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Embankment dam spillways and energy dissipators

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ABSTRACT: For the last fifty years, the design floods of a number of embankment dams were re-evaluated and the revised spillway outflows are often larger than the original design discharges. Several embankment overtopping protection systems were developed for earthfill structures, and the applications range from river dykes to tsunami protections including embankment dams. Well-known designs include timber cribs, sheet-piles, riprap and gabions, reinforced earth, minimum energy loss (MEL) weirs, embankment overflow stepped spillways and the precast concrete block protection systems. In this review, several design techniques are reviewed and discussed based upon prototype experiences. A critical analysis of their performances highlights that a safe operation of embankment dam spillways and associated energy dissipators relies upon a sound design and a good quality of construction, suitable flow conditions, together with regular maintenance.

1 INTRODUCTION

Worldwide the design floods of numerous reservoirs were re-assessed and the revised discharge outflows are typically larger than the original design flow rate. The occurrence of such large flood events would result in dam overtopping, with catastrophic consequences in the case of embankment dams when an insufficient storage or spillway capacity is available. A number of overtopping protection systems were developed for embankment dams. These design techniques encompass timber cribs, sheet-piles, riprap and gabions, reinforced earth and concrete overtopping protection systems including minimum energy loss (MEL) weirs, embankment overflow stepped spillways, precast concrete block protection systems developed by the Russian engineers (ASCE 1994, Chanson 2009). In a broad sense an embankment is an earthfill structure designed to hold water. The definition includes river dykes, coastal barriers for storm surge and tsunami protections, as well as natural lakes and landslide dams (Fig. 1). Figure 1 presents some embankment dam structures: Figures 1a and 1b show some relatively large dams; Figures 1b to 1d illustrate some in-stream embankment structures. Figures 1b and 1d highlight the downstream energy dissipator operation during small flood events.

All embankment dams are potentially erodible when overtopped, unless an overtopping protection systems is designed. During the last three centuries, a number of embankment structures failed worldwide (Smith 1971, Schnitter 1994). The most common causes of failures were dam overtopping and internal cracking. The former is linked with insufficient flood release capacity, and the latter results from a combination of poor understanding of geotechnical concepts, inappropriate construction standards and internal failure. Still today, embankment dam overtopping occurs during extreme rainfall events because of inadequate spillway capacity: e.g., Tous dam (Spain), Lake Ha! Ha! (Canada), Opuha dam (New Zealand), Glashüttte dam failure (Germany),

A number of embankment dam overtopping protection systems were developed during the last few decades. Herein several design techniques are presented and discussed, after a brief discussion of the embankment breaching process. The prototype experience gained during the past decades is analysed and discussed.

(a)

(b)

(c) (d)

Figure 1. Photographs of embankment dam and spillways (a) Melton dam, Melton VIC (Australia) on 30 January 2000; (b) Chinchilla minimum energy loss (MEL) weir, Chinchilla QLD (Australia) in operation on 8 November 1997; (c) Timber crib structure: Greenup weir, Inglewood QLD (Australia) on 21 January 2009 (Courtesy of Damien Roman); (d) Sheet-pile concrete slab embankment: Joe Sippel weir, Murgon QLD (Australia) in operation on 5 March 2013

2 EMBANKMENT BREACH DEVELOPMENT

The breaching process of an embankment dam is a relatively slow process, in comparison to the failure of a concrete arch dam. The latter may be a sudden, explosive failure, while an earthfill structure may be overtopped for sometimes before the breach develops progressively leading to the complete failure (Fig. 3). For example, the failure of the 100 m high Teton dam (USA) started around 11:00 am and the reservoir was drained by the evening; the breaching of the Zeyzoun dam (Syria) took more than three and half hours. In one case (Glashütte dam), witness reports indicated that the grass-lined downstream slope of the embankment was overtopped for more than 3 hours before the complete dam failure (Bornschein and Pohl 2003). Figure 2 presents some photographs of an embankment breaching experiment.

A number of studies on embankment breaching were conducted during the last fifteen years (Coleman et al. 2002, Rozov 2003, Chanson 2005, Dai et al. 2005, Hanson et al. 2005, Morris et al. 2007, ASCE/EWRI Task Committee on Dam/Levee Breaching 2011). Most experimental studies under carefully-controlled laboratory flow conditions, together with prototype observations, showed that the embankment breach starts with an initiation phase, followed by a rapid development of the breach, and then an enlargement of breach width once the breach invert reaches the main channel bed substrate. The visual observations highlighted a challenging similarity between the embankment breach process and the flow in a minimum energy loss weir inlet during the breach development (McKay 1970, Visser et al. 1990, Chanson 2004). This was nicely illustrated by two seminal laboratory studies of the breaching of non-cohesive embankment structures (Coleman et al. 2002, Rozov 2003). It is also seen in Figure 2. Figure 2a show the initial stages of the breach development, with a definition sketch in Figure 2b. Figure 3 shows some quantitative results based upon the re-analysis of data by Coleman et al. (2002).

