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Guidelines for Hydraulic Design of Stepped Spillways

U.S. Department of the Interior
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14. ABSTRACT
This document presents Reclamation's current state of the practice for the hydraulic design of stepped spillways. Included are discussions of flow regimes, design parameters (step height selection, crest treatments, energy dissipation, air entrainment, training wall design), and cavitation potential.

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Guidelines for Hydraulic Design of Stepped Spillways

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September 2015

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The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

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This work is a compilation of the work of many, in particular my wife, Kathleen Houston Frizell. Kathy retired from Reclamation in January 2011 and had begun working on this document. She spent much of her career at Reclamation working on hydraulic issues related to stepped spillway performance and design and was an internationally-known and respected researcher in this field. I came into the subject at the end of my career and have been most interested in air entrainment and possibility of cavitation damage on stepped chutes. In addition, internal peer review was provided by William Fiedler and external peer review by Professor Jorge Matos of the Instituto Superior Técnico, Universidade de Lisboa, Portugal. These reviewers provided numerous comments that have improved the utility of this document.

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Cover Photo: Upper Stillwater Dam, Utah

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Nomenclature

α – kinetic energy correction factor
C – local air concentration (volume of air per unit volume of air and water)
C_f – coefficient of friction
C_{mean} – mean depth-averaged air concentration
C_{meani} – mean depth-averaged air concentration at the point of inception
C_p – pressure coefficient
D_h – hydraulic diameter based on equivalent clear water depth (ft)
d_c – critical depth (ft)
d_i – depth at inception point (ft)
d_{cw} – depth of clear water normal to slope (ft)
E – energy head (ft)
ΔE – change in energy head
F_r – roughness Froude number $= q_{cw}/\sqrt{g \sin\theta\, k^3}$
F_1 – Froude number of flow at the toe of the spillway (entering the stilling basin)
F_* - alternate roughness Froude number $= F_r(\cos\theta)^{3/2}$
f – Darcy-Weisbach friction factor for non-aerated flow
f_b – Darcy-Weisbach friction factor at the pseudo-bottom
f_e – Darcy-Weisbach friction factor for air-water flow
g – gravitational constant (ft/s^2)
H_{dam} – height of dam (ft)
H_{max} – maximum energy head (ft) = $H_{dam} + 1.5 d_c$
H_o – total head over the crest (ft)
H_{res} – residual energy head (ft)
h_w – training wall height (ft)
k – step roughness height perpendicular to slope (ft)
l – step tread length (horizontal) (ft)
L – slope length in streamwise direction from crest (ft)
L_i – length along slope to inception point (ft)
N – number of steps
n – safety factor
θ – dam or spillway slope
Q – total discharge (ft^3/s)
q – specific discharge (discharge per unit width - ft^2/s)
q_{cw} – specific discharge of clear water
σ_c – critical cavitation parameter
s' – non-dimensional length to inception point
s – step height (ft)
S_f – friction slope of spillway (ft)
y – depth normal to slope (ft)
Y_{90} – depth where the mean air concentration is 90-percent (ft)
Y_{90u} – uniform flow depth where mean air concentration is 90-percent (ft)
Y_{cw} – equivalent clear water depth (air removed) (ft)

Background

Spillway selection and design are important pieces in the design of a dam project to protect and preserve the integrity of the dam's structure and associated facilities as well as preserving the safety of the people and property downstream. Selection of the type and size of the spillway can be dependent on many factors; the purpose of the dam, the location and surrounding geology, size and height of the dam, size of the drainage basin and flood potential, predicted frequency of use, and other socioeconomic constraints.

Stepped channels are just one of many types of spillways that may be selected. The use of stepped channels as spillways has a natural link to specific types of dam construction. In ancient times, large masonry blocks were often used for dam construction and thus provided for natural steps, typically on the downstream face, for overflow. In present day, stepped spillways are often paired with dam construction using roller-compacted concrete (RCC). RCC construction uses a series of uniform stepped lifts to form the dam, resulting in a natural stepped face. Many small low-head dams may use the unformed exposed surfaces of the RCC lifts as a flow surface whereas formed steps (horizontally slip formed with a curbing machine or formed conventional concrete) are typically used for higher head/high velocity applications. Earth and rock-fill embankment dams have also used stepped overlays of RCC or conventional concrete, or preformed concrete blocks placed over a section of the embankment to form a spillway, or serve as overtopping protection.

Reclamation has been involved with the design of dams and spillways in the western United States for more than 100 years. The first stepped spillway on a Reclamation facility dates back to 1911-1915 and the construction of the Lahontan Dam on the Carson River in Nevada, figure 1. While probably not the most hydraulically efficient design, the spillway took advantage of large steps within the spillway to dissipate energy and deliver the flows to the river channel at a central area downstream from the embankment.

