relationship is governed primarily by the weir geometry and the approach flow conditions. When the tailwater exceeds the crest elevation, the weir is submerged. For a given discharge over the weir, a higher upstream head is required to pass the flow under submerged conditions relative to free-flow conditions. Submergence is more likely to be of concern when low-head control structures are installed on mild sloping rivers or canals with constrictions and/or heavy vegetation downstream of the control structure.

Villemonte (1947) developed a relatively accurate and widely used relationship, based on a flow reduction factor, describing the submergence effects on the hydraulic performance of sharp-crested linear weirs. This relationship is referred throughout the paper as Villemonte's relationship. For lack of a specific labyrinth weir submergence relationship, Villemonte's submerged linear weir relationship has also been used, more recently, to describe submerged labyrinth weir performance (Taylor 1968; Falvey 2003). The correlation between Villemonte's relationship and Taylor's submerged labyrinth weir data (Taylor 1968) was poor, however, Villemonte's method produced conservative estimates (Falvey 2003). No alternative methods were found in the literature for estimating submerged labyrinth weir performance.

New dimensionless submerged head relationships were developed in this study, one for labyrinth weirs and one for linear sharp-crested weirs. The linear weir relationship correlates very well to Villemonte's prediction of sharp-crested linear weir submergence performance. The new submerged labyrinth weir relationship, however, proved to be significantly more accurate than Villemonte's relationship for describing submerged labyrinth weir performance.

Background

Considerable research has been reported describing submergence effects on the head-discharge relationships of linear weirs, however, little published information could be found regarding the effects of submergence on labyrinth weirs. Early researchers of labyrinth weir hydraulics neglected detailed research of submergence effects stating that such structures should not be designed for these conditions (Taylor 1968). The significant increase in flow capacity of labyrinth weirs, relative to linear weirs of the same width, however, has resulted in a wider variety of labyrinth weir applications, including applications where submergence is a factor.

Weir Basics

A weir head-discharge relationship relates the weir discharge to the upstream head required to pass that flow rate. For free-flow

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Note. Discussion open until August 1, 2007. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on October 12, 2005; approved on April 11, 2006. This paper is part of the ***Journal of Hydraulic Engineering***, Vol. 133, No. 3, March 1, 2007. ©ASCE, ISSN 0733-9429/2007/3-248–254/$25.00.

Technical Note

Piano Key Weir Submergence in Channel Applications

M. R. Dabling, S.M.ASCE[1]; and B. P. Tullis, M.ASCE[2]

Abstract: Weir submergence can influence head-discharge relationships for weirs used in channel applications when high tailwater conditions exist owing to a downstream control. Weir submergence describes a condition in which the water level downstream of the weir exceeds the weir crest elevation. When a weir becomes submerged, the driving head required to pass a specific discharge over the weir can increase significantly relative to a free-flow condition. In this study, the effects of tailwater submergence on laboratory-scale piano key weir head-discharge relationships were evaluated experimentally and compared with previously published data for labyrinth and sharp-crested linear weir submergence. The results of this comparison show that for relatively low levels of submergence, the piano key weir requires less upstream head relative to the labyrinth weir to pass a given discharge. This increase in efficiency was minimal (< 6%) and was reversed at higher submergence levels. **DOI: 10.1061/(ASCE)HY.1943-7900.0000563.** *© 2012 American Society of Civil Engineers.*

CE Database subject headings: Weirs; Submerging; Channels.

Author keywords: Piano key weir; Weir submergence; Submerged head-discharge relationship; Non-linear weir; Labyrinth weir.

Introduction

Weirs are commonly used for flow measurement, flow diversion, and/or flow control in canals, rivers, and reservoirs. Although weirs are generally designed to operate under free-flow conditions, they can become submerged under certain conditions. Weir submergence occurs when the downstream water level exceeds the crest elevation, a condition that is more common for weirs situated in rivers or canals with a mild slope, flow constrictions, and/or excess in-stream vegetation downstream. Submergence is quantified by the ratio of the total head downstream of the weir to the total head upstream. At low submergence levels, the elevated tailwater does not affect the free-flow total upstream head (H_o), a condition known as modular submergence. When the modular submergence limit is exceeded, the upstream total upstream head (H^*) increases relative to H_o for a given weir discharge (Q), increasing the potential for upstream flooding. Fig. 1 shows the difference between the submerged and free-flow hydraulic parameters.

Relative to traditional linear weirs (e.g., ogee crest), the use of nonlinear weirs to increase the flow capacity in discharge channels of limited width without significantly increasing the required H_o is becoming more common, especially for spillway rehabilitation. Linear and nonlinear weir designations refer to the layout of the weir wall(s) in plan view. Typical nonlinear weirs include labyrinth and piano key (PK) weirs (Fig. 2). The PK weir is essentially a rectangular labyrinth weir with cantilevered upstream and/or downstream apexes and ramped floors in the inlet and outlet cycles or keys. The PK weir's cantilevered apexes help to produce a longer weir crest length relative to a rectangular labyrinth weir with the same footprint.

The PK weir's reduced footprint relative to its crest length makes it particularly well suited for spillway applications with limited footprint space (i.e., on top of a narrow concrete gravity dam). The number of geometric parameters that PK weirs have is significant (see Pralong et al. 2011 for a full description of the PK geometric parameters); the influence of the PK weir geometric parameters on the discharge efficiency, either individually or collectively, is not well understood. On the basis of preliminary testing, however, Lempérière (2009) has recommended a standard PK weir geometry for design. Anderson (2011) found that the PK weir discharge capacity could be improved beyond the standard design by adding various design modifications (e.g., rounded noses on the upstream apexes; a parapet wall on top of the weir; improved crest shapes, such as a half-round).

In addition to the top-of-dam applications, recent interest has focused on using PK weirs in river and channel applications (Ho Ta Khanh et al. 2011). For channel applications without significant weir footprint restrictions, Anderson (2011) found that trapezoidal labyrinth weirs typically provide more discharge capacity per unit weir length than do PK weirs. For cases in which the footprint is constrained in width and/or length, space may be insufficient to accommodate the required labyrinth weir wall length; in such cases, a PK weir would likely produce a higher total weir discharge owing to its compact geometry (large weir length for a given footprint size). Anderson and Tullis (2012) found that a Type-A PK weir was hydraulically more efficient than a rectangular labyrinth weir with the same crest layout (i.e., same crest length and layout in plan view).

When deciding between a labyrinth and a PK weir for a channel application, the potential influence of submergence, along with the free-flow discharge capacity of both weirs, must be considered. In this study, the submerged discharge characteristics of PK weirs (with and without modifications) are evaluated and compared with the submergence characteristics of labyrinth and sharp-crested linear weirs.

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6. Labyrinth and PK Weirs hydraulic design and performance

Floating debris

For **Rehabilitation** projects, general comments :

- If other **existing gated** spillways => high specific flows

  => **Attract debris**

- Gates generally operated first for medium floods (often until 100 years Return period)
- As freeflow spillways, debris will pass when water head increases

6. Labyrinth and PK Weirs hydraulic design and performance

Floating debris

Debris-Blocking Sensitivity of Piano Key Weirs under Reservoir-Type Approach Flow

Michael Pfister[1]; Damiano Capobianco[2]; Blake Tullis, M.ASCE[3]; and Anton J. Schleiss[4]

Abstract: The collection of floating woody debris at flow control structures, such as spillways and weirs, can potentially result in reduced discharge efficiency (higher upstream head for a given weir discharge). Compared to less hydraulically-efficient control structures, piano key weirs have higher discharge efficiency (lower upstream heads for a given discharge), which may increase the likelihood of woody debris collection. A systematic laboratory study was conducted to evaluate the interaction between various piano key weir geometries and woody debris types and sizes. The results of individual (noncumulative) debris tests 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%. **DOI: 10.1061/(ASCE)HY.1943-7900.0000780.** © *2013 American Society of Civil Engineers.*

CE Database subject headings: Debris; Probability; Weirs; Control structures; Floods; Spillways.

Author keywords: Piano key weir; Woody debris; Blocking probability; Discharge capacity.

Introduction

Floating wooden debris (i.e., driftwood) is often transported by a river course during flood events. Flooding associated with intense rain events in relatively dry zones can wash out unstable trees or deadwood. Under extreme conditions, landslides or bank erosion may add entire trees to a river. Woody debris, often combined with urban waste, can cause serious problems at hydraulic structures such as weirs (Bezzola and Hegg 2007), bridge decks (Bezzola et al. 2002; Schmocker and Hager 2011) or bridge piers (Melville and Dongol 1992; Pagliara and Carnacina 2010). Near structures, debris can block and thereby reduce the open flow area for water passage, which leads to reduction in discharge capacity and increase of upstream flow depth. The blocking generated by woody debris at weirs and spillway inlets is of particular interest, as the latter may affect the safety of such structures. Namely, the maximum spilling capacity may be drastically reduced due to blocked gates and/or weir obstruction.

Small and irregular flow cross-sections where accelerations occur are often prone to debris blockage. Depending on the size of the woody debris and the weir geometry, non-linear weirs, such as piano key weirs (PKWs), can also function as debris collectors. The nonlinear nature of PKWs (folded back and forth in plan view to make repeating cycles or keys) allows for significantly longer weir lengths than could be achieved using a traditional linear weir for a given spillway channel width. The narrow design of the PKW's inlet and outlet keys produce an increased potential for debris collection (e.g., the weir functions similarly to a trash rack for longer woody debris). The high discharge efficiency of PKWs relative to linear weirs can also influence debris collection; the flow depth over a PKW for a given discharge will be less than would be required by linear structures, thus providing less flow momentum per unit of weir length for flushing debris. If the catchment is forested, the potential effects of driftwood on the PKW head-discharge relation should be considered as part of the hydraulic design (Leite Ribeiro et al. 2012a, b).

There are few studies published on the effect of debris at PKWs. Ouamane and Lempérière (2006) conducted preliminary model tests with floating debris. They observed that no debris was collected under the apex overhangs during the reservoir filling process. During PKW operation, however, debris got stuck in the inlet key at small discharges, resulting in a slight reduction in discharge efficiency. A discharge coefficient reduction of about 10% was reported for $H/P < 0.5$, where H = total upstream head measured relative to the PKW crest elevation, and P = weir height. As the weir's discharge increased, the debris was eventually washed downstream. In 2007, the Laboratory of Hydraulic Constructions (LCH) investigated [Barrage de Saint-Marc: Etude sur modèle des évacuateurs de cure (Saint-Marc Dam: Spillway model study), unpublished report] the effects of floating wood on the St. Marc Dam (France) emergency PKW spillway and reported no debris-associated reduction in discharge capacity when the gates of the adjacent principal spillway were fully open, attracting a significant portion of the debris. Laugier (2007) reports of systematic model tests with debris of the emergency PKW spillway of Goulours Dam (France), concluding that (1) an increasing head tends to eventually remove previously blocked debris; (2) the principal gated spillways attracts debris before the PKW starts operation; (3) most debris

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COMMISSION INTERNATIONALE
DES GRANDS BARRAGES

VINGT-CINQUIÈME CONGRÈS
DES GRANDS BARRAGES
Stavanger, Juin 2015

DISCHARGE CAPACITY OF PK-WEIRS CONSIDERING FLOATING WOODEN DEBRIS [1]

Michael PFISTER
Research and Teaching Associate, Laboratory of Hydraulic Constructions (LCH), Ecole Polytechnique Fédérale de Lausanne (EPFL)

Anton J. SCHLEISS
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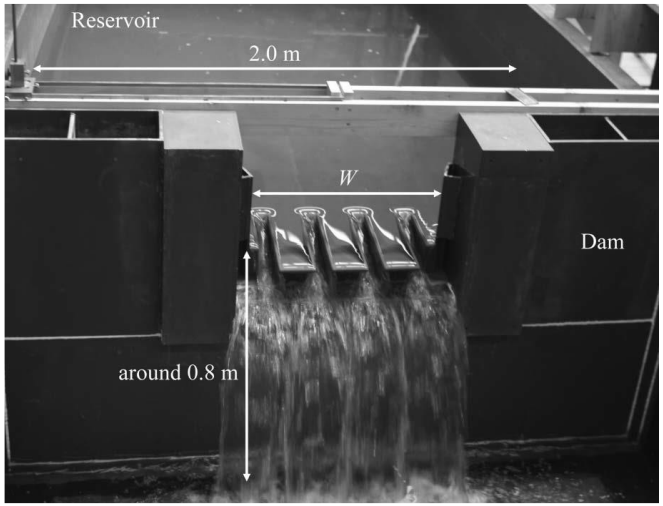
SWITZERLAND

6. Labyrinth and PK Weirs hydraulic design and performance

Floating debris

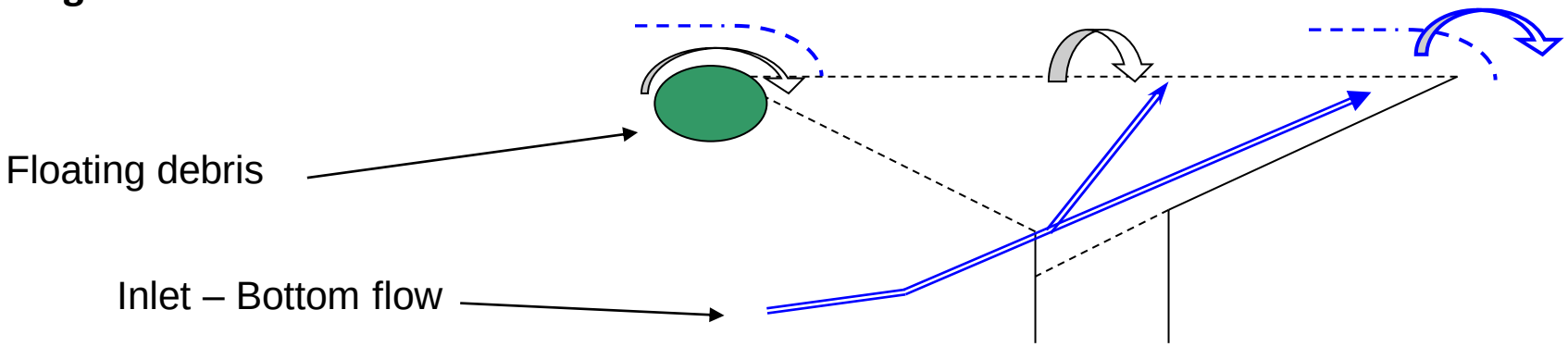
Physical models of PKW and floating debris

- 100% of blockage if D/H>1 for trunks (>0.5 with roots)
- No blockage if D/H<0.3 for trunks (<0.1 with roots)
- For 0.2<H/P<0.5, increase in H below 20% due to debris accumulation



6. Labyrinth and PK Weirs hydraulic design and performance

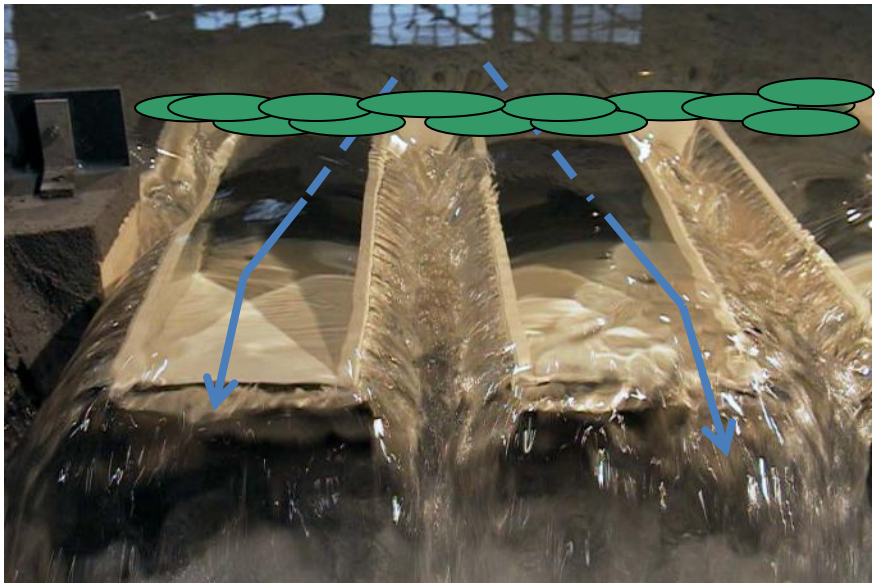
Floating debris



⇒ **Most of flow runs below the water surface**

⇒ **In case of debris blocking,**

Residual capacity ~ 80%



6. Labyrinth and PK Weirs hydraulic design and performance

- The outlet has an inclined downstream slab (crawling) which completely eliminates the risk (no air pocket)
- The inlet functions as a partially lowered flap gate but has a very small width
- Air pockets can exist downstream the sidewalls even if the flow is usually drowned in the upstream portion of the outlet crawling and if the jet of the 2 sidewalls facing each other can intersect at a certain flow, which may allow air to replenish the underside pocket
- The risk of vibration seems less critical for the hyperstatic structure that is a PKW