A flow net analysis was performed with the laboratory data of Coleman et al. (2002). The flow cross-section areas were measured along the equipotential planes at different locations and times. Some typical results are shown in Figure 3a, with the breach cross-sectional shapes below the water line at several longitudinal sections for a given time t since the start of breach development. The results highlihgted that the embankment breach flow was transcritical. Namely the flow was nearly critical between the inlet lip and throat, and the total head remained constant (Figs. 3b & 3c). The head losses occurred downstream of the throat when the flow streamlines diverged and some flow separation occurred at the lateral boundaries. Figure 3b and 3c show experimental results with the dimensionless total head H/H_1 as a function of the dimensionless centreline location, where H_1 is the upstream total head above downstream channel elevation and L is the embankment base length. Visual observations showed that the flow through the breach between the inlet lip to throat was very similar to the flow through a minimum energy loss (MEL) spillway inlet during the breach development. See, for example, a comparison Figure 2b and Figure 4.

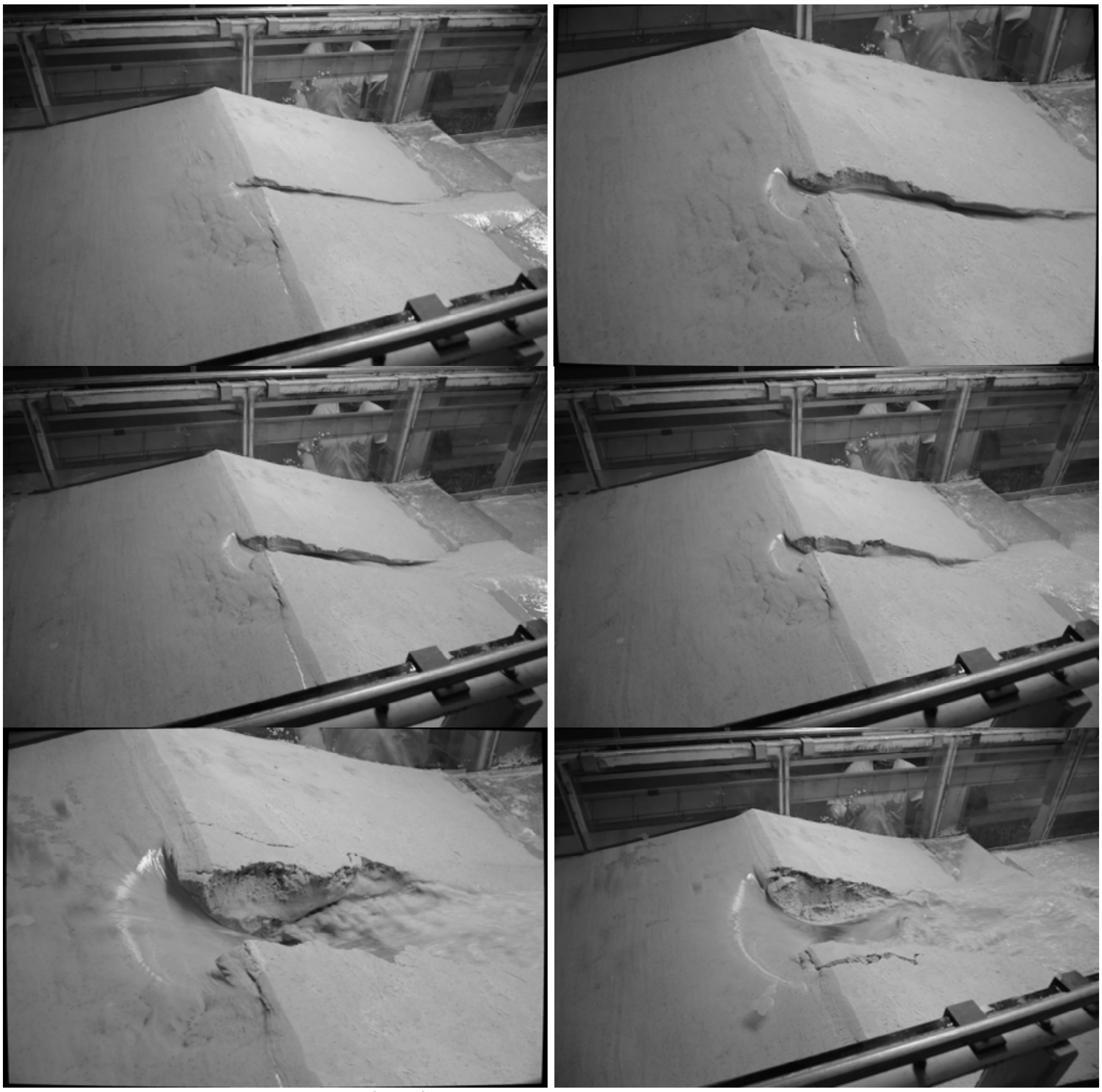
The length of breach inlet, measured along the breach centreline between inlet lip and throat, was typically $L_{inlet}/B_{max} = 0.5$ to 0.6, where B_{max} is the free-surface width at the upper inlet lip (Fig. 2b). During the development of the breach, the outflow discharge satisfied the Bernoulli equation at the inlet lip:

$$Q = C_D \times B_{max} \times \sqrt{g \times \left(\frac{2}{3} \times H_1\right)^3} \qquad (1)$$