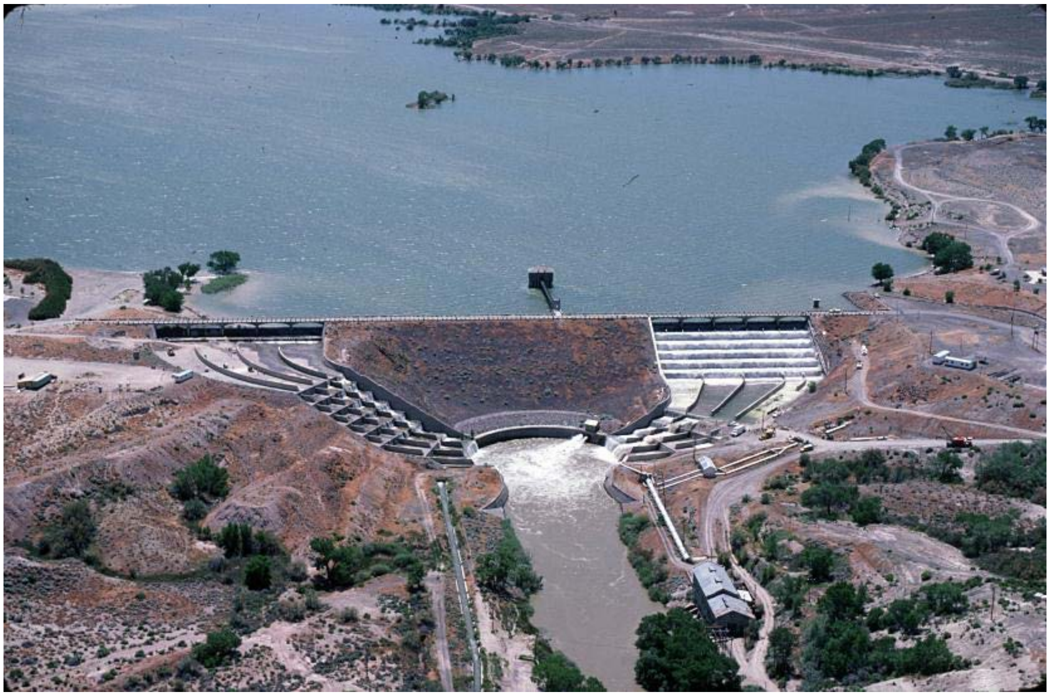

Figure 1: Lahontan Dam and spillways, Carson River, Nevada.

Many years and the construction of many large dams passed before Reclamation designed and constructed another stepped spillway. Between 1983 and 1987, Reclamation constructed what at the time was the highest roller-compacted concrete dam in the world. Upper Stillwater Dam on Rock Creek in Utah is a 300-ft-high (91 m) RCC gravity dam with a 600-ft-wide (183 m) uncontrolled stepped spillway section located in the central portion of the dam, figure 2.

Figure 2: Upper Stillwater Dam located on Rock Creek in Utah.

The stepped spillway was horizontally slip-formed with the majority of the steps 2-ft-high (0.61 m). Initial designs were completed with the aid of a physical model study (Houston (1987)) for a design discharge of 15000 ft^3/s (425 m^3/s) or a specific discharge of q=25 ft^2/s (2.32 m^2/s). However, while still in construction the inflow design flood was increased by nearly 5 times, to 74000 ft^3/s (2095 m^3/s), q=123 ft^2/s (11.45 m^2/s). The model was used again to verify that the design would perform adequately and a couple of small modifications were made to improve performance at this substantially higher design discharge.

Since that time, much of Reclamation's involvement with stepped spillways has been through the physical modeling of several site-specific structures for other agencies. In addition, several general research efforts - including construction of a large near-prototype outdoor flume at Colorado State University – have mainly concentrated on overtopping protection for embankment dams. Development of a patented wedge-shaped block for overtopping protection on earth and rock-fill embankments built on this experience, using the hydraulic characteristics of a typical stepped overflow channel to help provide ease of construction, energy dissipation, block stability, and natural aspiration to help drain the underlayment. Reclamation's most recent laboratory studies have been associated with the Joint Federal Project at Folsom Dam near Sacramento California. A newly designed auxiliary spillway features a high-head top seal radial gate control structure which flows onto a smooth chute transitioning to a stepped section of constant slope (0.4026), terminating in a modified Type III stilling basin. Of particular concern with this structure is the cavitation potential at the maximum flow rate of 312,500 ft^3/s (8860 m^3/s), resulting in an exceptionally large specific discharge, q=1850 ft^2/s (172 m^2/s).