Aeration

- French PKWs equipped with aeration pipes (Laugier et al., 2013) with first design rules :
  - Air flow=10% water flow
  - Diameter of air pipes to have Vair<50 m/s

Malarce dam PKW – France

Gloriette dam PKW – France

6. Labyrinth and PK Weirs hydraulic design and performance

Aeration

- New air network design method and analysis (Vermeulen et al., 2017):
  - Ervine and Elsawi formula applied to inlet and sidewalls only using parameters estimated by physical or numerical modeling and literature
  - ΔP<1.5m of water with estimation of head losses in the air network

$$Q_{air} = (q \times L) \times 0.26 \times \frac{L}{p} \times \left(\frac{d}{h}\right)^{0.446} \times \left(1 - \frac{V_m}{V_i}\right)$$

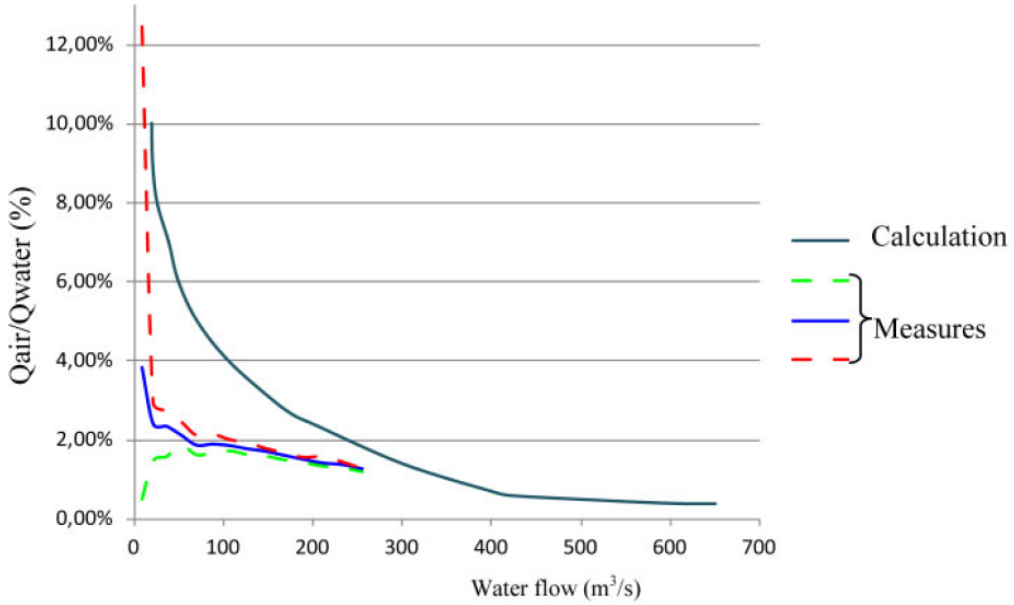
- Q_{air}, carried air flow (m^3/s)
- q, water linear flow overflowing inlets or sidewalls (m^3/s)
- L, nappe width (m)
- h, nappe thickness (m)
- p, nappe perimeter (it is assumed that $p = 2 \times (h + L)$) (m)
- d, falling jet height (m)
- V_i, jet speed at impact (m/s)
- V_m, minimum velocity required to entrain air (1.1 m/s according to authors)

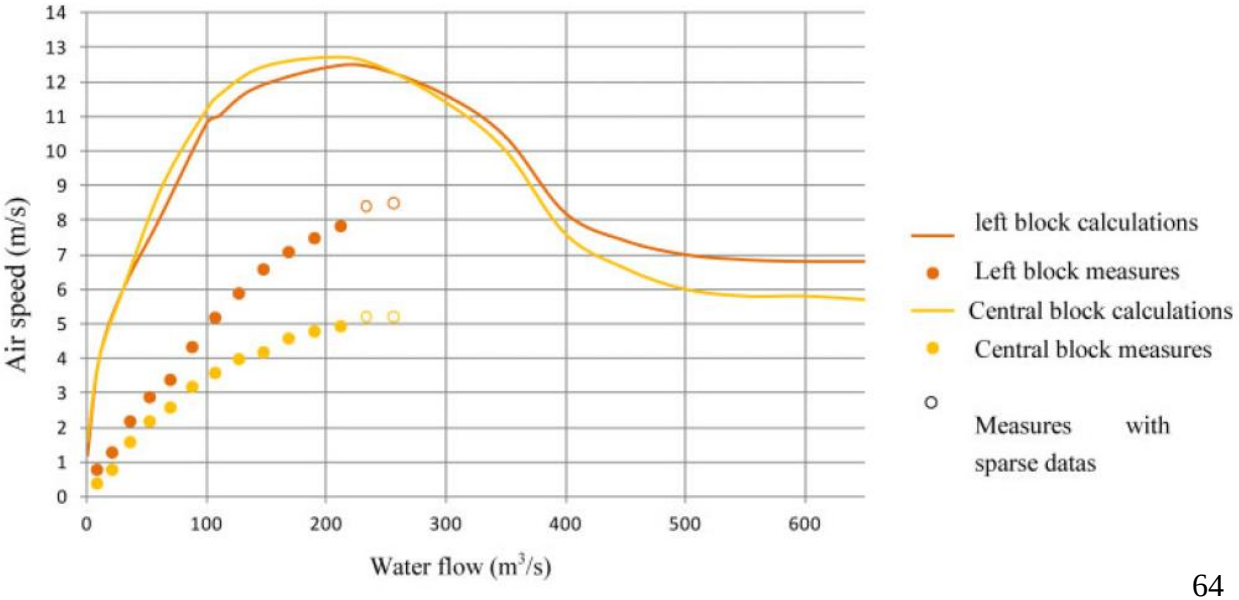
- $\Delta P < 1.5$ m of water (1200 m of air) for a bottom gate
- $\Delta P < 0.3$ m of water (240 m of air) for a flap gate
- $\Delta P < 0.01$ m of water (or 8 m of air) for a measuring weir

6. Labyrinth and PK Weirs hydraulic design and performance

Aeration

- For Malarce PKW: the section of the aeration pipes would have been divided by 2 with the new method





6. Labyrinth and PK Weirs hydraulic design and performance

Energy dissipation

- Design of energy dissipation systems mostly with scale model studies (Bieri et al., 2011; Erpicum et al., 2011; Leite Ribeiro et al., 2011)

- Stepped spillways, flip buckets or stilling basins have been built to date

La Raviège dam PKW – France

Erpicum et al, 2011

6. Labyrinth and PK Weirs hydraulic design and performance

Energy dissipation

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- Stepped spillways, flip buckets or stilling basins have been built to date

Gloriettes dam PKW – France

Bieri et al, 2011

6. Labyrinth and PK Weirs hydraulic design and performance

Energy dissipation

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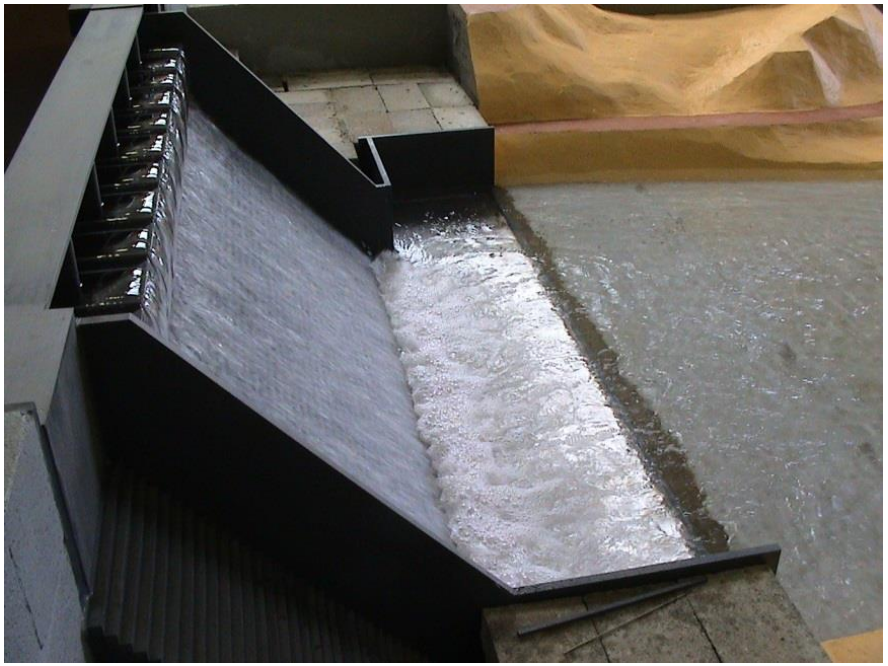


Ouldjet Mellegue dam PKW – Algeria

Erpicum et al, 2011

6. Labyrinth and PK Weirs hydraulic design and performance

Energy dissipation

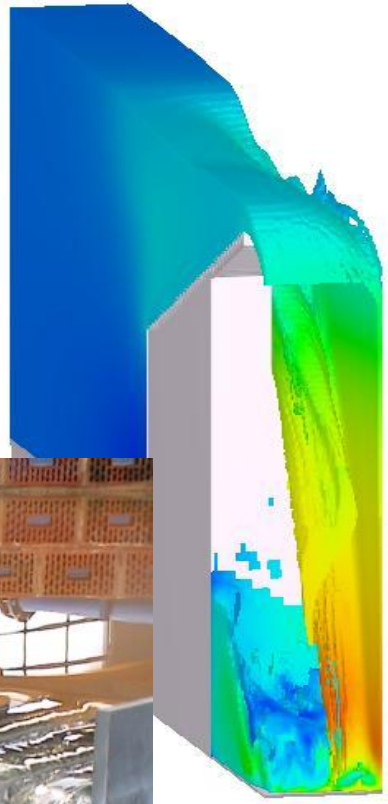
- Design of energy dissipation systems mostly with scale model studies (Bieri et al., 2011; Erpicum et al., 2011; Leite Ribeiro et al., 2011)
- Stepped spillways, flip buckets or stilling basins have been built to date
- Not yet results from prototypes operation
- First systematic experimental studies:
  - Stepped spillways (Ho Ta Khanh et al., 2011; Silvestri et al., 2013)
  - Stilling basin (Truong Chi et al., 2017; Pfister et al., 2017)

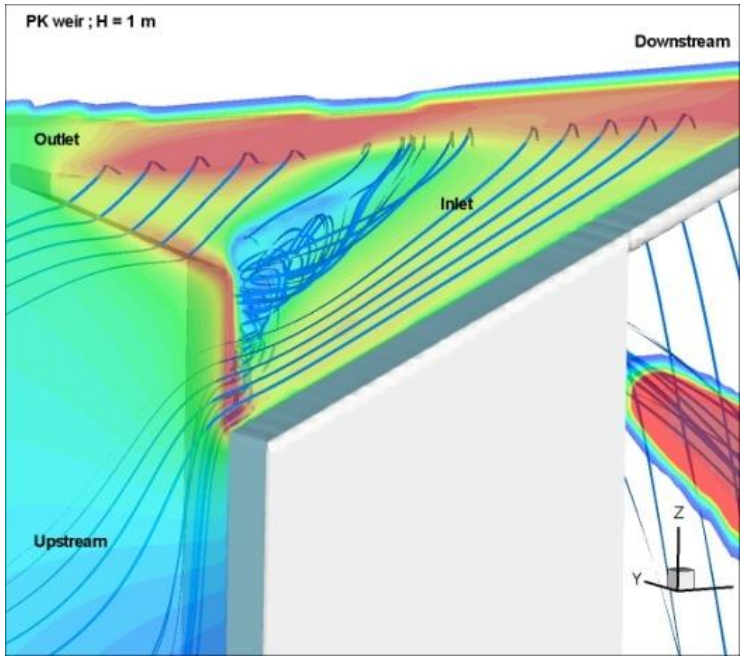
6. Labyrinth and PK Weirs hydraulic design and performance

Modeling of PKWs

In the first years mostly done by means of physical scale model

3D numerical modeling have then prooved to be acurate to discharge capacity estimations





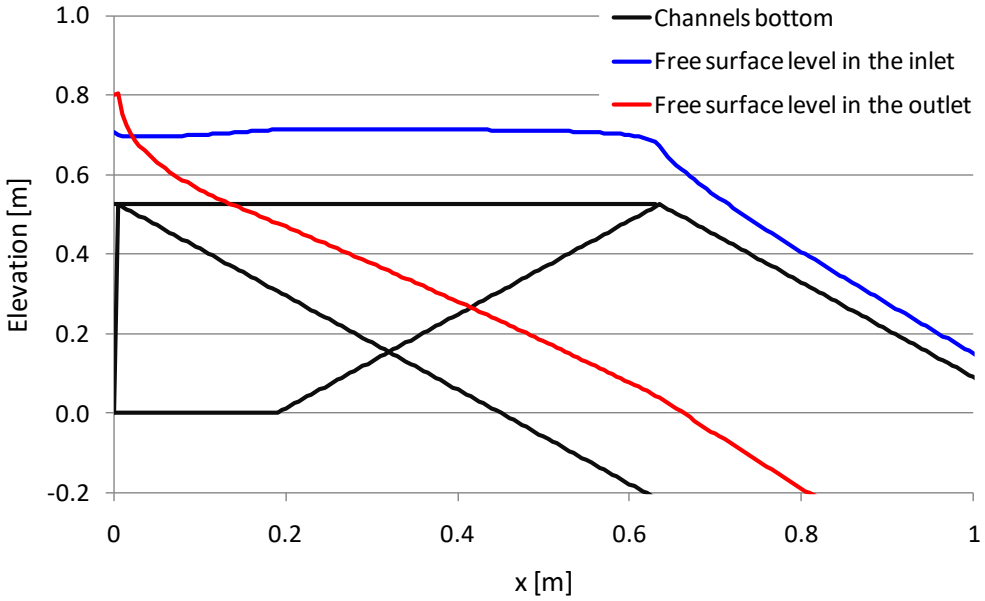
Pralong et al., 2011

Numerical models cannot reproduce all of the flow specifities (approach flow conditions, energy dissipation…)