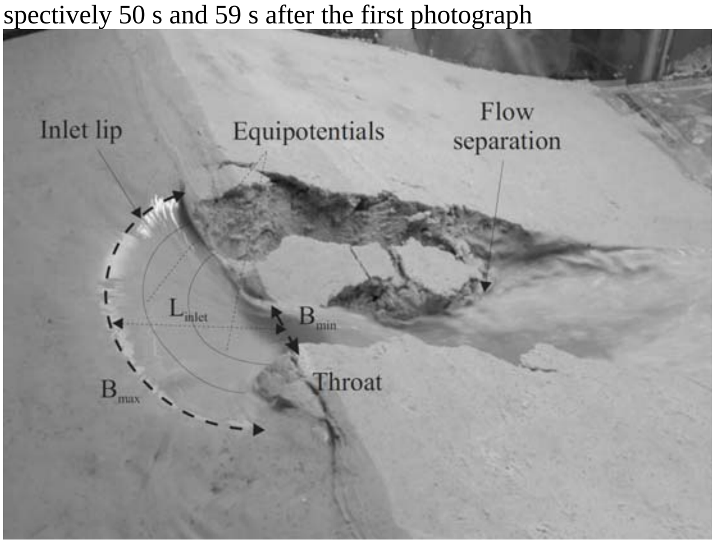
where C_D = dimensionless discharge coefficient with $C_D \sim 0.6$ on average, and the breach dimensions increase with time, resulting in the hydrograph of the breach. That is, in Equation (1), both the breach free-surface width B_{max} and upstream total head H_1 are functions of time as well as embankment characteristics and reservoir size. For a semi-infinitely long reservoir, a re-analysis of embankment breach data showed that the inlet lip elevation z_{lip}, the inlet lip width B_{max} and the throat width B_{min} varied with time as:

$$\frac{z_{lip}}{H_1} = 1.08 \times \exp\left(-0.0013 \times t \times \sqrt{\frac{g}{H_1}}\right) \qquad (2)$$

$$\frac{B_{max}}{H_1} = 2.73 \times 10^{-4} \times \left(t \times \sqrt{\frac{g}{H_1}}\right)^{1.4} \qquad (3)$$



(a) Photographic observations: the first four shots were taken 5.2 s apart; the last two shots were taken respectively 50 s and 59 s after the first photograph



(b) Breach development definition sketch

Figure 2. Physical modelling of non-cohesive embankment overtopping and breaching at the University of Auckland - H = 0.3 m, L = 1.5 m, d_{50} = 0.3 mm, constant upstream head experiment - Flow direction from left to right with the reservoir on the right

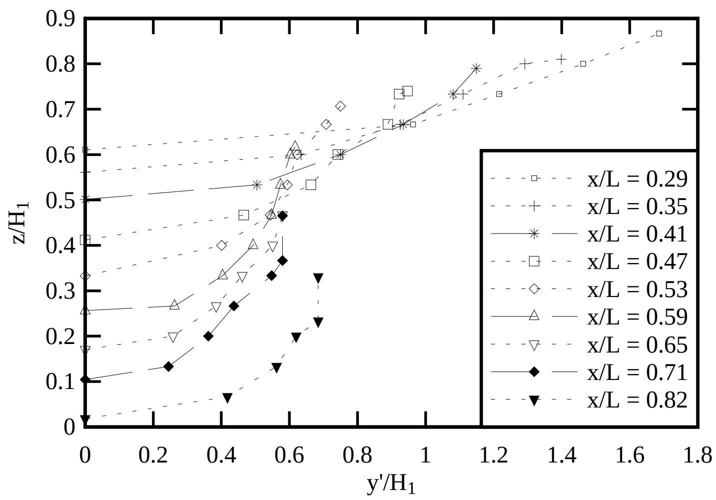
$$\frac{B_{min}}{H_1} = 4.01 \times 10^{-7} \times \left(t \times \sqrt{\frac{g}{H_1}} \right)^{2.3} \quad (4)$$

where g = gravity acceleration, z_{lip} = inlet lip elevation on the breach centreline and B_{min} = free-surface width at the breach throat (Chanson 2004). Equations (2), (3) and (4) were derived for cohesionless materials and they are valid only during the breach development.