Introduction

While stepped spillways can be dated back to antiquity, the hydraulic design of stepped spillways has interestingly been an active topic for research over the past 50 years. Numerous studies in laboratories all around the world have added to the knowledge base in the field and have provided a broad-based perspective on design questions. There have been several notable papers and books that have in particular discussed hydraulic design guidance for stepped channels operating in the nappe and skimming flow regimes; Essery & Horner (1978), Pravdivets & Bramley (1989) , Chanson (1994a), Christodoulou (1999), Boes & Minor (2000), Matos (2000), Chanson (2001), Chanson (2002), Boes & Hager (2003b), Frizell (2006), Gonzalez & Chanson (2007), Hunt (2008), Renna & Fratino (2010), Meireles et al. (2012), Matos & Meireles (2014), and Chanson et al. (2015). These works have primarily focused on the hydraulics of stepped spillways. Many more studies have been performed that focus on specific features of the flow on stepped spillways.

This document will present generalized design guidance for stepped spillways with regards to their hydraulic characteristics and performance. It will pull largely from experiences at Reclamation; however, studies and experience from all over the world will be included in order to provide a broad perspective of the design and use of stepped spillways today. In terms of construction, the last 25 years have likely been the most active period in history for stepped spillways.

The guide will reflect the current state-of-the-practice, realizing that with passing time, improvements and changes in design philosophy may certainly affect the appropriateness of some of these contents. The structure of this document will present basic flow regimes found on stepped spillways and then discuss design details such as: crest treatments, step height selection, air entrainment of the flow, energy dissipation (on spillway and stilling basin), converging chutes and wall heights, and cavitation potential.

Flow Regimes on Stepped Spillways

Water cascading over a spillway crest onto a series of steps can be categorized into two basic flow regimes; nappe flow and skimming flow. There is a transitional band between these two regimes that may include features of each regime but in general should be avoided as a primary design condition. Nappe flow is defined as a series of free-falling sheets (or jets) of water tumbling from one step to the next. Depending on the channel slope, step dimensions, and the flow conditions, the formation of a hydraulic jump can occur on each step, further enhancing energy dissipation. The analysis and design of a spillway with nappe flow can be approached as a series of individual successive drop structures. Usually as the discharges increase or slopes steepen, the onset of skimming flow will exist. In skimming flow, the step tips form a virtual-boundary above where the flow skims in a reasonably coherent stream down the spillway, although highly turbulent and aerated over much of the length of the chute. Within the triangular cavities that exist below this virtual bottom and enclosed by the step boundaries, secondary flow in the form of three dimensional recirculating eddies fills the space, as shown by Matos (1999), Chanson (2002), Gonzalez & Chanson (2008), and Matos & Meireles (2014). This recirculation is maintained through exchange of energy with the skimming flow in a highly intense shear layer that exists along the line of the virtual boundary. The delineation of flow regimes on stepped spillways has also been presented in some other studies such as Chamani & Rajaratnam (1999a), Ohtsu et al. (2001), James (2001), Matos (2001), Chanson (2002),and Chanson et al. (2015). Figure 3 show data from numerous researchers and presents data as a function of d_c, the critical flow depth divided by the step height versus step height/step length (slope).