6. Labyrinth and PK Weirs hydraulic design and performance

Modeling of PKWs

Simplified 1D model can be used for quick early design

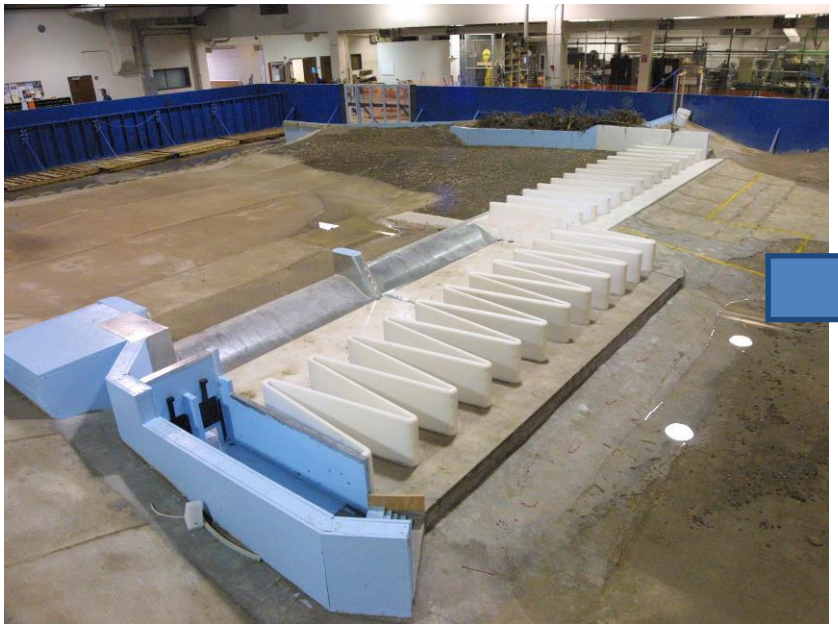


http://www.pk-weirs.ulg.ac.be

6. Labyrinth and PK Weirs hydraulic design and performance

Scale effects

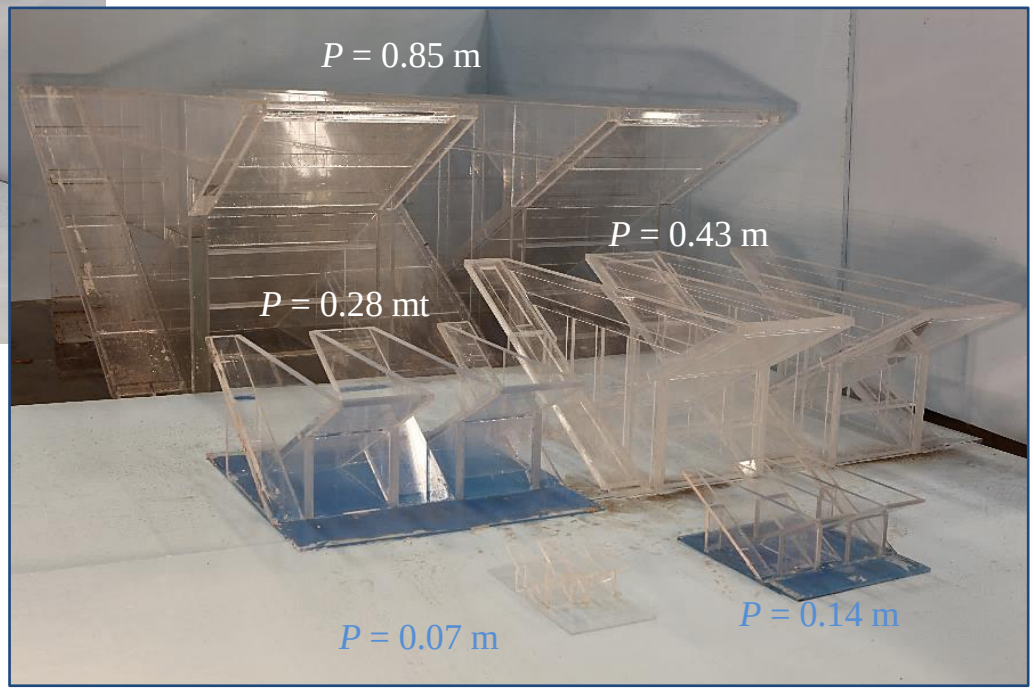
Physical model testing



Lake Brazos Dam Model Study (1:15 Scale)
Design: Freese and Nichols, Inc.
Model photo courtesy of Dr. Blake Tullis, Utah State University
Prototype photo: http://kxxv.images.worldnow.com/images/13978660_G.jpg

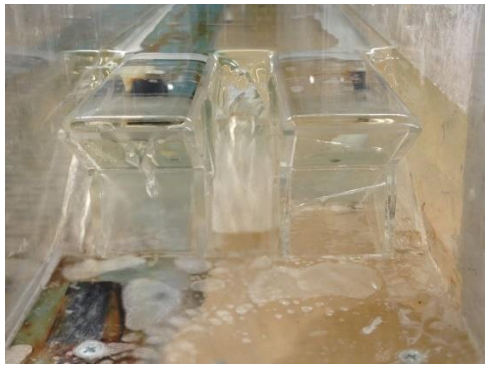
6. Labyrinth and PK Weirs hydraulic design and performance

PKW models

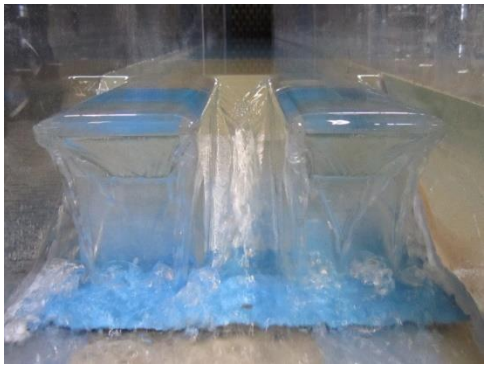


6. Labyrinth and PK Weirs hydraulic design and performance

Nappe behavior H_T/P=0.05



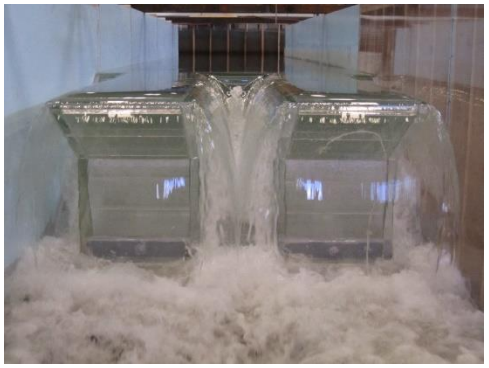
P = 0.07 m



P = 0.14 m

P = 0.28 m

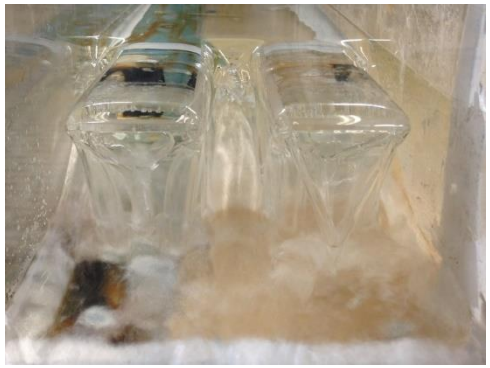
P = 0.43 m



P = 0.85 m

6. Labyrinth and PK Weirs hydraulic design and performance

Nappe behavior H_T/P=0.20



P = 0.07 m

P = 0.14 m

P = 0.28 m

P= 0.43 m

P = 0.85 m

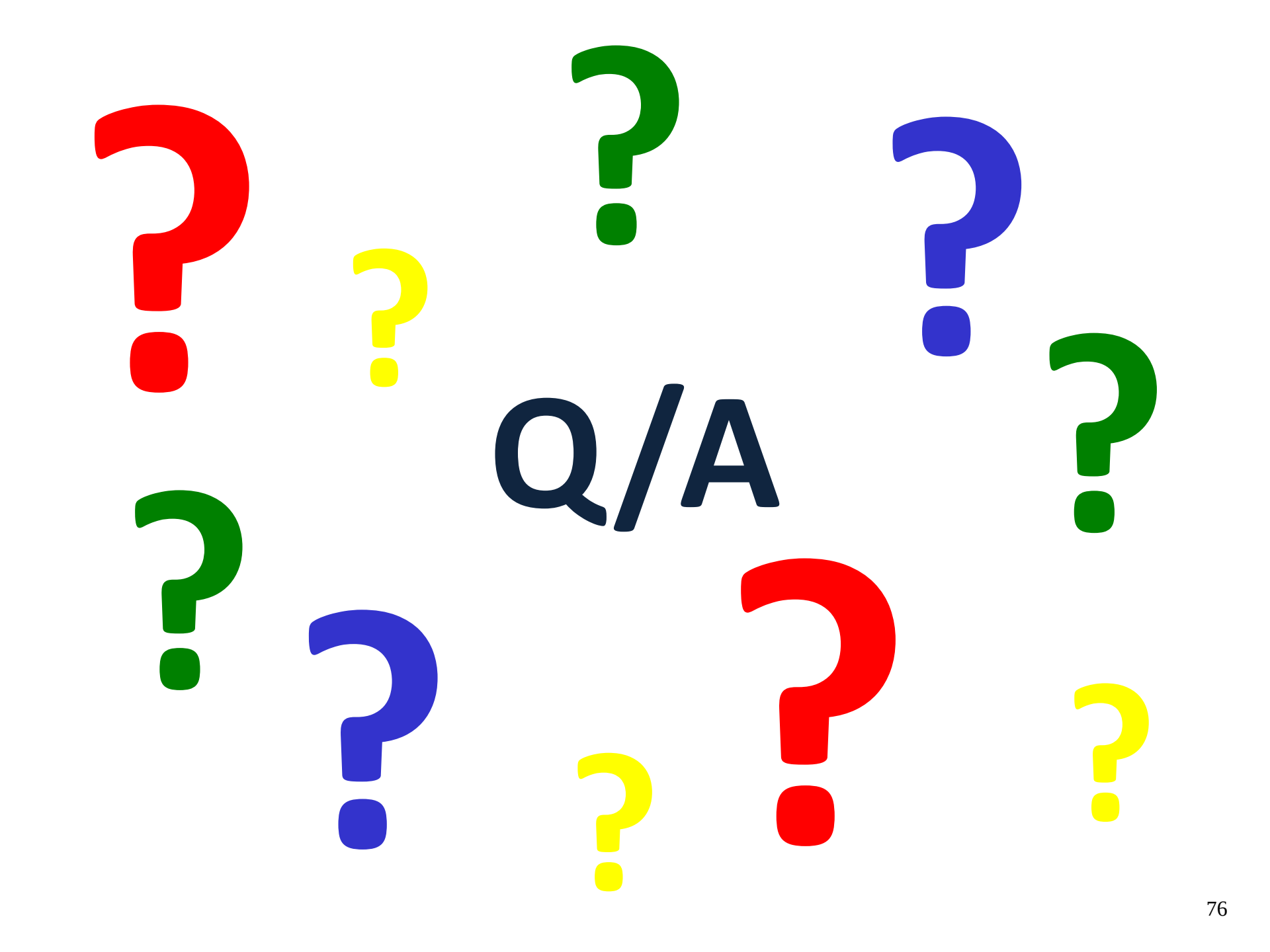
6. Labyrinth and PK Weirs hydraulic design and performance

Scale effects

Recommended minimum heads

Reference	Minimum Head (mm)	Weir Type	Shape/Parameter	Crest Radius, *R* (mm)
Erpicum et al. (2016)	30	Piano Key	Head-discharge	
Erpicum et al. (2016)	60	Piano Key	Nappe trajectory	
Ettema (2000)	25	Overflow	Rating curve	
Johnson (1996)	1-28	Linear	Flat-top	
Leite Ribeiro et al. (2012)	50	Piano Key	Nappe trajectory	
Pfister et al. (2012)	30	Piano Key	Head-discharge	
Pfister et al. (2013)	30	Piano Key	Half-cylindrical	5-300
Young & Tullis (2017)	8-16	Labyrinth	Half-round	5-57
Young & Tullis (2017)	7-10	Labyrinth	Quarter-round	5-57

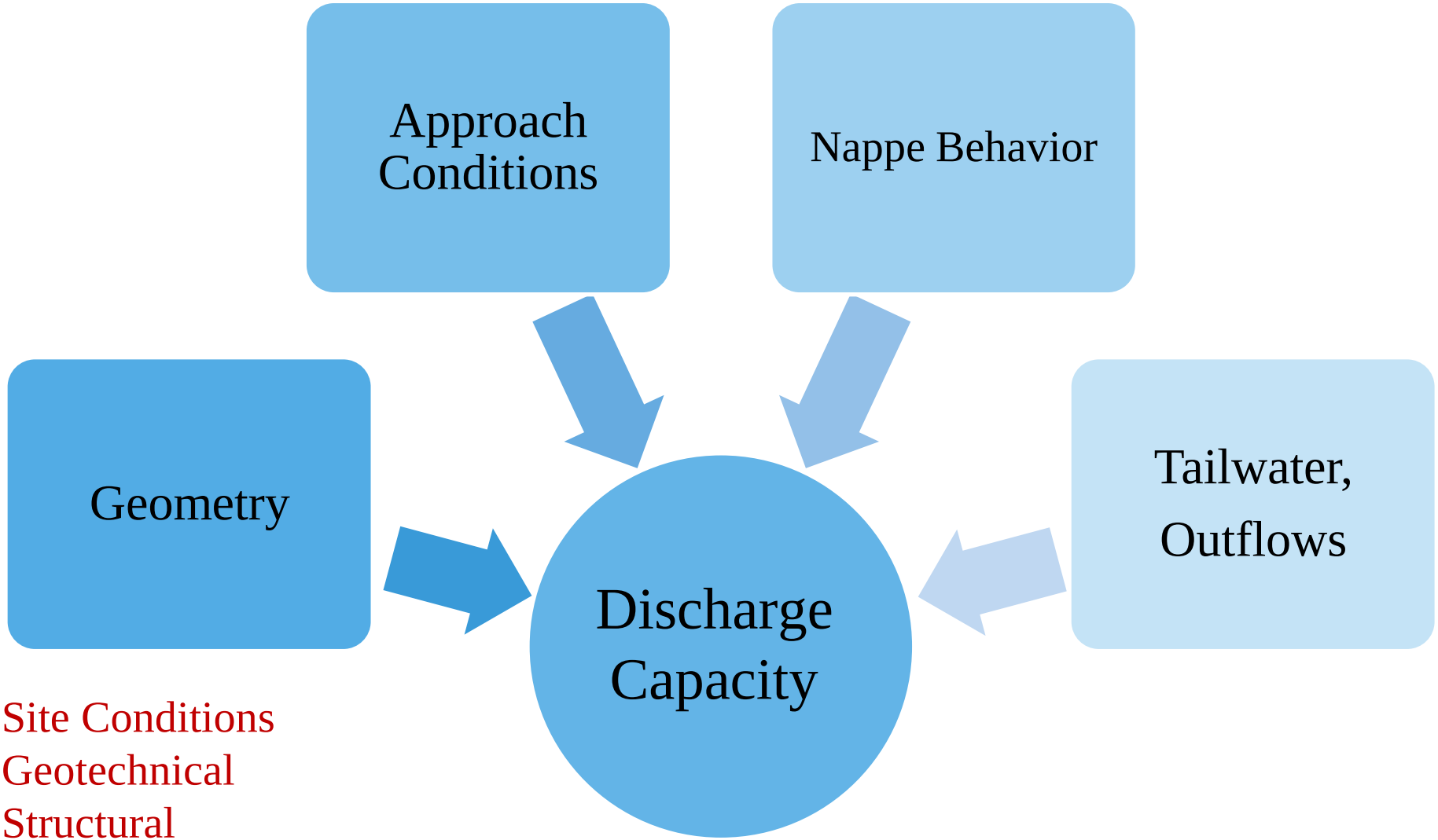
Range of 30-60 mm
(1.2-2.4 in)


Q/A

Design Overview

7. Hydraulic Design Overview

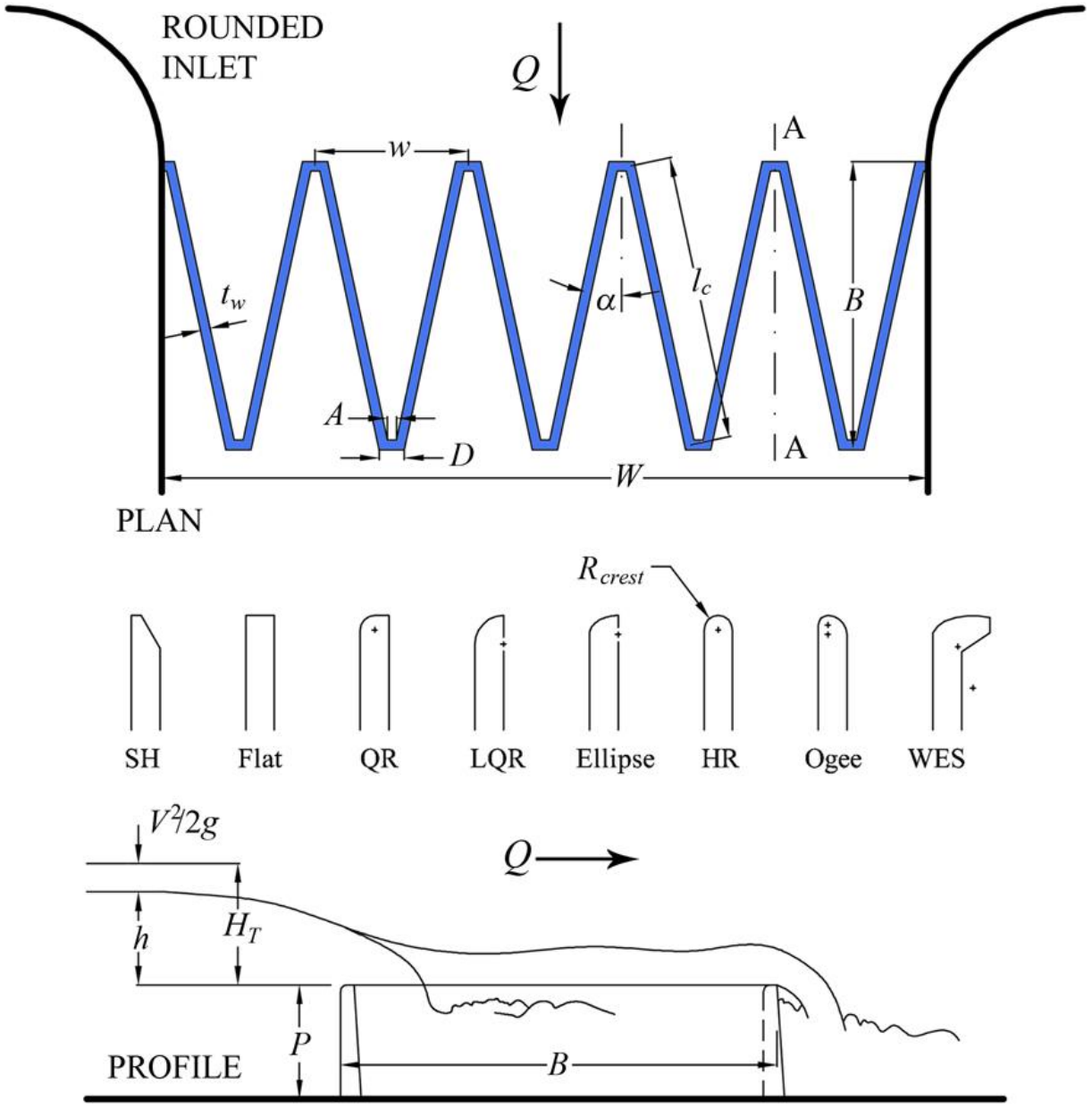
Spillway Lifecycle – Entire System/Performance