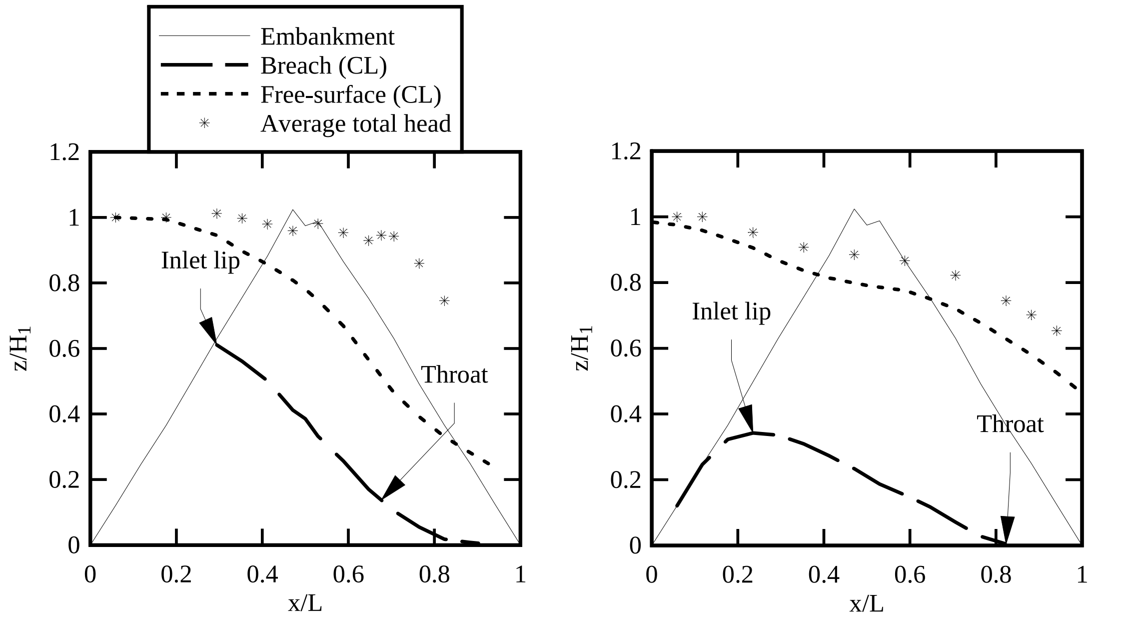
3 EMBANKMENT SPILLWAY SYSTEMS (1) THE MINIMUM ENERGY LOSS (MEL) INLET DESIGN

3.1 *Presentation*

The minimum energy loss (MEL) inlet design is a novel embankment spillway system introduced in Australia during the 1960s (McKay 1971,1978). The first MEL inlet structure was the Redcliffe storm waterway system (1960); the structure is still in use and passed floods greater than its design flow (Q_{des} = 25.8 m^3/s) without damage (McKay 1970, Chanson 2007). The MEL inlet design was developed to pass large floods with minimum energy loss and afflux, where the afflux is a quantitative measure of the upstream flooding caused by the hydraulic structure: i.e., the afflux is the rise in upstream water level caused by the presence of the embankment dam. Upstream of the inlet, the water discharge is smoothly converged towards a streamlined MEL chute, designed to yield a nearly-constant total head along the waterway (Figs. 1b &4). The approach flow region and MEL waterway are streamlined to avoid significant form losses. At design conditions, the flow may be critical from the inlet lip to the chute toe. The MEL inlet system was developed for embankment dam applications where the river catchment is characterised by large rainfalls and a very small bed slope. Figure 1b presents an overflow MEL embankment weir. Figure 4 presents a prototype operation during a small spill: the efficient inlet design allowed and extra 0.457 m of water storage for the same maximum discharge capacity (McKay 1971).



(a) Breach cross-sectional shape along equipotential planes below water line, t = 87 s, Q_{breach} = 0.024 m^3/s



(b) Longitudinal bed elevation and total head along breach centreline, t = 87 s, Q_{breach} = 0.024 m^3/s
(c) Longitudinal bed elevation and total head along breach centreline, t = 147 s, Q_{breach} = 0.071 m^3/s (same legend as Fig. 3b)

Figure 3. Laboratory measurements of non-cohesive embankment dam breaching - Data set: Coleman et al. (2002), data re-analysis by Chanson (2004,2005), embankment height; H_1 = 0.30 mm, length: L = 1.7 m, upstream and downstream slopes: 1V:2.7H, 1.6 mm sand, constant upstream head experiment

Figure 4. Minimum energy loss spillway inlet at Lake Kurwongbah (Brisbane QLD, Australia) in operation on 29 January 2013