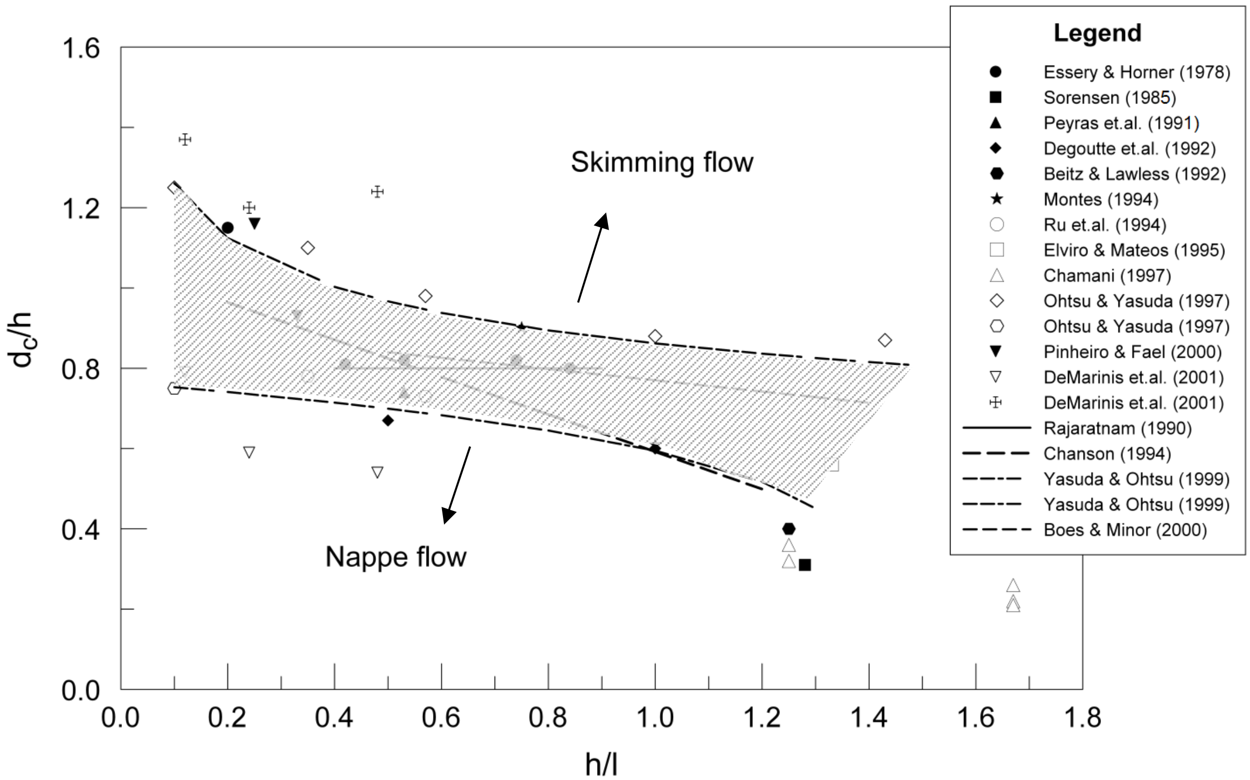

Figure 3: Summary graph showing existing data and demarcations between nappe and skimming flow regimes proposed by several researchers over many years of study.

Nappe Flow

The majority of spillways on dams generally do not operate in the nappe regime, sometimes also referred to as the jet regime. This regime is typically found with low discharges (thin flows) and/or relatively large step heights. While some spillways are designed in this manner, nappe flow is usually found on structures with small drops of limited discharge. Larger structures can experience nappe flow when beginning to operate at small discharges and flow depths. The tumbling nature of nappe flow can lead to enhanced energy dissipation; however the range of discharges typically will be limited unless the spillway length is such that the step treads are relatively long (figure 4).

There are various criteria available for the designer to use in the determination of flow regime. The large cross-hatched zone (fig. 3) between the two curves of Yasuda & Othsu (1999) is the region where the transition between nappe and skimming flow occurs. In order to insure a specific regime for the design, the advice is to keep design parameters away from this central region (transition zone) if possible. Recent studies of nappe flow characteristics have generally featured flatter slopes more typical of embankment-type dams, (Fratino, et al., 2000, Peruginelli & Pagliara 2000, Pinheiro & Fael 2000, André 2004, Fratino 2004, Toombes & Chanson 2008, and Renna & Fratino, 2010).

Figure 4: Spillway of the Robert Bourassa dam in Quebec, Canada. Excavated from rock, this unlined channel is almost 3280 ft (1 km) in total length, there are 10 steps each 32.8-ft-high (10-m) with tread lengths of up to 590 ft (180 m), and width of 400 ft (122 m), design discharge is 574,000 ft^3/s (16250 m^3/s).

Flow generally transitions from nappe to skimming flow as the discharge increases. The ability to predict the unit discharge where this transition occurs is important to the designer, particularly if there is concern with low-flow operations. Pinheiro & Fael (2000) present a good summary of equations developed by many researchers. As most stepped spillways of any height tend to operate in the skimming flow regime, the onset of skimming flow is perhaps the most interesting condition to understand. Transitional flow is that region where characteristics of nappe flow and skimming flow may exist in some combination on the spillway. There is still some debate on the conceptual definition of transitional flow. Chamani & Rajaratnam (1994) showed that for a limited range of spillway slope (20°- 40°), nappe flow occurred roughly up to $\boldsymbol{d_c/h}$ = 0.8. Chanson (1994b) proposed an equation for the onset of skimming flow for a range of slopes from 11.3° to 38.7°, showing higher required values of $\boldsymbol{d_c/h}$ as the spillway slope decreases:

$$\frac{d_c}{h} > 1.057 - 0.465\frac{h}{l} \quad . \tag{1}$$

A similar equation was proposed by Boes & Hager (2003b) for the range of slopes, 25° to 55°, with slightly less dependence on the slope than proposed by Chanson.

$$\frac{d_c}{h} = 0.91 - 0.14\frac{h}{l} \tag{2}$$

Skimming Flow

Skimming flow is that condition where water flows down the stepped surface in a somewhat coherent stream above a line connecting the tips of the steps. The zones within the triangular spaces beneath this virtual or pseudo "bottom" contain recirculating eddies that remain stable through the exchange of energy with the skimming flow in a highly intense shear layer just above the step tips. The steps behave as macro-roughness elements and are important in the dissipation of energy along the slope as well as being responsible for enhancing the self-aeration of the flow over that of a smooth chute of similar slope and specific discharge.