Site Conditions
Geotechnical
Structural
Constructability
Economics

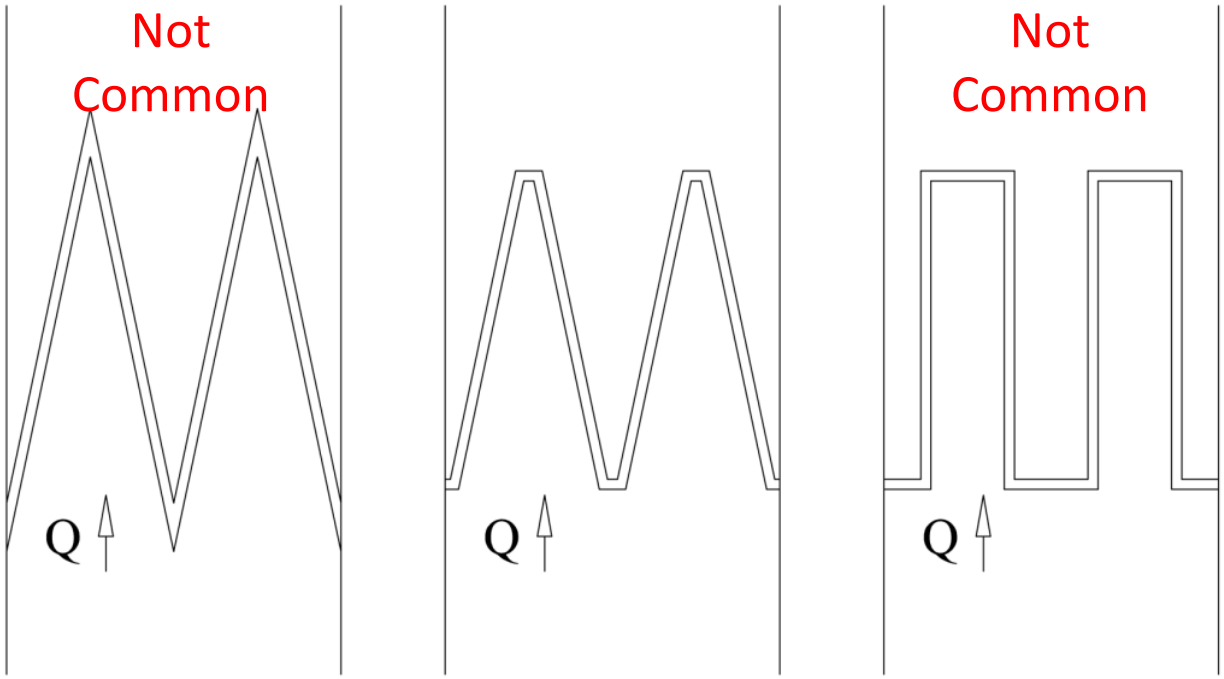
Moderated by Project Objectives

7. Design Overview - Labyrinth



6. Labyrinth and PK Weirs hydraulic design and performance

Apex/cycle details

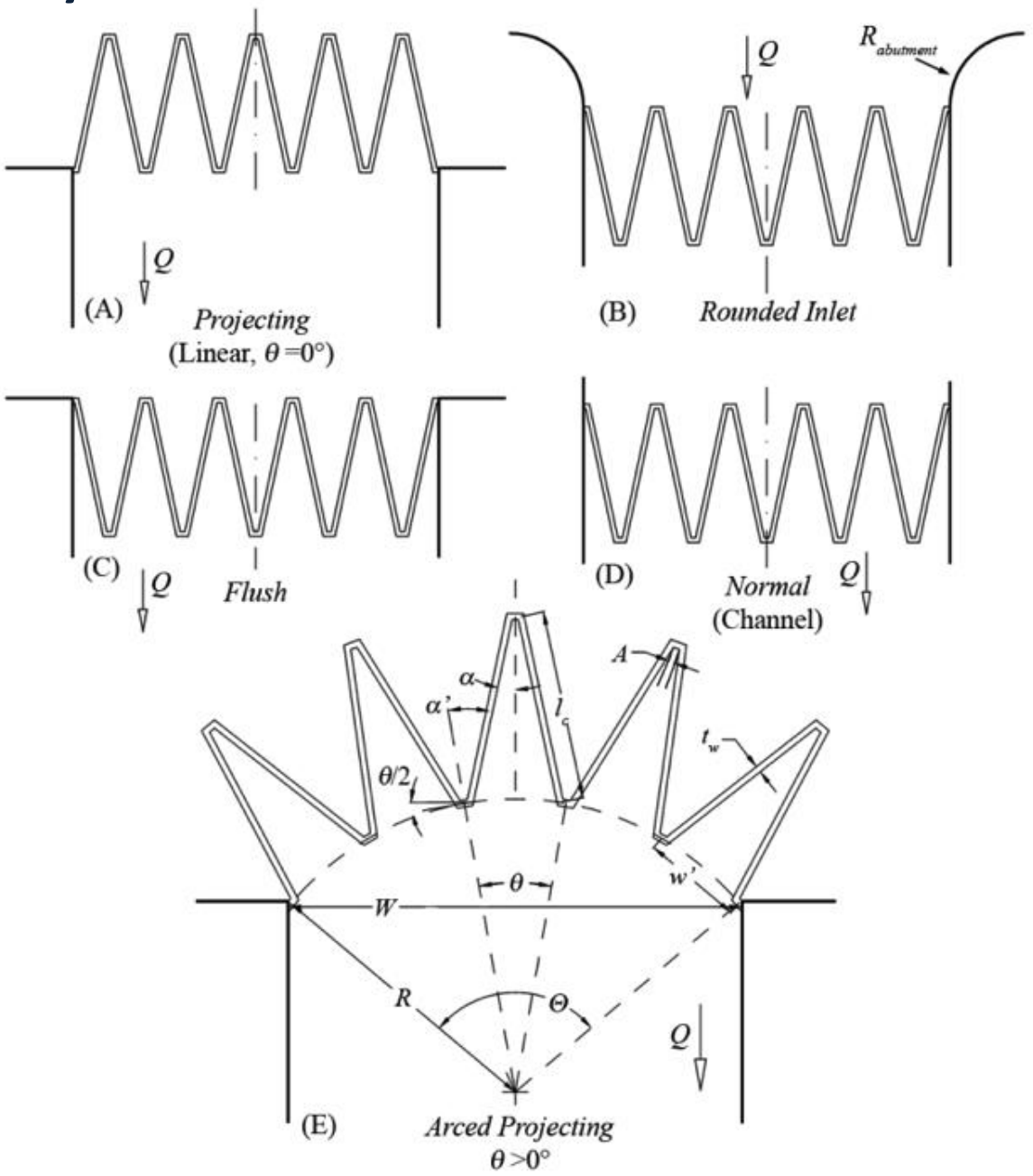


triangular trapezoidal rectangular

- Difficult form work
- Debris impact concerns
- Ineffective length at apex

- Less efficient hydraulically than a PKW

7. Design Overview - Labyrinth



7. Design Overview - Labyrinth

Discharge Coefficients

H_T/P

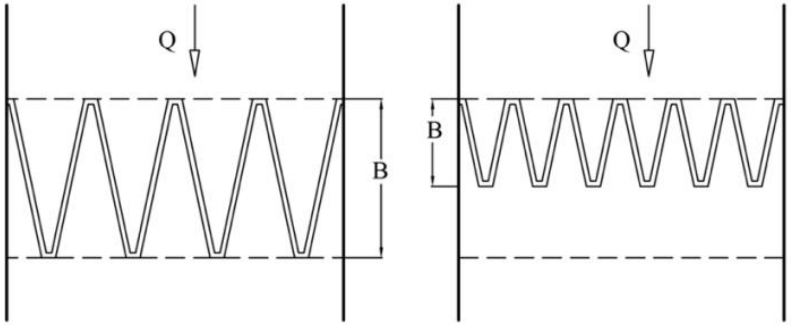
Site-specific Conditions

Cycle Geometry

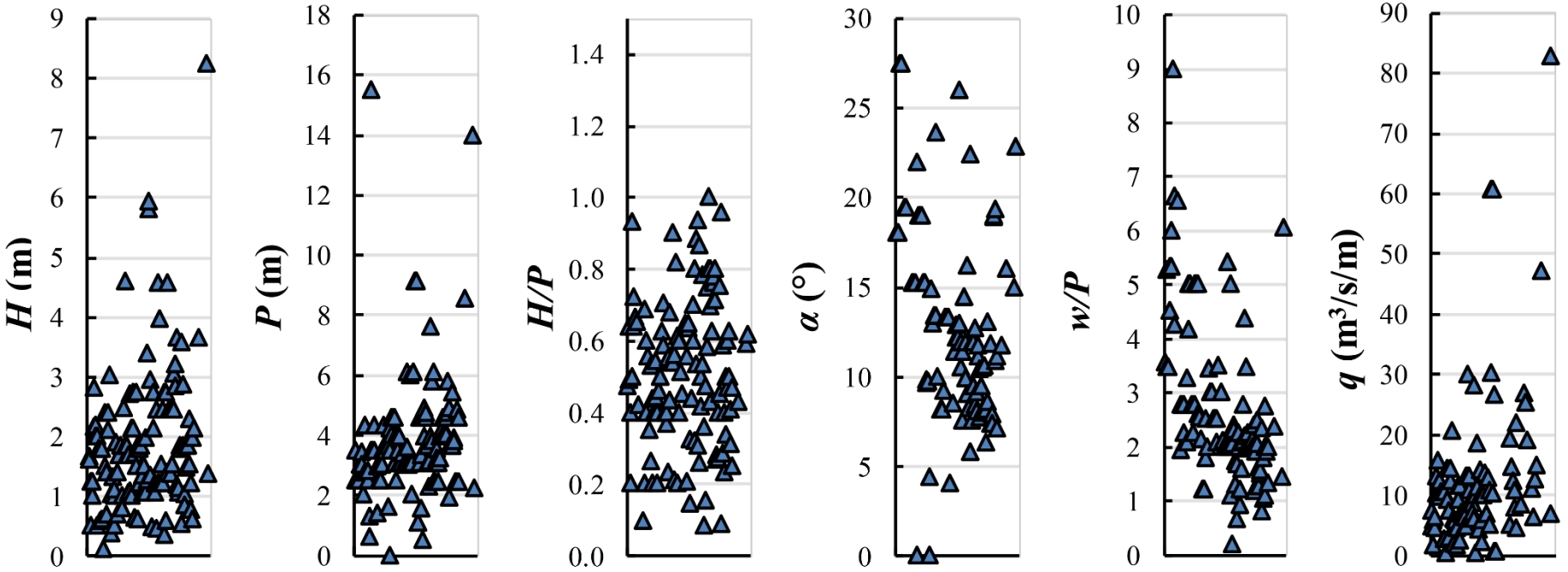
Valid Range
Ratio distortion (P and DS cycle capacity)

First-order approximation

Layout



7. Design Overview - Labyrinth



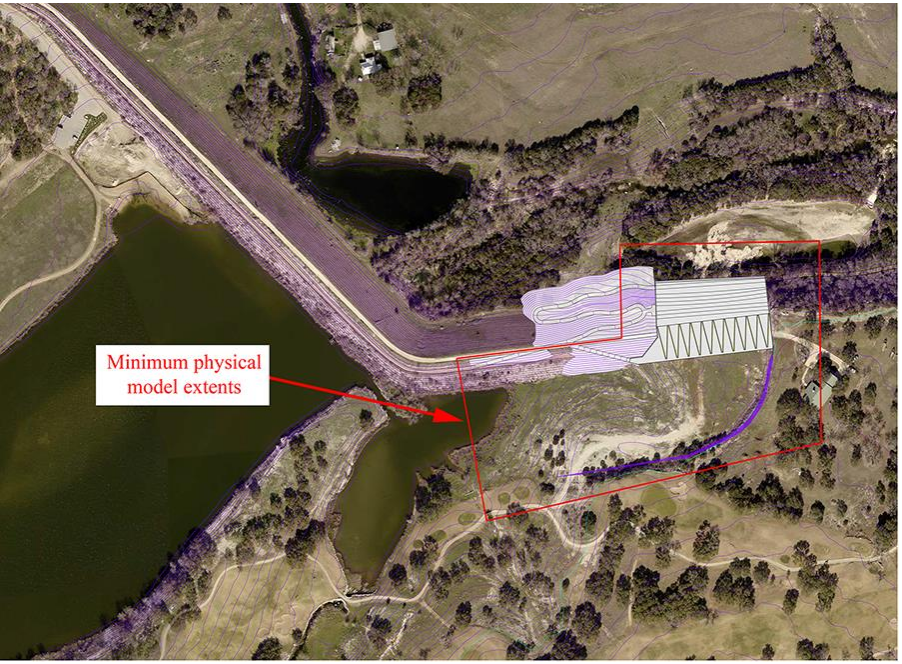
7. Design overview - Labyrinth

Parameter	Min	Mean	Max
Q (cms)	14	1,407	24,609
H (m)	0.1	1.9	8.3
P (m)	0.5	3.9	15.5
H/P	0.1	0.5	1.0
α (°)	0.0	12.9	27.5
W (m)	2.1	54.4	380.4
w (m)	0.9	10.5	95.1
L_c (m)	7.9	197.7	1441.2
N	1.0	5.8	45.0
B (m)	5.8	17.5	37.6
L_c/W	0.7	3.8	15.6
w/P	0.2	2.9	13.0