3.2 *Hydraulic design*

The purpose of a MEL inlet is to minimise afflux and energy dissipation at the design discharge, while avoiding scour and bank erosion at the toe of the chute. The inlet is curved in plan to converge the chute flow and the chute slope is relatively flat. A MEL inlet is basically a streamlined channel with converging chute sidewalls and the spillway chute is relatively flat. A downstream energy dissipator is concentrated near the channel centreline at the downstream end. At the chute toe, the inflow Froude number remains low and the rate of energy dissipation is small compared to a traditional weir. As an example, the Chinchilla MEL weir (Fig. 1b) was designed to give zero afflux at design flow (Q_{des} = 850 m^3/s); in 1974, the overflow discharge was estimated at 1,130 m^3/s and the measured afflux was less than 100 mm (Turnbull and McKay 1974). Assuming a relatively broad crest and a smooth approach without head loss, the discharge capacity of the MEL inlet equals:

$$Q = B_{max} \times \sqrt{g} \times \left(\frac{2}{3} \times (H_1 - z_{crest}) \right)^{3/2} \tag{5}$$

where H_1-z_{crest} = upstream head above spillway crest and B_{max} = crest width (see definition in Fig. 2b). A MEL spillway channel could be designed to achieve critical flow conditions at any position along the chute and, hence, to prevent the occurrence of a downstream hydraulic jump with high tailwater conditions. Assuming negligible energy loss along the inlet, the channel width B at any elevation z-z_{crest} beneath the crest above the weir toe would satisfy:

$$B = B_{max} \times \left(\frac{H_{des} - z_{crest}}{H_{des} - z} \right)^{3/2} \tag{6}$$

where H_{des} = design upstream head. Equation (6) is only valid at design flow conditions. In practice, the variations of the tailwater elevations with discharge are important and a weak jump may take place at the inlet toe as seen in Figure 1b. The downstream conjugate depth is fixed by the tailwater conditions downstream of the hydraulic jump.

3.3 *Prototype experience*

The MEL spillway structures were designed with the concept of constant total head, hence zero afflux. The above pre-design calculations must be validated with some solid physical modelling, typically using 1:50 to 1:80 undistorted scale models with fixed bed.

The MEL overflow spillways may be built over earthfill structures and protected by concrete slabs. The construction costs must be minimum. The operations of a number of MEL spillways and weirs were documented, with a complement of field inspections and discussions with designers and operators (Chanson 2003,2009). A number of MEL structures were observed to operate at design flow conditions and for floods larger than design. Inspections during and after flood events showed the sound operation together with little maintenance. The successful operation of several structures for over 40 years has highlighted some key operational considerations. Some improper approach flow conditions could affect adversely the spillway operation. MEL weirs are typically earthfill structures. An efficient drainage system must be installed underneath the chute slabs. A known issue is the overtopping risk during construction as for the Sandy Creek weir and Chinchilla weir (twice).

4 EMBANKMENT SPILLWAY SYSTEMS (2) THE CONCRETE STEPPED SPILLWAY

4.1 *Presentation*

During the last three decades, a number of embankment dams were equipped with a overflow concrete stepped spillway (Chanson 2001, Gonzalez and Chanson 2007) (Figs. 1a, 1d & 5). Applications included both primary and secondary spillway structures: Figure 1d shows a sheet-pile concrete slab embankment weir with an overflow spillway across the whole width of the river bed. Figures 1a and 5 illustrate embankment dams equipped with a secondary embankment overflow stepped spillway.

Most modern stepped spillways consist of flat horizontal steps, although different step configurations may be considered (Andre et al. 2004, Gonzalez and Chanson 2008, Guenther et al. 2013). The preferred construction method is the placement of roller compacted concrete (RCC) overlays on the downstream embankment slope (Ditchey and Campbell 2000, Gonzalez and Chanson 2007). During the construction, the RCC is placed typically in a succession of 0.2 to 0.4 m thick overlays with a width greater than 2.5 m for proper hauling, spreading and compacting. The advantages of the RCC construction include the cost effectiveness and the short duration of construction. Exposed RCC is frequently used for secondary spillways with infrequent overflows. In harsh climatic conditions, or for a primary spillway, a conventional concrete protection layer may be installed. In all the cases, a drainage layer beneath the concrete overlays is essential to prevent uplift pressures. Its purpose is to relieve pore pressure at the interface between the embankment and concrete stepped spillway. The drainage layer may be complemented by a series drain holes formed through the RCC during placement. At the downstream end of the overflow, a cutoff wall must be built to prevent the undermining of the concrete system during discharge.

Figure 5. Choctaw 8A auxiliary spillway (USA) (Courtesy of Craig Savela and USDA)

4.2 *Hydraulic design considerations*

An embankment stepped spillway is designed to operate in a skimming flow regime (Chanson 2001). During the design process, the constraints include the embankment height, embankment downstream slope and design discharge. The variable parameters comprise typically the type of crest shape, the chute width and possibly the step height, although the step height h is always selected as a multiple of the RCC overlay height (Gonzalez and Chanson 2007).