One of the most active research topics in stepped spillways in the past few decades has involved the skimming flow regime, and in particular detailed studies of the effects of highly aerated flow on frictional resistance and energy dissipation; two of the major parameters that interest spillway designers. There are distinct regions of skimming flow down a stepped spillway and interestingly they are similar to those seen in flow over spillways with smooth inverts, (Wood (1983, 1991)). Figure 5 shows these regions for a typical free-overflow crest.

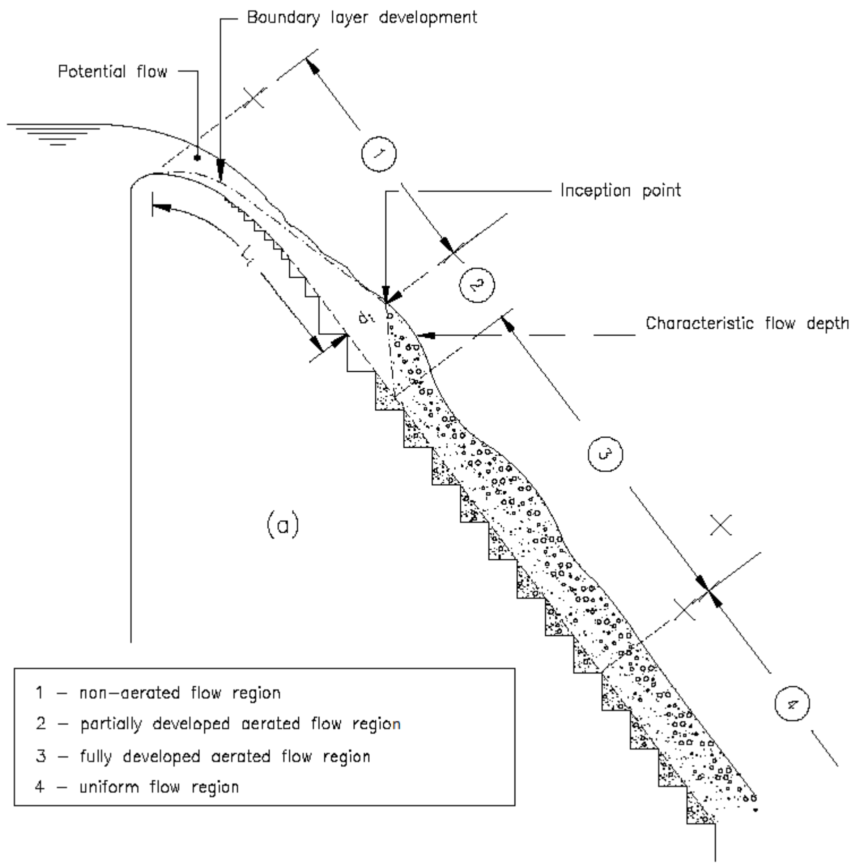

Figure 5: Skimming flow regions (adapted from Matos 1999).

These regions include: (1) the non-aerated region close to the crest, (2) the partially developed aerated flow region beginning at the inception point, (3) the fully developed aerated flow region, and (4) the uniform flow region where the depth, velocity, and air concentration are in equilibrium. Figure 6 shows important variables defining the steps, overlaid on a photograph of skimming flow in a scale model. Due to the aerated nature of skimming flow, the flow depth can be represented in alternate ways, the two most common are; $\boldsymbol{Y_{90}}$ - the depth where the air concentration equals 90-percent and $\boldsymbol{d_{cw}}$ - the equivalent clear water depth (i.e. the depth corrected by using the mean air concentration to remove the bulking (increased apparent depth) caused by the air entrainment), where:

$$d_{cw} = (1 - C_{mean})Y_{90} \tag{3}$$

with C_{mean} being the mean (depth averaged) air concentration.