Reference: Crookston et al. 2019

7. Design Overview - Labyrinth

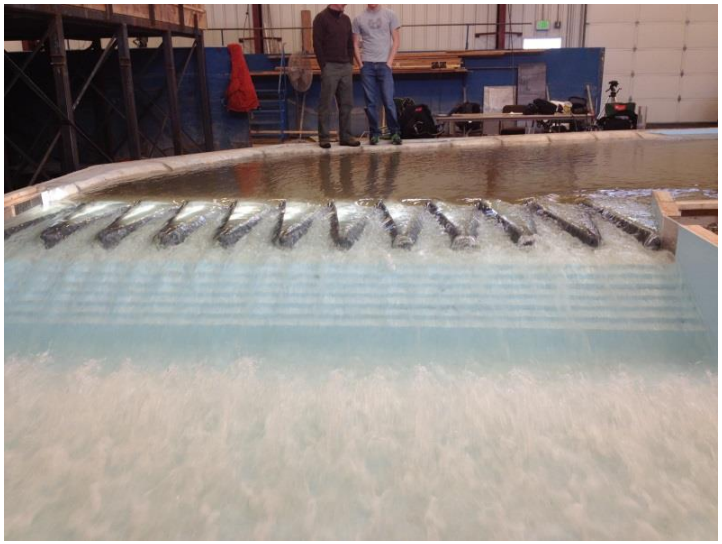
Configurations/Abutments



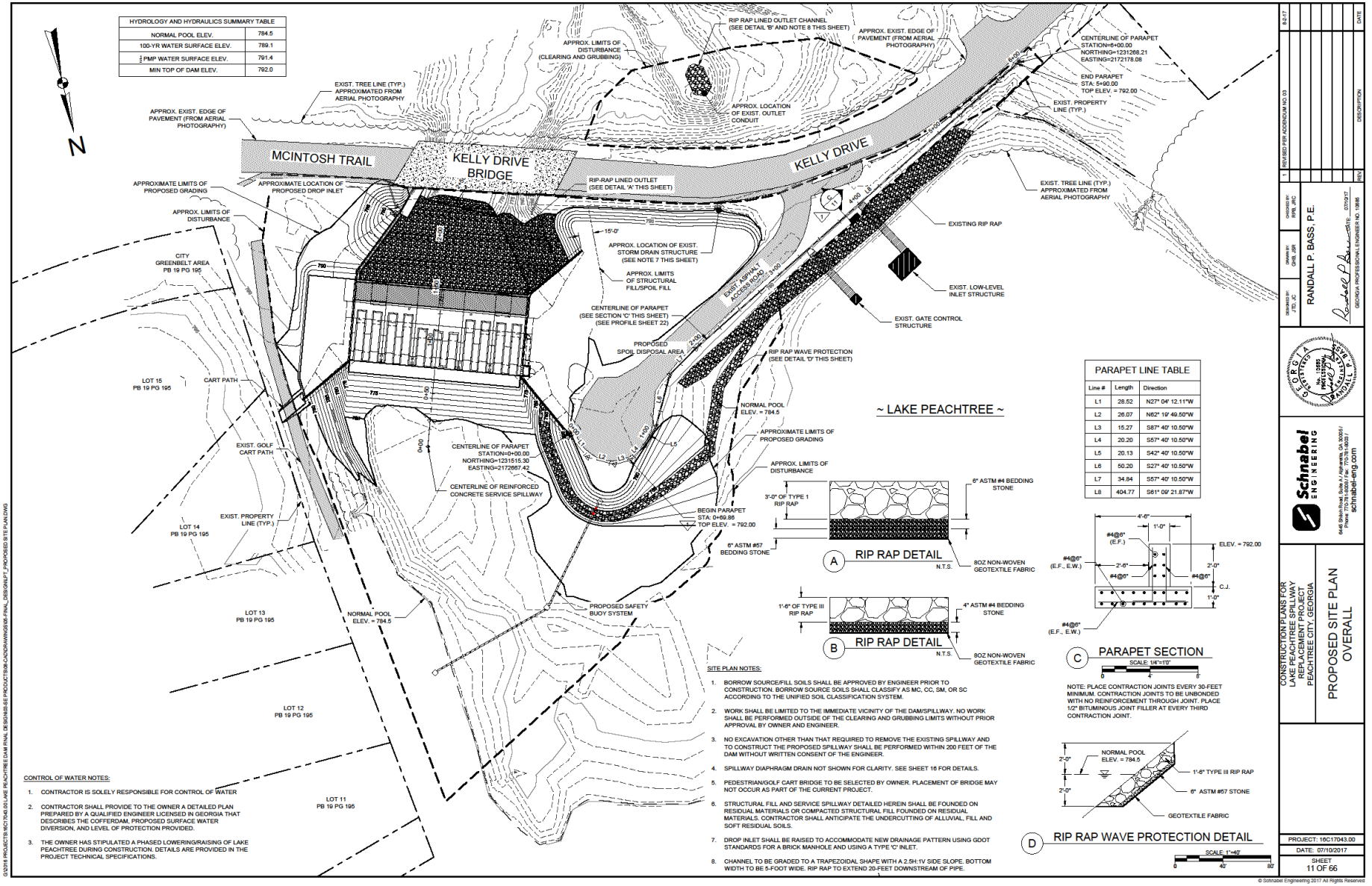
FREESE AND NICHOLS

7. Design Overview - Labyrinth

Configurations/Abutments

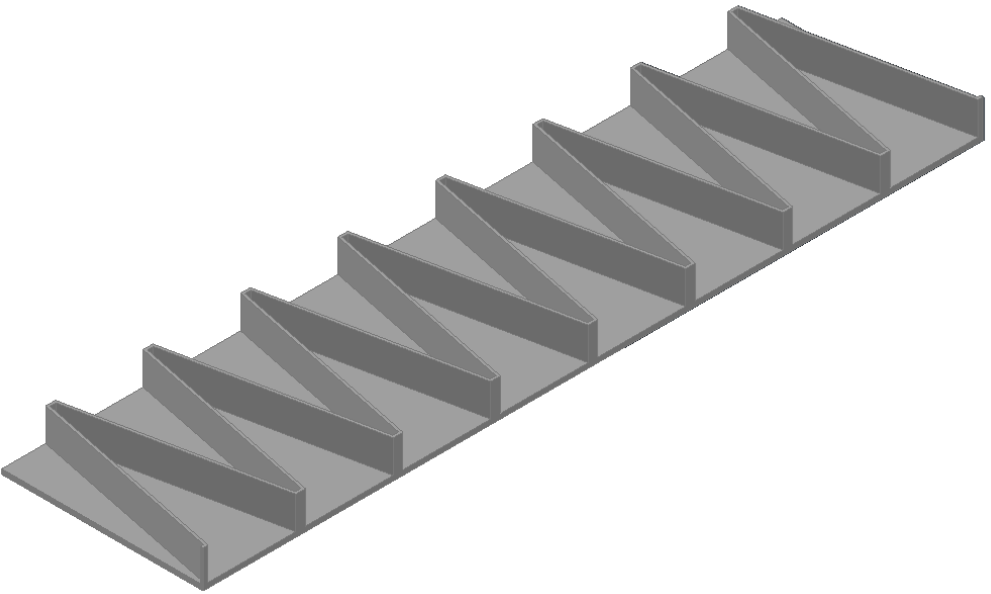


7. Design Overview - Labyrinth



7. Design Overview - Labyrinth

7. Design overview - Labyrinth



- P = 6.1 m
- W = 18.3 m (W/P = 3)
- α = 12°
- H = 4.9 m (H/P = 0.8)
- n = 8
- Q = 6,570 cms