In a skimming flow, the upstream flow region is characterised by a developing boundary layer (Amador et al. 2006, Meireles and Matos 2009) (Fig. 6). When the outer edge of the boundary layer interacts with the free-surface, the turbulent shear stress becomes greater than the surface tension force per unit area resisting the interfacial breakup and free-surface aeration takes place (Ervine and Falvey 1987, Chanson 2009b). The characteristics at the inception point of free-surface aeration are:

$$\frac{L_I}{h \times \cos\theta} = 9.72 \times (\sin\theta)^{0.080} \times \left(\frac{q}{\sqrt{g \times \sin\theta \times (h \times \cos\theta)^3}} \right)^{0.71} \tag{7}$$

$$\frac{d_I}{h \times \cos\theta} = \frac{0.403}{(\sin\theta)^{0.04}} \times \left(\frac{q}{\sqrt{g \times \sin\theta \times (h \times \cos\theta)^3}} \right)^{0.59} \tag{8}$$

where q = discharge per unit width (q = Q/B), L_I = longitudinal distance from the chute crest, d_I = flow depth at the inception point, g = gravity acceleration and θ = angle between the pseudo-bottom formed by the step edges and the horizontal (Fig. 6).

When the spillway chute is long and the flow reach uniform equilibrium (i.e. normal flow conditions), the characteristic flow depth d equals:

$$d = \sqrt[3]{\frac{f_e \times q^2}{8 \times g \times \sin\theta}} \tag{9}$$

where f_e = Darcy friction factor estimated based upon experimental air-water flow friction factor data (Chanson et al. 2002, Chanson 2006). If the flow does not reach normal flow conditions before the downstream end of the spillway, the flow is gradually varied downstream of the inception point of air entrainment. Combining some well-documented experimental results together with theoretical calculations, an empirical correlation was derived in terms of the downstream spillway velocity as a function of the upstream above crest and discharge (Gonzalez 2005):

$$\frac{U_w}{V_{max}} = 0.00105 \times \left(\frac{H_1}{\sqrt[3]{q^2/g}}\right)^2 - 0.0634 \times \left(\frac{H_1}{\sqrt[3]{q^2/g}}\right) + 1.202 \tag{10}$$

where H_1 = upstream total head above chute toe, d_c = critical depth, V_{max} = ideal flow velocity deduced from the Bernoulli principle, and U_w = downstream velocity. Such an approach may be used for pre-design calculations assuming a friction coefficient f_e = 0.2, although the method was only validated for moderate stepped spillway slopes ($15^o < \theta < 25^o$). These preliminary estimates must be checked with some solid physical modelling, based upon undistorted scale models with a scaling ratio no greater than 3:1 (Chanson and Gonzalez 2005, Felder and Chanson 2009).

For short stepped spillways and large discharges, the flow may not be fully-developed before the downstream end of the chute. That is, the chute length may be smaller than the distance between crest and inception point of free-surface aeration. A simple method was developed to predict the depth-averaged flow properties (Chanson 2001, Meireles and Matos 2009).

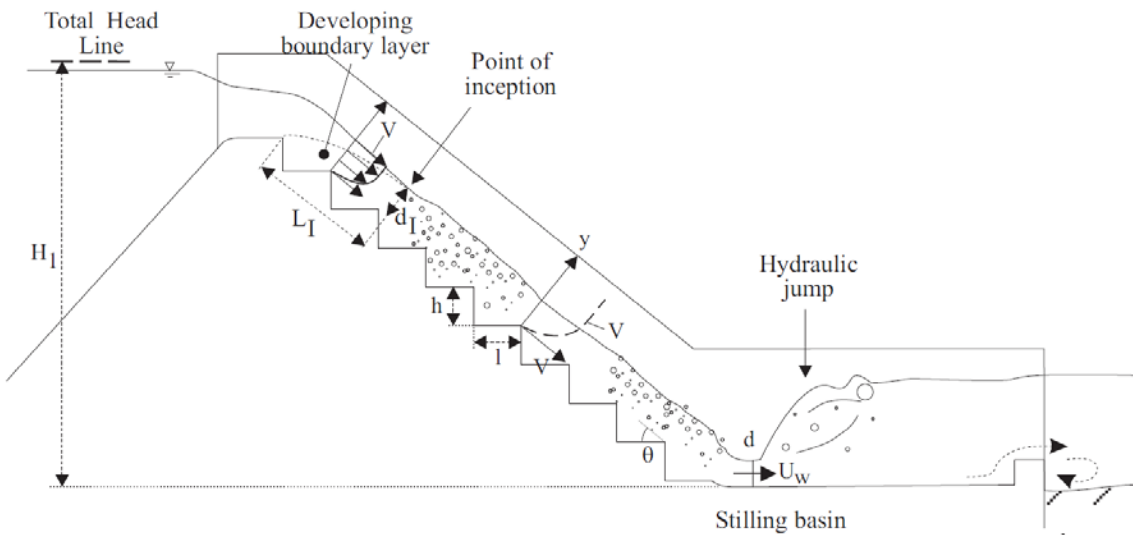


Figure 6. Definition of sketch of an embankment stepped spillway operation with downstream hydraulic jump stilling basin