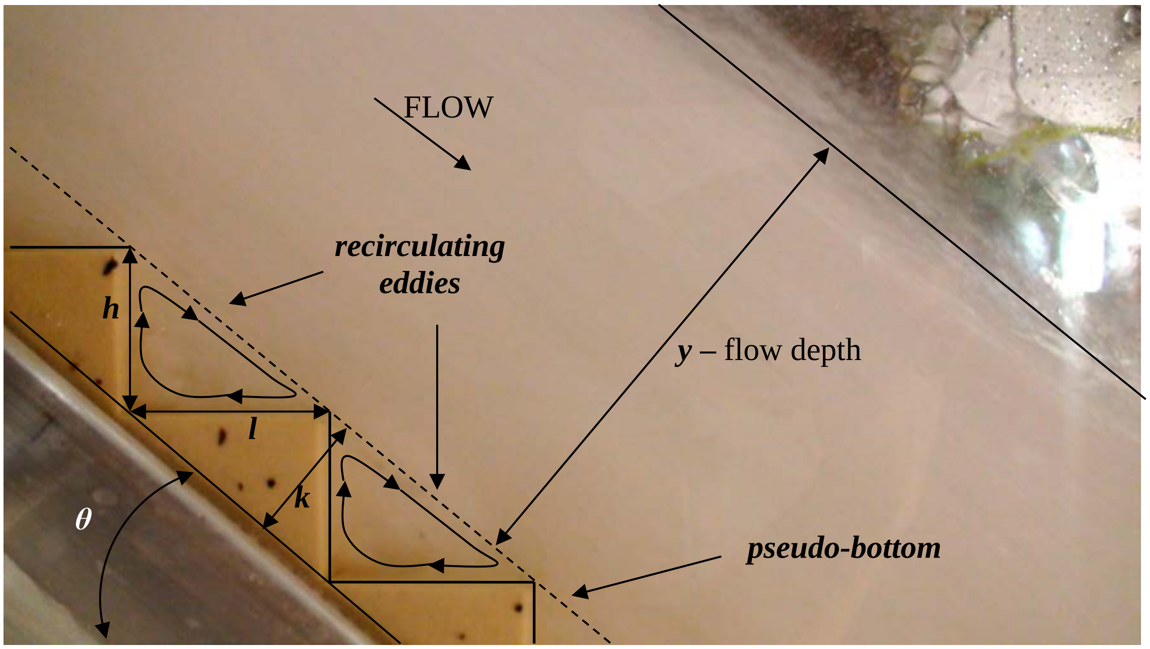

Figure 6: Skimming flow over a 0.8 to 1 (51.3º) stepped chute.

In the fully aerated uniform flow region, an important feature is the evaluation of the clear water depth. Both the aerated depth measured through the flume sidewalls and the characteristic depth Y_{90} show a wavy pattern of decreasing amplitude along the chute; however, this is not observed in the equivalent clear water depth. The equivalent clear water depth is much less than Y_{90} due to both the entrained and entrapped air effects in the depth where the air concentration is equal to 90-percent**.** This confirms that the friction factor and energy dissipation will be significantly overestimated if the aerated flow depth is used in the computation (Matos and Quintela (1995a,b)). In the uniform flow region, near the downstream end of the chute, both Y_{90} and the observed aerated depth become practically constant. Experimental evidence has shown that the equilibrium mean air concentration in skimming flows is similar to the mean air concentration in uniform aerated flow on smooth-invert chutes of identical slope (e.g., Ruff and Frizell (1994), Gaston (1995), Matos (2000), Boes and Hager (2003a)). Many researchers have also used a design parameter known as the roughness Froude number:

$$F_r = \frac{q_{cw}}{\sqrt{g sin\theta k^3}} \tag{4}$$

a function of the specific discharge, the step roughness height normal to the slope and the slope. Boes and Hager (2003a) described a modified roughness Froude number (F_*) using the vertical step height, h, instead of k. In this guideline, F_r will be used with $F_r = F_*/(cos\theta)^{3/2}$.

Crest Treatments

The most common crest treatments for stepped spillways are free uncontrolled overflows. This is especially true on smaller RCC structures but also is common on dams of significant hydraulic height. Uncontrolled crests generally take the form of a broad-crested weir (flat) or perhaps the more efficient ogee-shaped crest. Maximizing reservoir storage has also led to the use of fixed vertical weirs, both straight and labyrinth designs, placed on top of a broad crest; pneumatically controlled crest gates; and a variety of standard spillway gate designs, most commonly radial gates. Each of these crest treatments results in particular features that the designer must address with the cost not always being the deciding factor.

Free overflow

Free overflow crests are the most common crest treatment used for stepped spillways. Much of the modern research on this topic has detailed flow conditions on the steps resulting from a free uncontrolled overflow. This has resulted in many important performance design parameters being linked to the free overflow designs. In addition many of the transitional step treatments that have been developed are specific to free overflow crests, albeit specific to the crest shape. The crest shape generally varies with the slope of the spillway. The steeper the spillway slope the more likely some type of streamlined crest shape such as an ogee will be used.