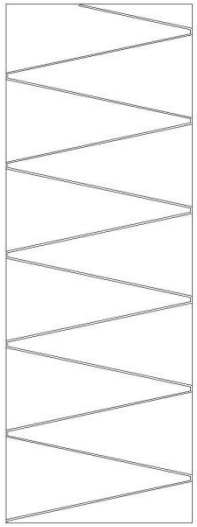
7. Design overview - Labyrinth

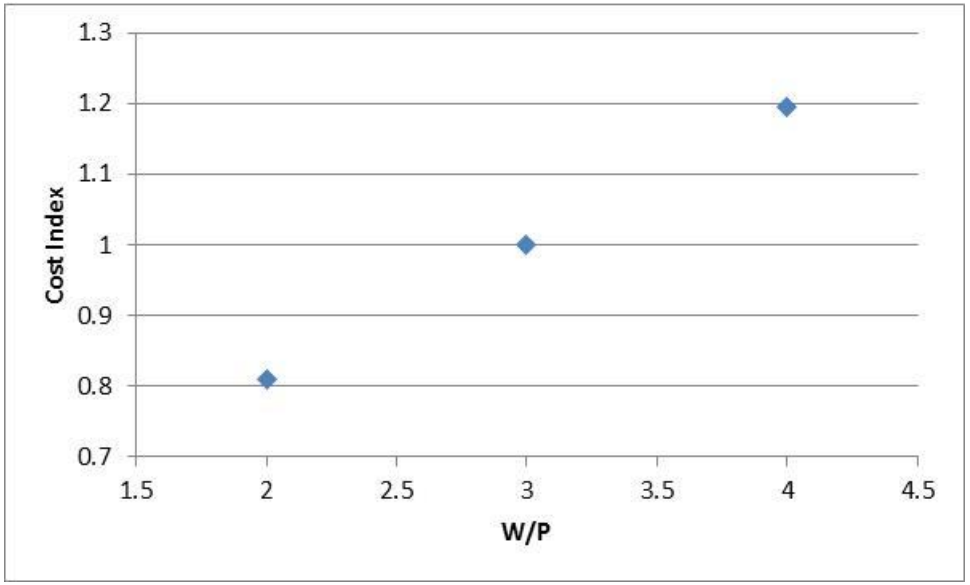
W/P Ratio: Adjust W

W/P = 2

W/P = 3

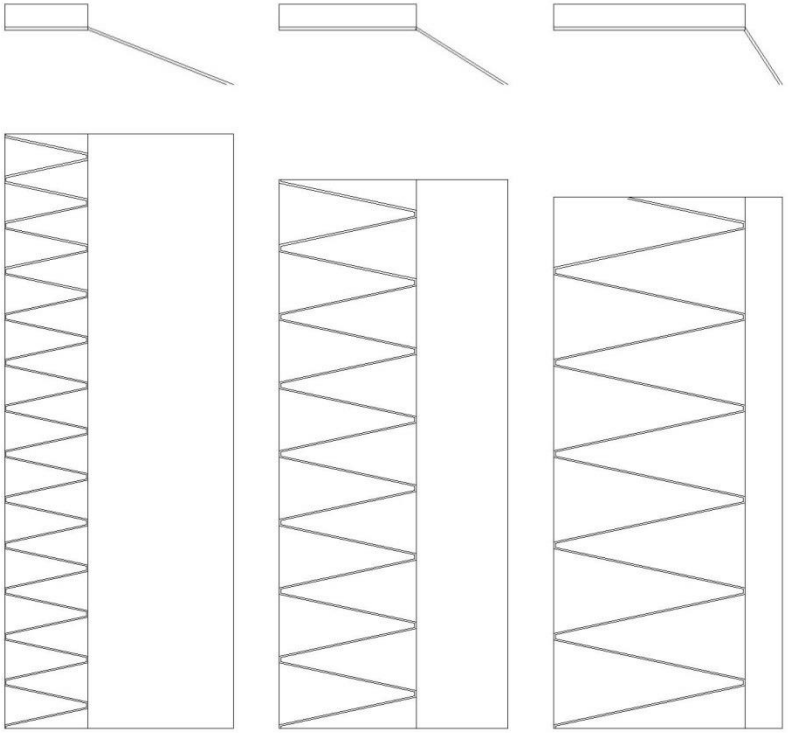
W/P = 4

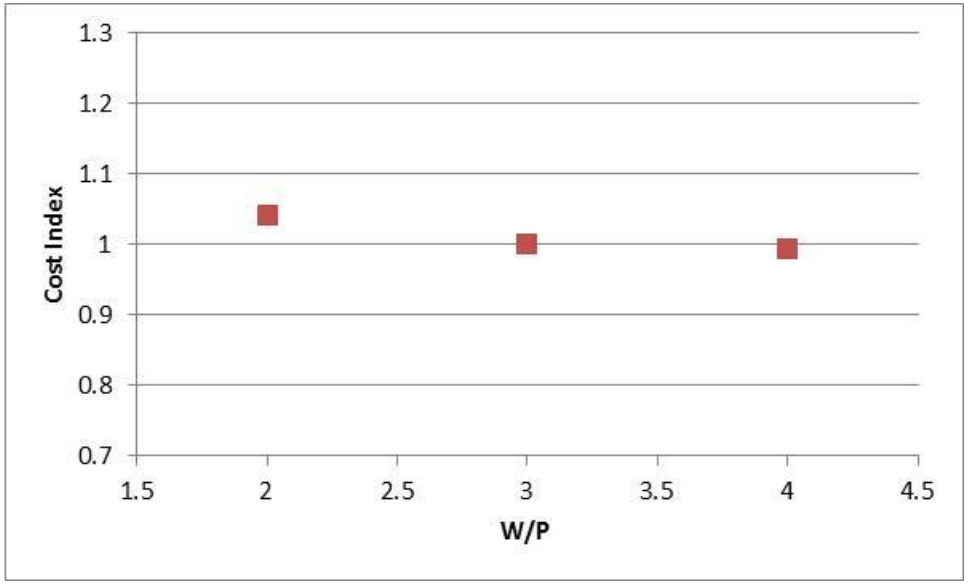




7. Design overview - Labyrinth

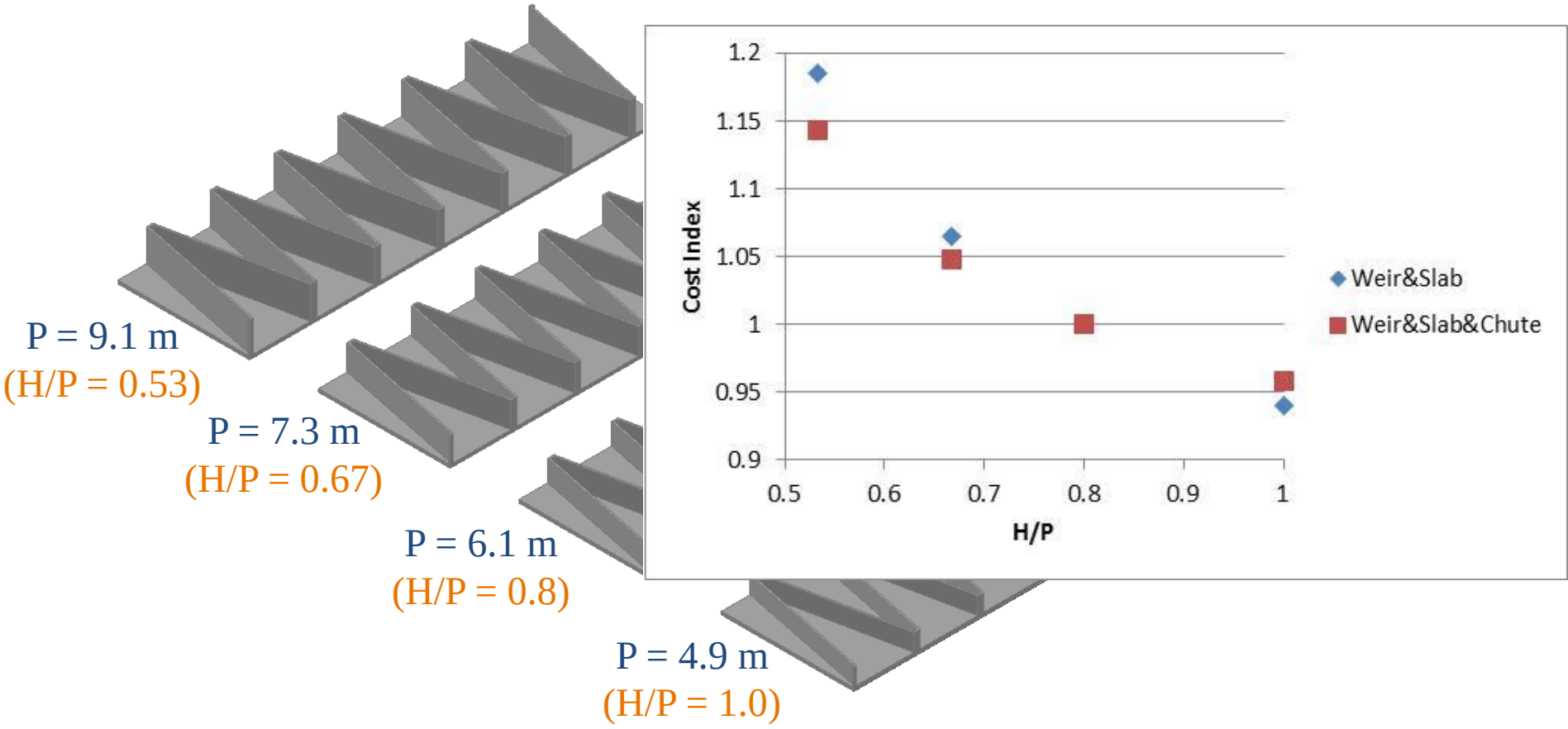
W/P Ratio: Adjust W





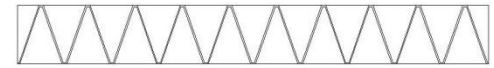
7. Design overview - Labyrinth

H/P Ratio: Adjust P

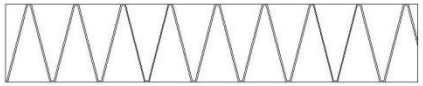


7. Design overview - Labyrinth

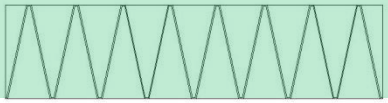
Side wall angle



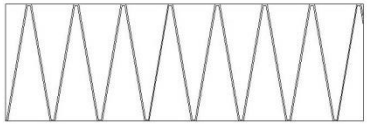
α = 20°



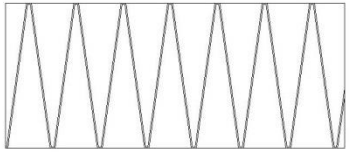
α = 15°



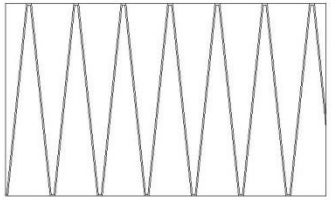
α = 12°



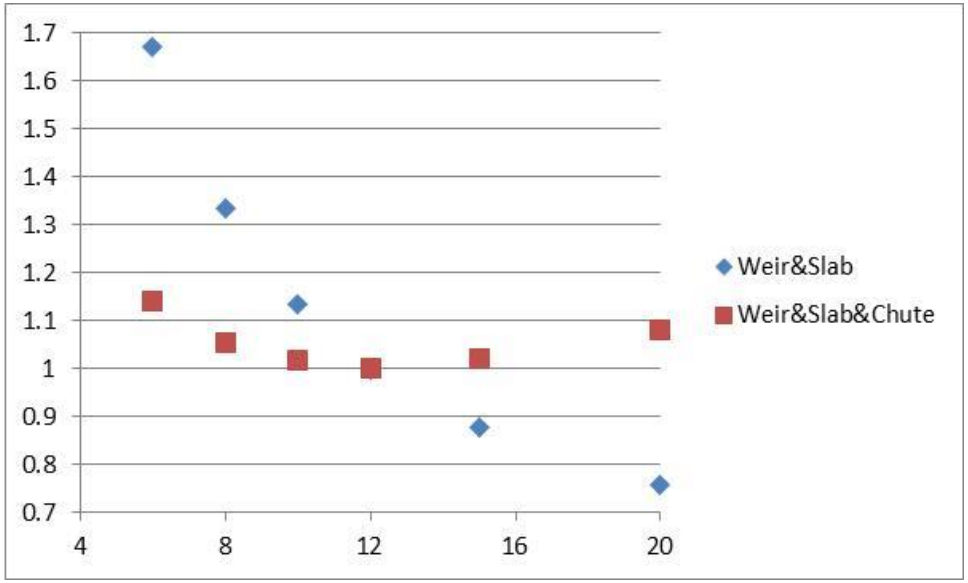
α = 10°



α = 8°



α = 6°



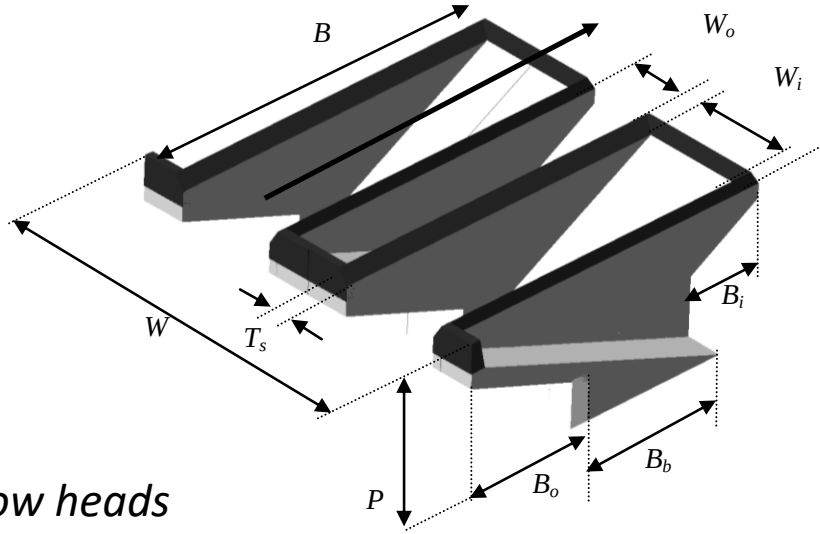
7. Design overview - Labyrinth

Labyrinth spillways

ID	Project Name	State	No. of Cycles —n	Weir Height — P (ft)	Total Spillway Width — nxW (ft)	α	W/P	Max H/P
1	Roy F. Varner Reservoir Dam	GA	8	10	160	13.9	2.0	0.70
2	Dog River - original	GA	8	15	240	13.0	2.0	0.60
3	Lake Sovereign Dam	GA	2	5	50	12.6	5.0	0.80
4	Concourse Lake Dam	VA	2	10.9	24	10.5	1.1	0.32
5	Accord Pond Dam	MA	2	1.7	7	12.5	2.1	0.88
6	Hollis G Lathem Reservoir Dam	GA	3	15	90	12.6	2.0	0.53
7	Unicoi State Park Dam	GA	3	16	115	10.3	2.4	0.94
8	Whitewater Creek Dam	GA	12	13	360	13.3	2.3	0.31
9	Yahoola Creek Dam	GA	5	15	130	8.9	1.7	0.87
10	DRA Detention Structure[1]	GA	4	13.5	81	16.7	1.5	0.30
10	DRA Detention Structure[2]	GA	4	15.5	81	16.7	1.3	0.13
11	Waterford Lake Dam	NC	3	10	60	8.6	2.0	0.50
12	Lake Upchurch Dam	NC	6	12	120	8.6	1.7	0.42
13	Capital City Country Club Dam	GA	1	10.2	23	23.8	2.3	0.78
14	Pine Lake Dam	GA	2	7.5	30	6	2.0	0.53
15	Pye Lake Dam	GA	3	13	36	7.0	0.9	0.08
16	Dog River – raised	GA	8	25	240	12.2	1.2	0.36
17	Huntington Hills Dam	SC	3	12.5	78	9.7	2.1	0.15
18	Linville Land Harbor Dam[3]	NC	4	15	120	12.2	2.0	0.60
18	Linville Land Harbor Dam[3]	NC	4	19	120	12.2	1.6	0.47
19	Pine Run Dam[1]	PA	0.5	6	11	9.1	3.7	0.75
19	Pine Run Dam[2]	PA	3.5	7.9	77	9.1	2.8	0.33
20	Lake Townsend Dam[1]	NC	2	20	84	12.8	2.1	0.75
20	Lake Townsend Dam[2]	NC	5	21	210	12.8	2.0	0.67
21	Leaser Lake Dam[1]	PA	0.5	12	12.5	14.0	2.1	0.67
21	Leaser Lake Dam[2]	PA	1.5	13.5	37.5	12.3	1.9	0.79
22	Greystone Development Dam	GA	1	8	35	8.7	4.4	0.59
23	Indian Run Dam[1]	PA	0.5	9	16	10.9	3.6	0.89
23	Indian Run Dam[2]	PA	2	11.5	64	10.9	2.8	0.48
24	Upper Norton Dam	VA	3	10.5	63	12.1	2.0	0.43
25	Upper Owl Creek Dam[1]	PA	0.5	8.5	9.75	10.1	2.3	0.94
25	Upper Owl Creek Dam[2]	PA	2.5	11	48.75	10.1	1.8	0.50
26	Lake Natalie Dam	GA	4	10.5	84	9.8	2.0	0.76
27	Charter Colony, LTC 20/25[1]	VA	0.3	10.7	3.54	7.0	1.1	0.36
27	Charter Colony, LTC 20/25[2]	VA	1.7	13.5	20.06	6.2	0.9	0.07
28	Kauffman Dam[1]	PA	0.5	5.5	9	8.6	3.3	0.91
28	Kauffman Dam[2]	PA	4.5	8	81	8.0	2.3	0.31
29	South River No. 29	GA	3	13	78	10.5	2.0	0.77
30	Tired Creek Dam	GA	3	13	78	9.0	2.0	0.72
31	Beaver Creek Dam	PA	30	6.25	300	10.1	1.6	0.96
32	Charter Colony, LTC 30	VA	4	12	72	8.8	1.5	0.28
33	Charter Colony, LTC 5	VA	2	12	37	8.2	1.5	0.23
34	Dees Pond Dam	GA	2	12	56	19.2	2.3	0.40
35	Ford Road Dam	GA	1	17.75	14	18.2	0.8	0.34
36	Hard Labor Creek Dam	GA	4	15	128	7.9	2.1	0.80
37	Lake Carroll Dam	GA	4	12	86	10.6	1.8	0.50
38	Long Island Drive	GA	1	12.75	13	18.4	1.0	0.47
39	Minsi Lake Dam[1]	PA	5	10	125	11.4	2.5	0.75
39	Minsi Lake Dam[2]	PA	5	12.5	125	11.8	2.0	0.44
40	Mountain Cove Dam	GA	1	8	22	7.7	2.8	0.63
41	Pilot Mountain State Park Dam	NC	2	8	17.5	8.8	1.1	0.50
42	South River No. 4	GA	2	13.5	54	10.3	2.0	0.78
43	Upper Dam	ME	4	14	112	10.4	2.0	0.28
44	Yellow River No. 3	GA	2	13	62	7.7	2.4	0.27

Reference: Paxson et al, ICOLD 2013.

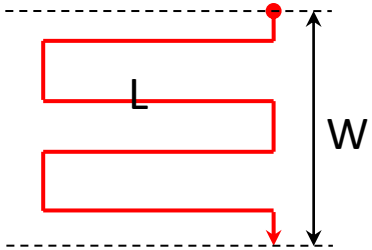
7. Design Overview – PKW



PKW main features

Main geometrical parameters :

- Ratio L/W



Efficiency for low heads

- Height H/P — *Better approach flow "supply" in inlet keys*
  Less saturation in outlet keys
- Key width W_i / W_o — *Inlet to be large enough for lateral weir "supply"*
  and flow saturation in outlet keys
- Sidewall thickness T_s — *Structural design data => can affect up to 20%*
  between steel / concrete

7. Design Overview – PKW General Design

- Rehabilitation Or New dam ?

- Hydraulic optimization vs Technico-economical optimization (Machiels, 2012)

- Existing dams
  - Available space on top of dam
  - **Cost** - Reservoir level during works ⇔ Plant operation
  - **Dam Safety** - Flood passage during works

7. Design Overview – PKW General Design

1. Rehabilitation issues :

⇒ (very) old existing dams

⇒ Flood passage safety issues

- Typically : 1 – 1,5 m head
- Limited available space

- Ratio L/W = 6 to 7
- Height P > 4-5 m
- Ratio key width W_i / W_o >= 1,3
- Sidewall thickness : 20/35-40 cm (variable)

=> q_s > 8 – 10 m³/s for H = 1 m

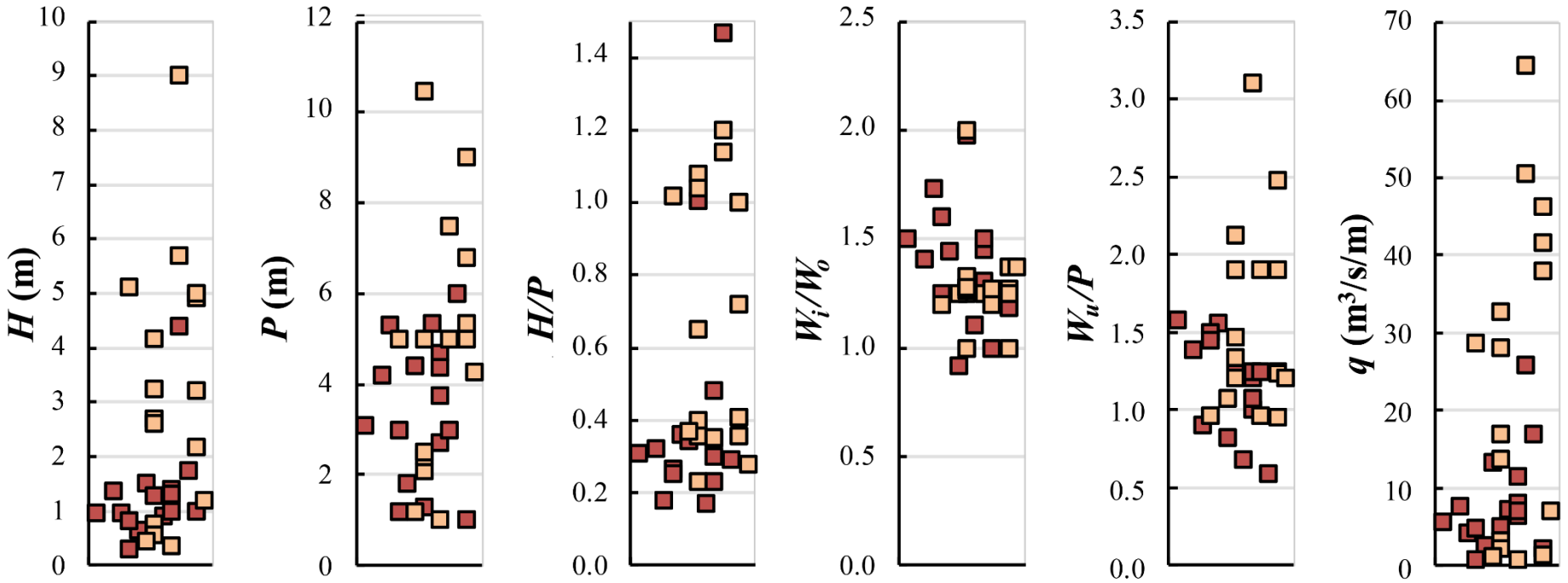
2. New dam :

- Variable head
- Larger available space

- Ratio L/W = 4 to 5
- Height P > 4-10 m
- Ratio key width W_i / W_o >= 1
- Sidewall thickness : 20/35-40 cm (variable)

=> q_s > 4 – 7 m³/s for H = 1 m

7. Design Overview – PKW



7. Design Overview – PKW

	Rehabilitation			New			Overall	
Parameter	**Min**	**Mean**	**Max**	**Min**	**Mean**	**Max**	**Min**	**Mean**
Q (cms)	**4.35**	428.50	4,288	13	5,250	**9,700**	4.35	1,608
H (m)	**0.3**	1.34	4.4	0.65	4.5	**9**	0.3	2.2
P (m)	**1**	3.41	9	1.8	5.45	**10.45**	1	4.1
H/P	**0.16**	0.46	**1.47**	0.36	0.84	1.2	0.16	0.55
W (m)	**3.4**	31.11	103	4.8	135.5	**301.8**	3.4	56.2
W_u (m)	**1.24**	3.82	8.60	2.8	7.97	**15.3**	1.24	5.07
L_c (m)	**13.5**	160.34	650	22	721.37	**1704**	13.5	300.1
N	**2**	9.71	24	8	20.8	**60**	2	14
B (m)	**2.16**	8.97	21.6	5.1	16.11	**30.68**	2.16	11.11
L_c/W	**2.55**	4.93	**8.24**	4.55	5.14	5.96	2.55	5.19
W_u/P	**0.59**	1.28	**3.10**	0.96	1.53	1.9	0.59	1.40
W_i/W_o	**0.92**	1.36	**2.00**	1.00	1.26	1.44	0.92	1.32

Note: Bold text identifies the min and max value between rehabilitations and new projects. Overall refers to the entire set of projects regardless of type.