5 EMBANKMENT SPILLWAY SYSTEMS (3) GABION STEPPED WEIR

A gabion is a basket filled with earth or stone for use in engineering, and the construction technique is extensively used for earth retaining structures and hydraulic structures. Its advantages are the stability, low cost, flexibility and porosity of the construction material. The gabion porosity in particular is important to prevent the build-up of uplift pressures. Modern box gabions consist of rockfill material enlaced by a basket or a mesh, shaped like a rectangular box. Typical gabion dimensions are heights of 0.5 to 1 m, with a width equal to the height and length-to-height ratio between 1.5 and 4. Longer gabions may be subdivided into cells by inserting mesh diaphragms to strengthen the box. The wire is normally made of soft steel with a zinc coating. The durability of gabion structures relies heavily upon the quality of mesh and wire. The gabion filling consists of loose or compacted rocks. The stone size must equal at least 1 to 1.5 times the mesh size but should not be larger than 2/3 of the minimum dimension of the gabion. The use of small-sized stone, typically 1.5 times the mesh size, permits a better adaptability of the gabions to deformation.

The dimensions of the gabion box and the design discharge are the two basic design parameters controlling the hydraulic operation of the stepped chute. The step height h is typically the gabion height, although it might be twice or three times the gabion height in some cases. The stepped chute slope ranges from 1V:4H to 1V:2H. For a gabion structure only, the choice of a steep slope with a skimming flow regime operation may reduce the number of gabions and the overall structure cost. For an embankment with gabion overtopping, a flat slope may be more appropriate for the stability requirements of the earthfill structure (Fig. 7). Figure 7 illustrates a gabion stepped weir. The design considerations for the stability of gabion weirs are generally the same as for any gravity structure. The calculations of structural stability involve checking the stability of the weir against overturning, sliding and uplift. Inclined (upward) gabion-stepped spillways may also be used (Peyras et al. 1991). Larger energy dissipation is achieved but their construction requires greater care. In comparison with concrete spillways, the flow above a stepped gabion chute is characterised by some interactions between the surface overflow and seepage flow, and a rougher surface of the gabion steps. The former aspect was discussed by Kells (1993,1995), while Gonzalez et al. (2008) documented the effect of step roughness for a 1V:2.5H stepped chute.

The performances of gabion stepped weirs are often restricted by the gabion resistance to damage and their stability. Sediments and debris carried by the stream flow may affect and fracture the gabion mesh. With large-size debris, it is common practice to protect the step surfaces with timber, steel sheets, concrete facing or even reinforced concrete slab (Agostini et al. 1987, Peyras et al. 1991).

Figure 7. Gabion stepped spillway at Robina (Gold Coast QLD, Australia) on 2 April 1997, shortly after completion: h ~ 0.5 m, h/l ~ 0.5

6 EMBANKMENT SPILLWAY SYSTEMS (4) PRE-CAST CONCRETE BLOCK SPILLWAY

6.1 *Presentation*

The pre-cast concrete block spillway design was developed in Russia by late Professor Gordienko (Gordienko 1978, Pravdivets and Bramley 1989). The chute is made of individual blocks placed in an overlapping staircase fashion (Fig. 8). Figure 8 shows an experimental earth dam in Siberia equipped with a pre-cast concrete block spillway for primary flood release. An interesting feature is the flexibility of the channel bed allowing differential settlements of the earthfill embankment. Another feature is the fairly short construction time on site, while and stepped design contributes to some energy dissipation along the chute.

The Russian engineers developed a strong expertise in the design of concrete wedge blocks. This was supported by extensive testing. For large discharges, each block should be tied to adjacent blocks, possibly made of reinforced concrete. A step height-to-length ratio in the range 1:4 to 1:6 may ensure maximum stability of the blocks during the overtopping. Drains must be placed in areas of sub-atmospheric pressure to relieve uplift pressures.