Broad (or flat) crests are common to the designs of many stepped spillways, in particular on low RCC or embankment dams. Most of these structures have flows that pass through critical depth on the broad-crest and then enter the stepped portion of the chute. They typically have uniform step heights, but are prone to issues due to flow acceleration over the crest and the pressure distribution at the brink. Frizell et al. (1991) showed that for design heads from about 5- to 50-percent of the crest length in the flow direction, critical depth generally occurred in the last 25- to 33-percent of the crest length for any channel shape. Dodge (1988) investigated overtopping of embankments and noted that erosion typically begins just downstream from the brink of the crest and downstream slope. Frizell et al. (1991) investigated pressures at the brink of various fixed embankment slopes and noted from the point where critical depth occurred, the pressure profile begins decreasing and drops below hydrostatic conditions at the brink. Curvilinear flow exists as it nears the brink and the resulting minimum pressure occurs just over the brink, quickly returning to hydrostatic conditions on the sloping portion of the chute. More recently, Felder & Chanson (2012) and Zhang & Chanson (2015) provided very detailed studies on broad crests with stepped spillways.

More steeply sloped and higher spillways tend to have a more efficient crest shape such as a traditional ogee crest. Bureau of Reclamation (1987) and the U.S. Army Corps of Engineers (1977) provide equations that can predict the underside of the nappe profile in order to design the step profile for steeply sloped spillway chutes. The nappe equation for a vertical upstream dam face and low approach velocity is:

$$y = \frac{x^{1.85}}{2H_o^{0.85}} \tag{5}$$

where x and y are the horizontal and vertical coordinates from the apex of the crest and H_o is the head. A typical treatment for the steps downstream from the brink of the crest is to match the nappe profile (Sorenson 1985, Houston 1987, Frizell 1990, Bindo et al. 1993, and Mateos & Elviro 2000). Small steps are typically used in the upper part of the crest to prevent splashing at flows much lower than the design flow. The step heights then vary to match the nappe profile and remain constant once the point of tangency for the constant sloped section is reached. The first steps on the upper portion of the crest are generally smaller, in order to minimize splashing that may occur when the flow separates from the spillway surface then reattaches further downstream. This is especially prevalent at low flows, significantly less than the design conditions. Hanna & Frizell (1997) determined during testing of Buckhorn Dam that it was important to design the profile shape for less than the design head to prevent the flow from springing free near the top of a very steep sloped stepped spillway. During construction of a test section for the spillway on the Upper Stillwater Dam, it was determined that the crest needed to be widened in the streamwise direction, resulting in a very steep upper section of 0.32:1 that intersected the 0.6:1 slope 72 feet (22 m) down the chute . The first step's outer edge matched the estimated lower nappe profile. This profile was also designed for less than the design head. The next steps increasingly protrude into the theoretical nappe shape for about 10 ft, (3 m) below which the intersections of the risers and treads match the nappe curve. Although there have been no significant spills, thin overflows have proved to spring free and fall over many of the steps in this upper section without contacting the spillway surface.

Weirs

Weirs of various types are sometimes added to crests in order to increase storage capacity, mixed use flexibility, or provide added flood protection. The weirs can be fixed height and are often a shape that will provide increased discharge capacity as well, such as a labyrinth, figure 7-8. The addition of a Piano Key Weir (PKW), in particular on the top of a gravity dam where limited space makes a labyrinth an unviable solution, should be considered (Labyrinth and Piano Key Weirs – PKW 2011, Labyrinth and Piano Key Weirs II – PKW 2013). In general, there are not any special design parameters concerning the stepped chute that exist because of a specific type of weir at the crest structure. Several studies have looked at labyrinth weirs placed atop a broad crest with the discharge flowing down a stepped chute, Tullis & Crookston (2008). General guidance suggests providing some length of the flat crest downstream from the weir and before the steps begin to allow the flow to redirect onto the steps.

Figure 7: Labyrinth weir leading into a stepped chute, Standley Lake, Westminister, Colorado.

Figure 8: Labyrinth weir leading onto a stepped chute model, Lake Turner Dam, Texas. Photo from Utah State University Water lab website.

Gated Spillways

The use of traditional gated control structures on stepped spillways is fairly uncommon. There are relatively few examples that have been constructed that the designer can draw on for experience. Adjustable flap gates, such as the Obermeyer type, can be installed and used to enhance mixed use operations of a structure (figure 9). These can be designed into the spillway or can be retrofitted.