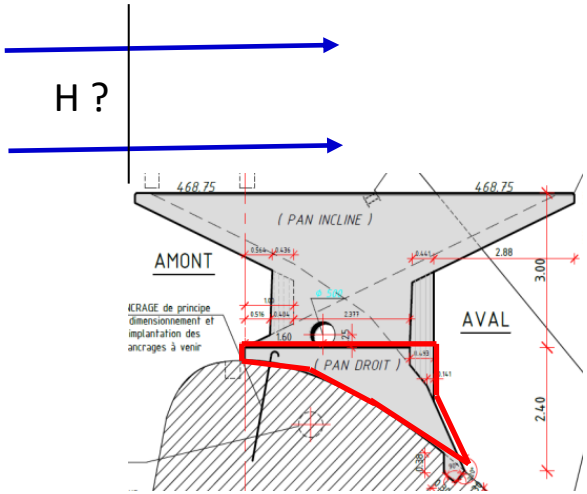
7. Design Overview – PKW Cost

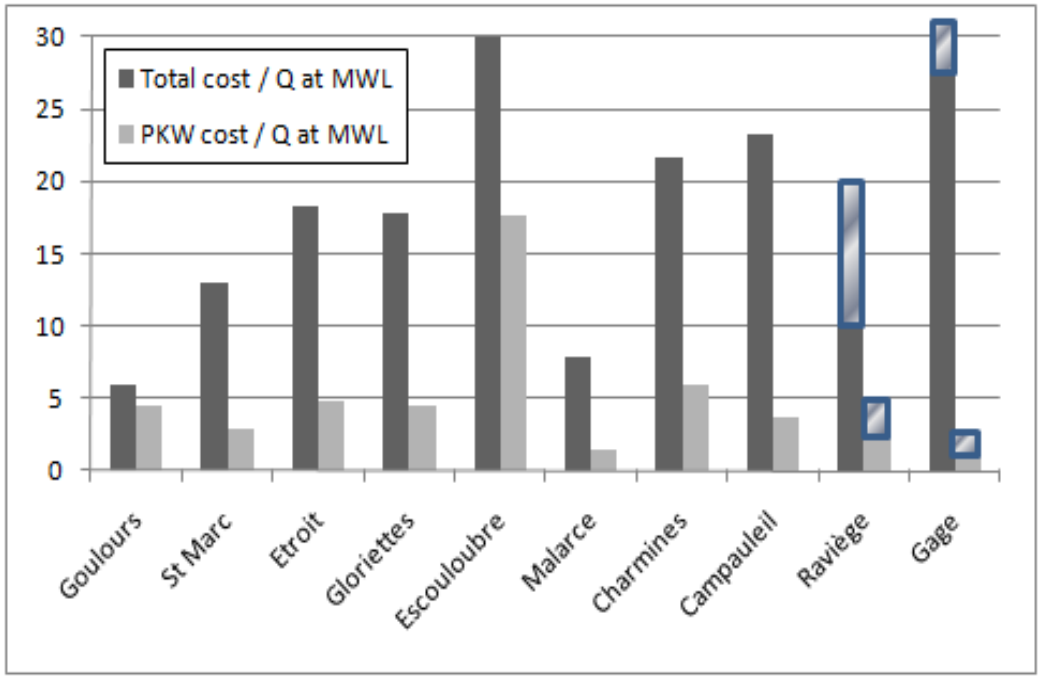
Comparison with other spillway type solutions studied:

- (Flap) gate : usually more expensive by 40% + maintenance + reliability operation
- Fuse gates can be in competition with PKW but some Owners might be reluctant

- PKW cost ratio quite constant
  Total cost ratio varies from 5 to 20 k€/(m³/s)
  Weight of local considerations (topography, access, installations..)

- Overall project cost difficult to assess
  - ✓ Dam stability issues
  - ✓ Installation / access / Demolition
  - ✓ Dissipation structures

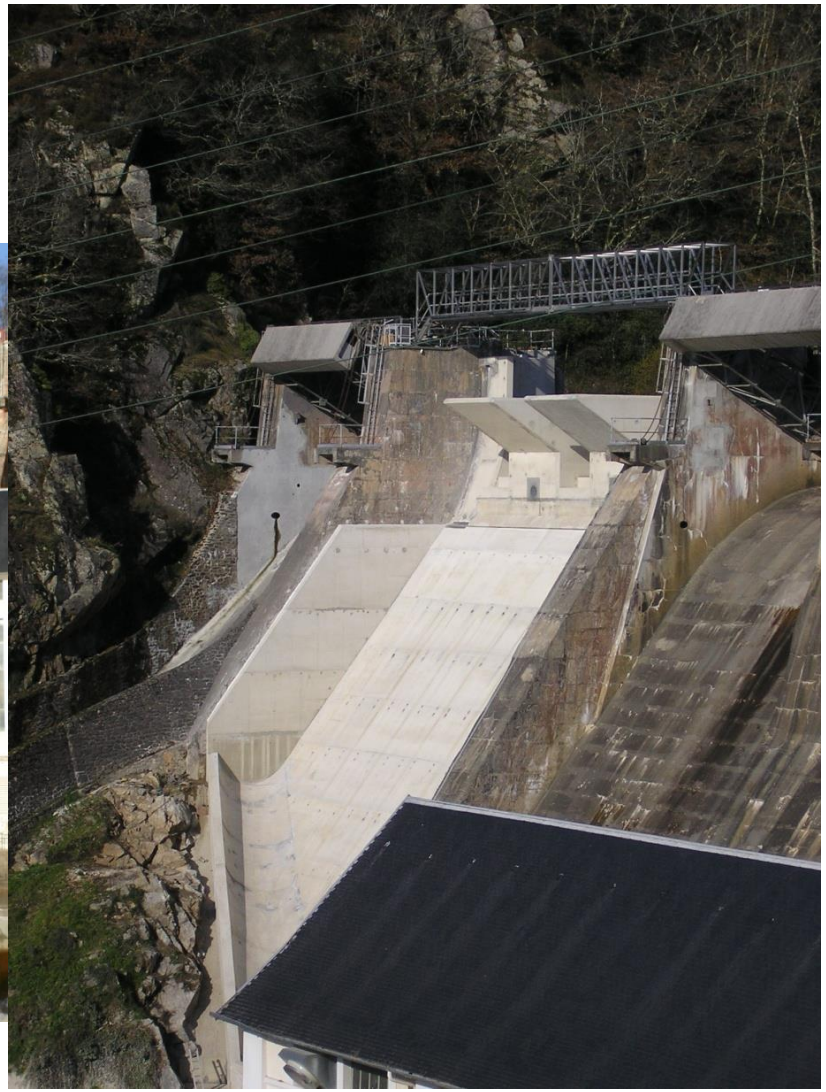




- Little overcost at end of works Design / Construction ⇔ **Simple** structures easy to build and design

7. Design Overview – PKW – Costs – Dissipation structures

- Example : Saint-Marc PKW
- Ski jump and downstream slab



7. Design Overview – PKW – Costs – Dissipation structures

- Example : Gloriettes PKW
- Long downstream channel

7. Design Overview – PKW – Costs – Dissipation structures

- Example : Record dam : the lower ratio cost/discharge at MWL

7. Design Overview – PKW – Planning

- Schedule : 4 to 24 months
  1 or 2 seasons

- Sometimes : Preparatory works 1 year before : Access road, working platform, woods etc..

- Typical schedule :
  - ✓ Season 0 : Access and platform
  - ✓ Season 1 : Downstream dissipation structures
  - ✓ Season 2 : PKW

  HIGHLY Depends on the Owner operation constraints !

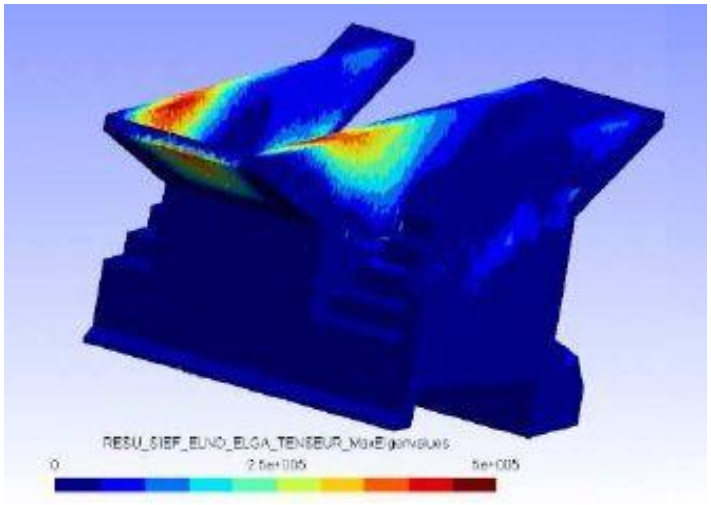
9. Design considerations and design examples

- **Structural design – Internal stability**

- Thin structures < 50 cm
- But very hyperstatic and stiff

=> **<u>Reinforcement ratio is moderate</u> < 90 kg / m³**

- Attention to :
  - Durability : Specific « hydraulic » concrete
  - **Thermal stresses**
  - Ice loadings (when applicable – Smart design)



Critical loading : thermal

9. Design considerations and design examples

- **Structural design – Overall stability**

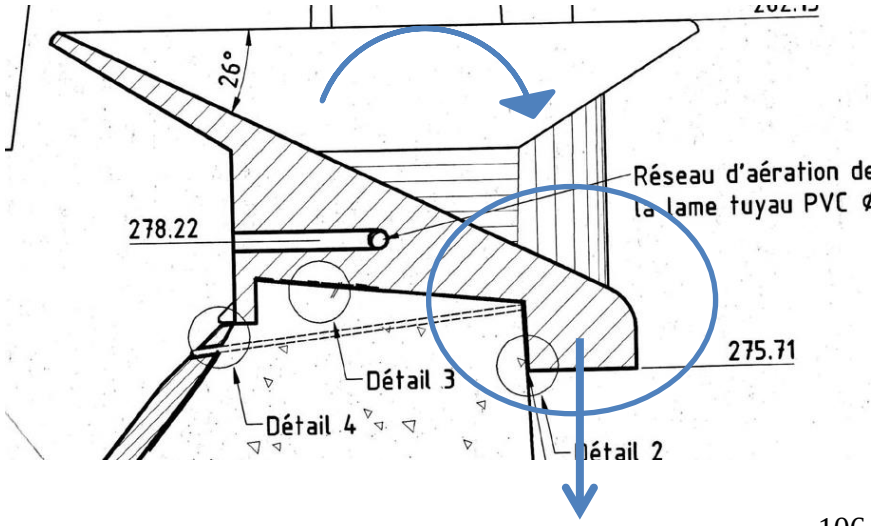
  - PKW are ~ **self stable**…
    Overturning /Sliding Safety Factor > 1

  - Particular attention to seismic loading
    ⇒**Amplification** up to 4 for a 40 m concrete dam !!

  - Possible **to avoid anchoring of** the PKW to the dam if required ⇔
Dam concrete swelling pathology (St-Marc / L'Etroit)

⇒ The PKW simply rests on top of dams and behaves as gravity structure

⇒ Add a upstream counterweight beam to :
  - Increase stability factor
  - Emergency system in case of sliding



9. Design considerations and design examples

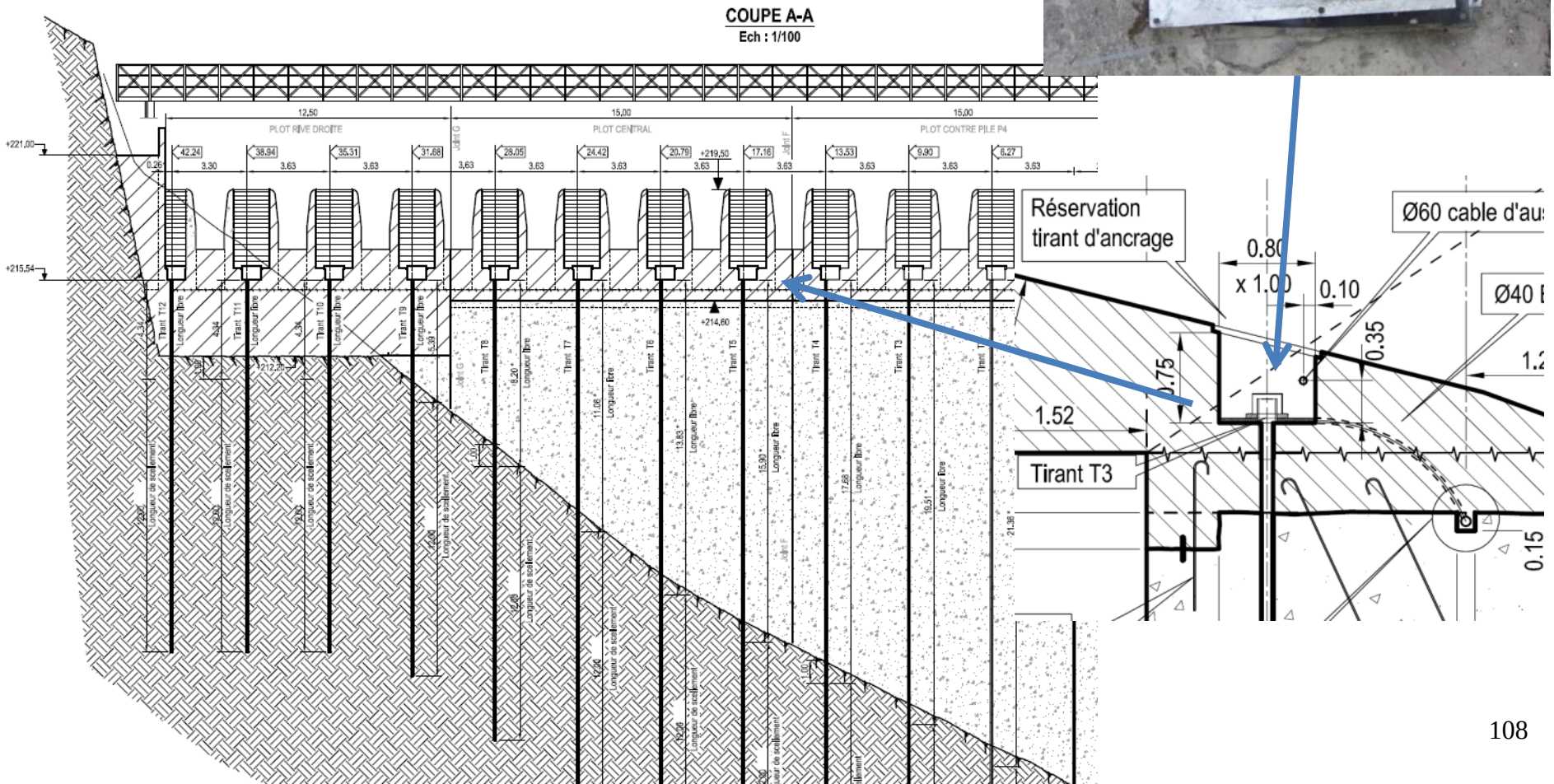
- Improve self-stability

⇒ Upstream slabs thickened

(Raviege)