Figure 8. Precast concrete block spillway of Volymia dam (Siberia, Russia) (Courtesy of Prof. Y. Pravdivets) - This experimental structure (H=20 m) operated for more than 15 years, the spillway discharging ice and water from early spring to late fall

6.2 *Practical considerations*

For any embankment dam, an uppermost basic criterion is the stability of the material. Seepage may occur in saturated embankment, resulting in uplift pressures which might damage or destroy the entire structure. In a typical precast block chute design, an adequate drainage is essential, and the blocks lay on a filter and erosion protection layer. The layer filters the seepage flow out of the subsoil and it protects as well the subsoil layer from erosion by flow in the drainage layer. The protection layer reduces drastically the uplift pressures acting on the concrete blocks. Usually a geotextile membrane is laid on the embankment before the placing of the layer, and another covers the protection layer before the installation of the blocks. Drains are typically placed in areas of sub-atmospheric pressures (e.g. vertical step face) to relieve uplift pressures. The location of drains must be appropriately selected to avoid reverse flow in the drains and dynamic pressures associated with hydraulic jumps at low flows. Grinchuk et al. (1977) recommended that the total area of the drainage holes should be 10-15% of the exposed step area, although Baker et al. (1994) suggested that an open area of 2 to 5% could be optimum.

Laboratory model tests showed that the aspiration on the vertical step face increased with increasing downward slope of the steps (Frizell 1992).

The seepage flow in the embankment dam must be predicted accurately to make the appropriate provision for drainage and evacuation of seepage flow through the blocks. Note that the seepage may be influenced by the infiltration into the downstream slope caused by the spillway flow, in addition to the flow through the embankment.

7 HYDRAULIC DESIGN CONSIDERATIONS

For an embankment dam, the uppermost important design criterion is the stability of the earthfill embankment at any stage, including during droughts and flood events. The construction must be of good quality and the design must be sound and kept simple. Seepage may occur in saturated embankment yielding unacceptable uplift pressures; that is, an adequate drainage is essential. For all spillway designs, a filter and erosion protection layer is typically laid on the downstream embankment slope beneath the overflow protection.

The hydraulic design of embankment dam spillway is critical. Some key points include (a) the maximum discharge capacity estimate, (b) the downstream dissipation structure and (c) the high level of hydraulic expertise required. First the spillway capacity must be correctly estimated to prevent any overflow over the unprotected embankment section. At the downstream end of the spillway, the turbulent kinetic energy of the flow must be dissipated safely. The most common dissipation designs include the hydraulic jump stilling basin (Figs. 1b & 6) and a flip bucket to deflect the water away from the chute toe (Fig. 8). The experience has shown that the hydraulic design of embankment spillways require a high level of expertise in dam and hydraulic engineering.

The successful operation of many structures highlighted the importance of regular maintenance. Some basic down-to-earth considerations must be considered. There were accounts of vandalism in a few projects, including motor bikes riding up and down a precast block spillway, thus damaging the pre-cast concrete blocks, and locals stealing mesh of gabion structures to build local fences. Alternative embankment overtopping protection systems include timber cribs, sheet-piles, riprap, macro-roughness and reinforced earth (Chanson 2001,2009).

7.1 *Energy dissipation*

A number of construction techniques have been used for embankment dam spillways. In all cases, the safe dissipation of the kinetic energy of the flood flow is critical. A common design is the stepped profile, which increases the rate of energy dissipation on the spillway chute, thus reducing the size of the downstream energy dissipator (Chanson 2001, Ohtsu et al. 2004). Importantly the turbulent kinetic energy dissipation above the steps must be carefully estimated, in particular for large discharges per unit width corresponding to the skimming flow regime. A characteristic feature of skimming flows is the high level of turbulence and free-surface aeration (Rajaratnam 1990, Peyras et al. 1991). The water flows downs the steps as a coherent free-stream skimming over the pseudo-bottom formed by the step edges. In the step cavities, the turbulent recirculation is maintained through the transmission of shear stress from the free-stream. At the free-surface, air is continuously trapped and released, and the resulting two-phase mixture interacts with the flow turbulence yielding some intricate air-water structure associated with complicated energy dissipation mechanisms (Chanson and Toombes 2002, Gonzalez and Chanson 2008).

Two energy dissipation designs are the hydraulic jump stilling basin and the flip bucket. The skip jump deflects the water away from the dam toe, but it is usually restricted to small structures. The stilling basin design is more common. The jump must be confined to a reinforced area, away from the unprotected embankment slope and natural river banks.

8 CONCLUSION

In recent years, a number of embankment overtopping protection systems were developed for earthfill dams, coastal barriers and river dykes The overtopping protection systems include concrete stepped overtopping protection, minimum energy loss (MEL) spillway, gabion stepped spillways and precast concrete block protection systems. For embankments higher than 5 to 10 m, the concrete stepped spillway is a sound design technique well-suited to small to large discharges. The flow down the stepped cascade is characterised by some strong aeration, high turbulence of the flow and a significant rate of energy dissipation.

A number of embankment dam spillways have been in operation for three to four decades. The prototype experience provides valuable informations. Based upon past accident and failure forensic investigations, it is clearly understood that a safe operation relies upon a sound design and a good quality of construction, suitable flow conditions, together with regular maintenance. Ultimately there is no better proof of design soundness than successful prototype operation.

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