9. Design considerations and design examples

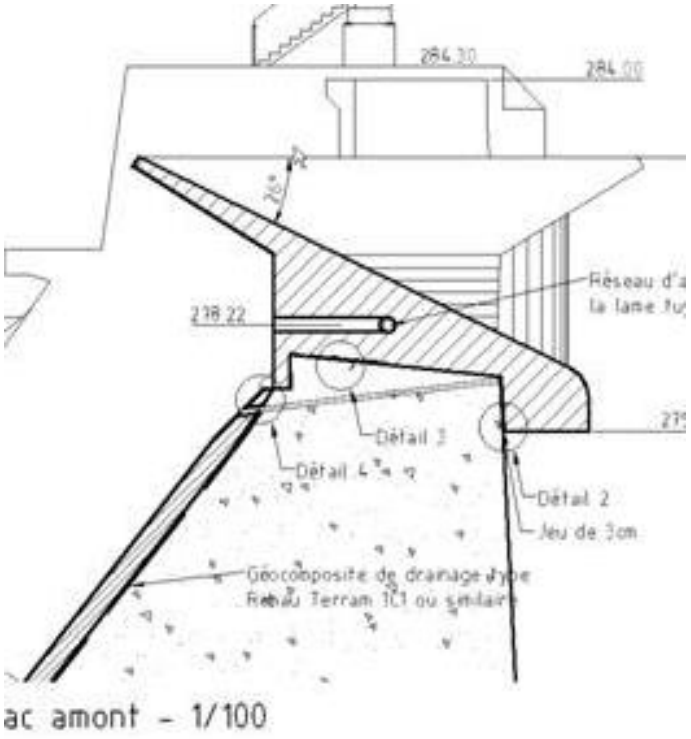
PRESTRESSED ANCHORS



9. Design considerations and design examples

- **Drainage system**

=> Specific treatment of PKW / Dam interface to reduce uplift pressure

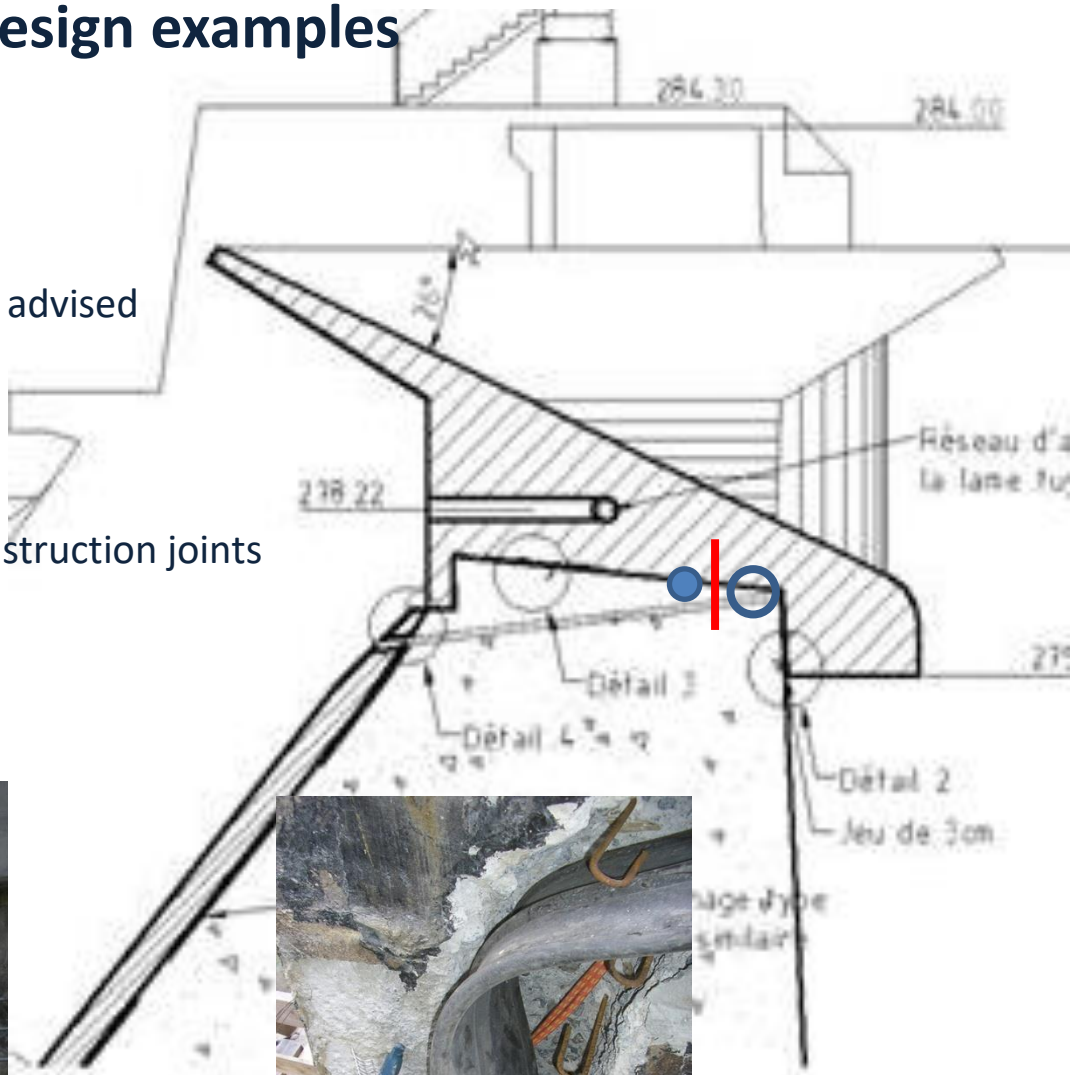


PKW interface drainage

9. Design considerations and design examples

Waterproofing

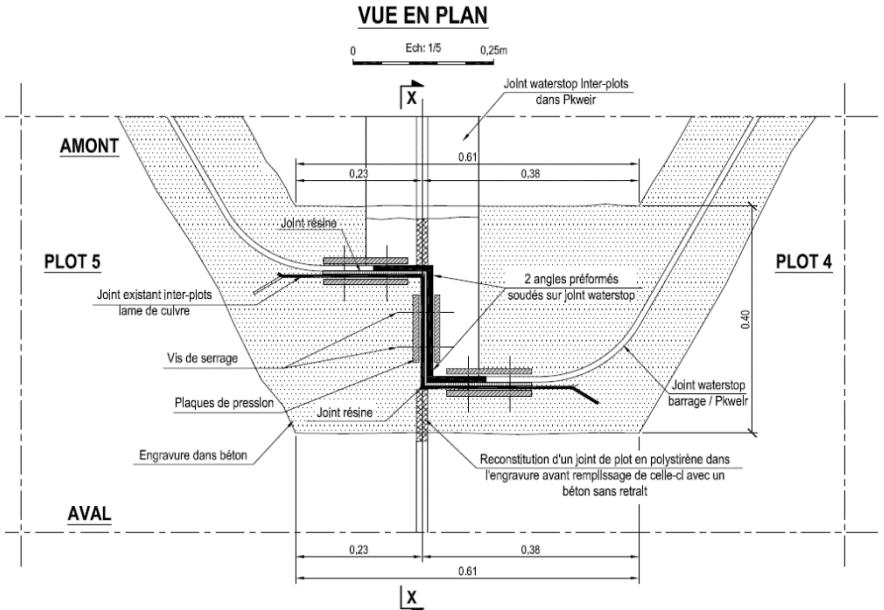
- Waterproofing – Triple / Quadruple barrier advised
  - Waterstop
  - Hydroswelling joint
  - Let injection pipes
  - Apply watertight resin on concrete construction joints (durabilty)

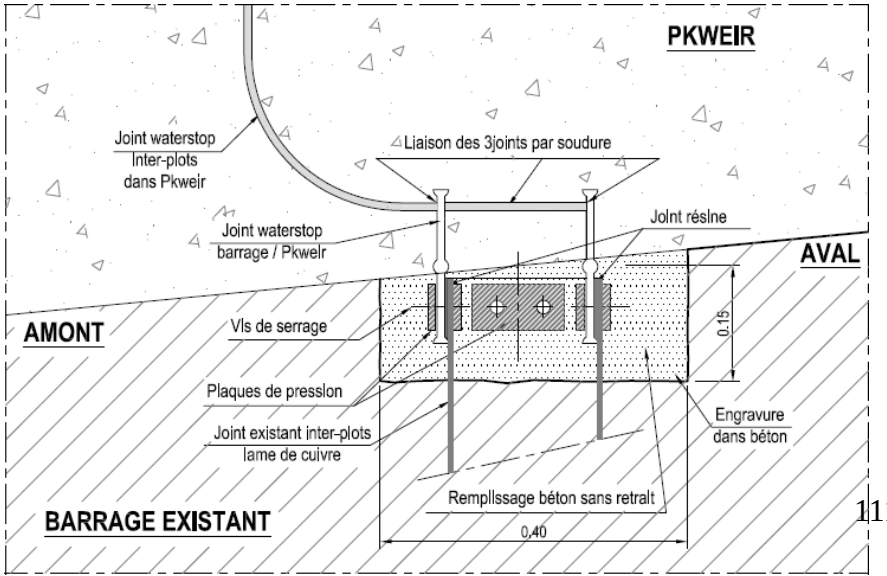


9. Design considerations and design examples

Structural Waterproofing

- Complicated connexion with existing joints





9. Design considerations and design examples

Crest shape - **circular** or half circular

- Easy and cheap
- Better discharge for low head
- Better for **durability**

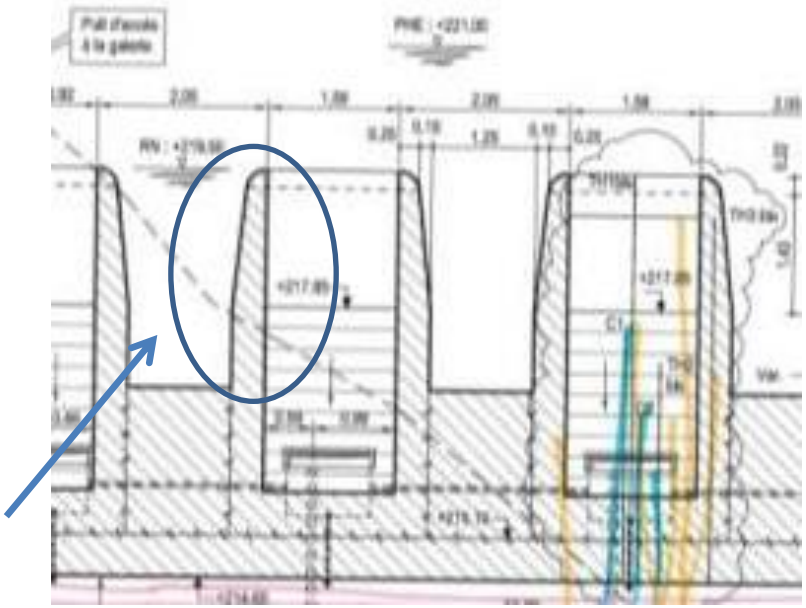
Nose

- Slight hydraulic improvement
- Cheap
- Good for overall stability
- Basic triangular shape

9. Design considerations and design examples

- Sidewall : **Variable thickness**
- Ts ⇔ Up to 15% impact

⇔ Hydraulic + structural optimization
5% increase
(cf paper *Laugier,2011* and *2017*)



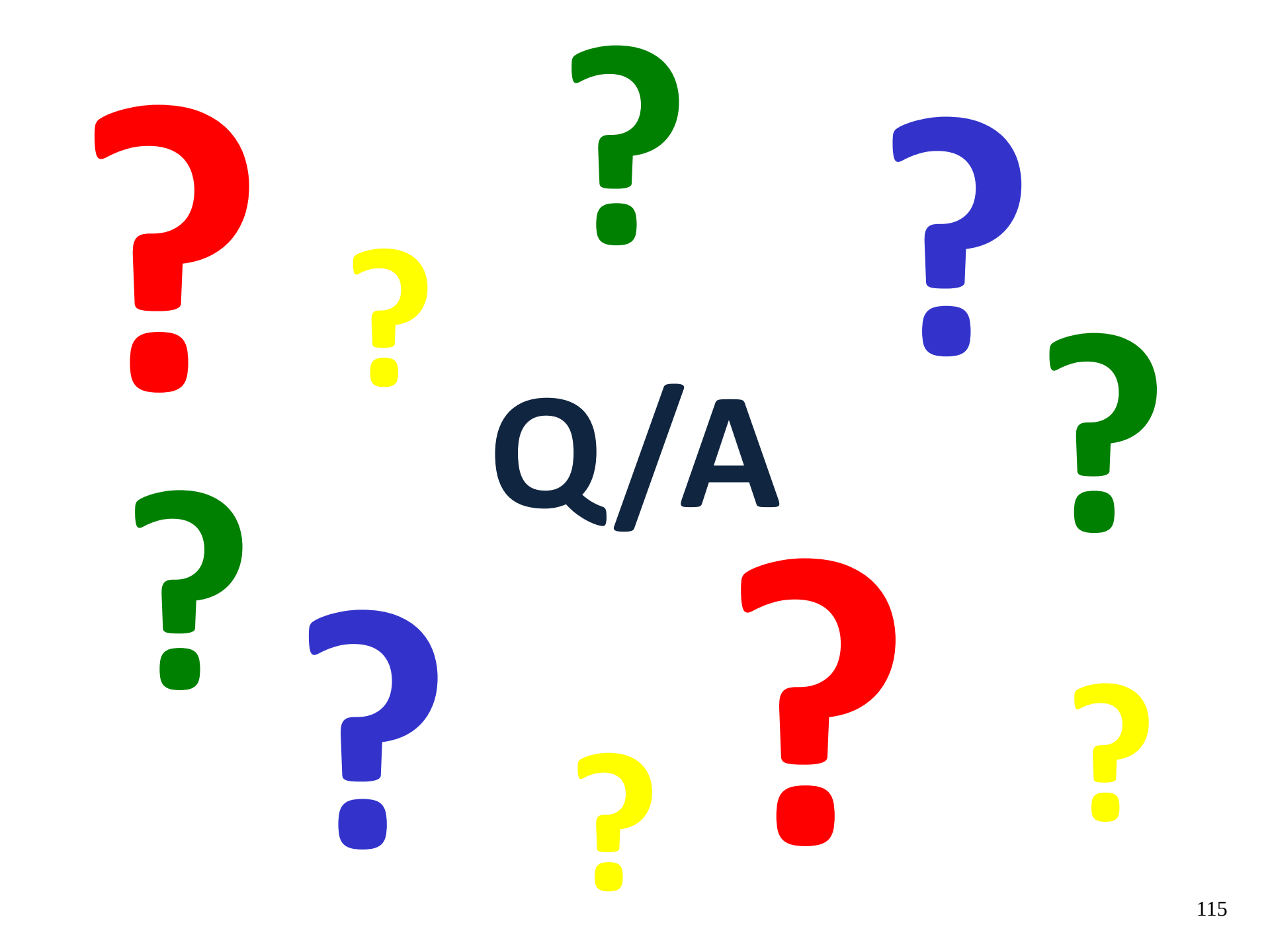
9. Design considerations and design examples

Structural PKW Material

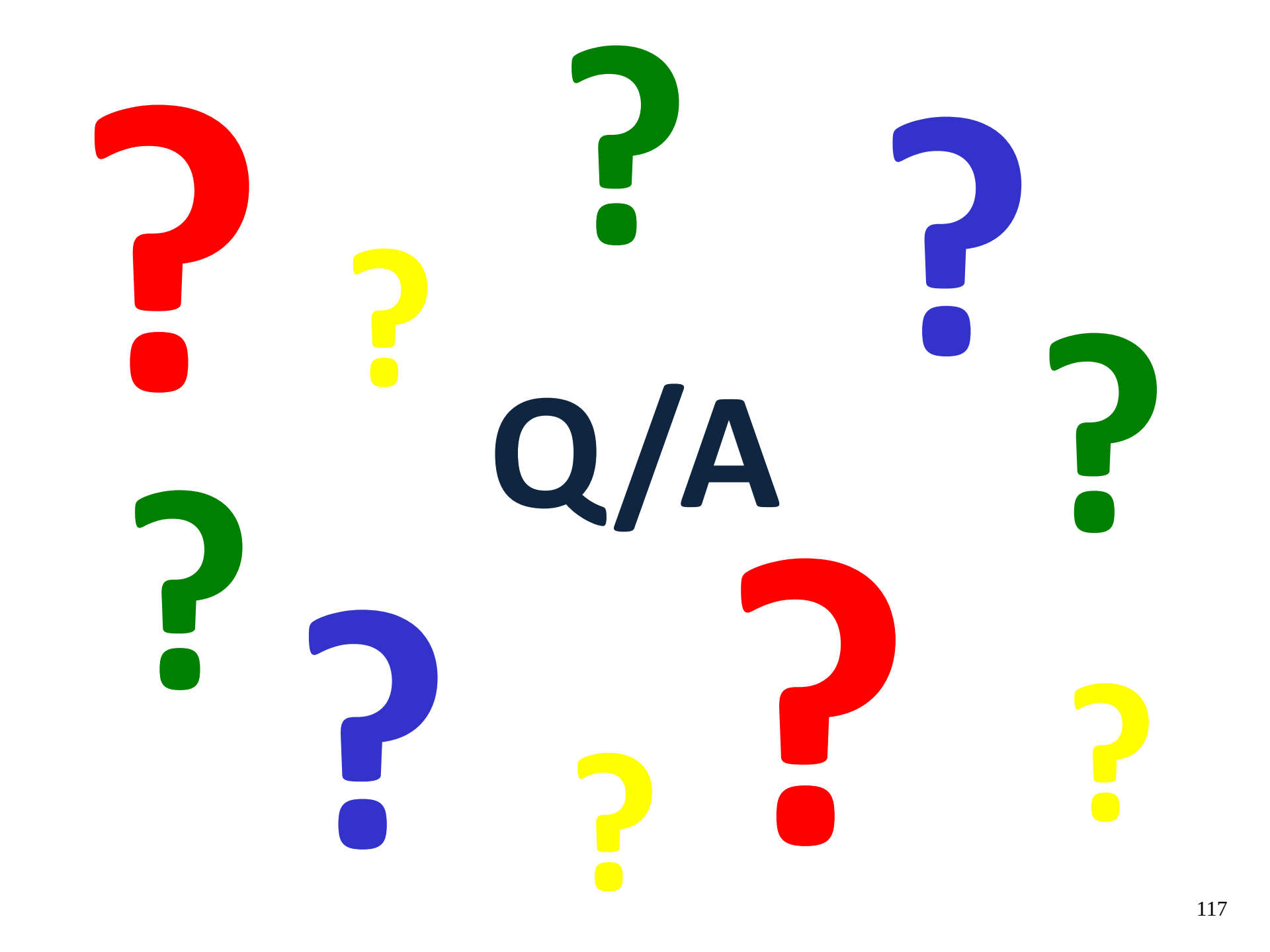
- So far : All EDF PKW are in Reinforced Concrete
  - Precast units possible

- Steel seems possible for small units

- Sandwich steel / concrete could be a better compromise
  eg Hydroplus fusegate in Australia – PKW2011
  - Significant thickness reduction (20 cm thick for 6 m high structure)
  - Used as sacrificial formwork => Speed up the construction process


Q/A

Coffee break


Q/A

Thank you for your attention

More information?

http://fr.hydrocoop.org/piano-keys-weir/
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