Lahontan Dam (figure 10) features a control structure with multiple radial gates. The Robert Bourassa dam spillway features a control structure with vertical lift gates (figure 4). These two examples are on stepped spillways that operate in the nappe flow regime where the flow coming from the gates is less critical than for a steeper stepped spillway that will have skimming flow. Amador et al. (2004) performed scale model studies of a stepped spillway controlled by a radial gate for conditions of underflow, overflow, and a combination of both. They recommended that the steps begin after the start of the uniform sloped portion of the chute. This allows for a more uniform flow condition to establish prior to the beginning of the steps. The important feature with gated flow is to realize that the Froude number of the flow is typically increased and thus many design parameters are affected, probably most important is the length down the slope until aeration begins. The Folsom Dam auxiliary spillway currently under construction is a hybrid spillway featuring a head structure with 6 top-seal radial gates under 98 ft (30 m) of head releasing onto a smooth chute 169-ft-wide (51.5 m) with a constant slope of 0.02 for a distance of 2000 ft (610 m). At that point the chute becomes stepped with a slope that parabolically increases to 0.4026 over a distance of 400 ft (122 m) downstream of the first step. Along this reach of increasing slope, the step heights also increase to the maximum offset of 3 ft (0.98 m) which is maintained for about 275 ft (83.8 m) down to a modified Type III stilling basin (figure 11).

Figure 9: Ohuha Dam, New Zealand. Obermeyer adjustable flap gates installed on existing crest.

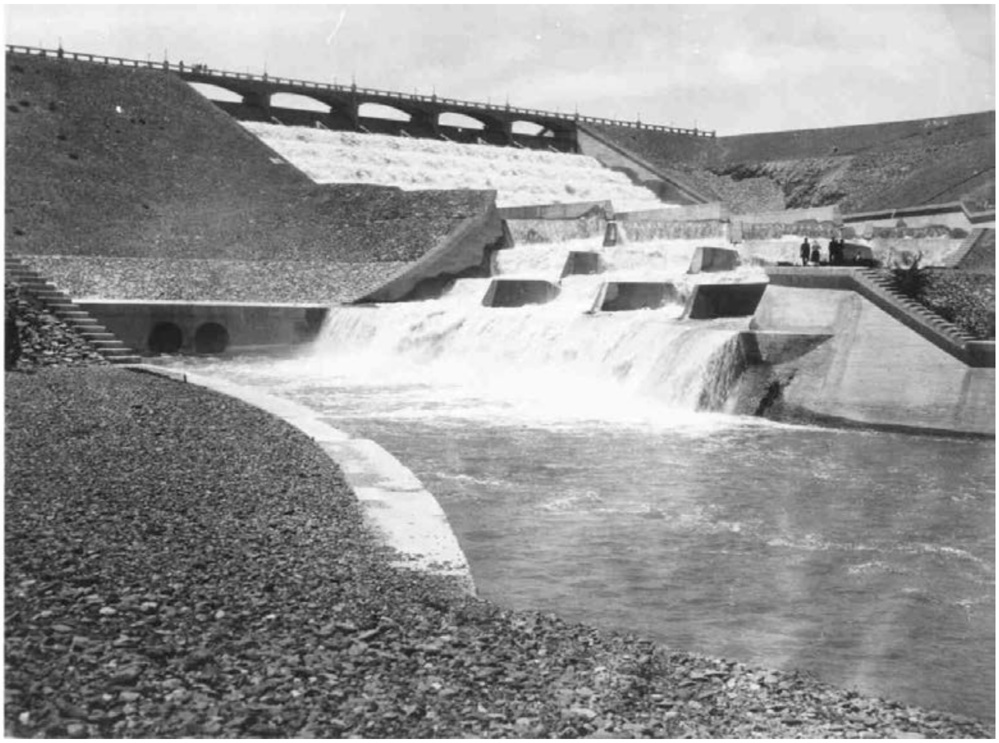

Figure 10: Lahontan dam spilling from a radial gate control structure.

Figure 11: Auxiliary spillway on the Folsom Dam, California under construction, April 2015.

Step Height Selection

The height of the steps has broad implications with regard to energy dissipation, flow resistance, flow regime, and attainment of uniform flow conditions. As critical as this design parameter may seem, the step height is not fine-tuned or optimized but often is a result of the construction technique used and cost effectiveness. Typically in the United States, step heights are constructed in even foot increments, i.e. 1-, 2-, or 3-ft-high (0.30-, 0.61-, 0.91-m). The slope of the chute often does not impact the step height, resulting in longer tread lengths on flatter slopes.

Stephenson (1991) determined that for a steep slope (0.7:1), the optimum step height in skimming uniform flow was $h = \frac{1}{3} d_c$. This recommendation was based on a negligible gain in energy dissipation for step heights greater than h. Others have confirmed that for skimming uniform flow, the step height is not an important design parameter (Boes & Hager, 2003b; Matos, 2000). Boes & Minor (2002) and Boes & Hager (2003b) describe the equivalent clear water depth on the spillway as a function of slope only, with no reference to the step height.

Ward (2002) studied 1- and 2-ft-high steps in a 2:1 large scale flume and found good agreement with a criteria developed by Tozzi (1992) for steeper slopes that described the optimum step height based on energy dissipation for a variety of unit discharges as $\frac{d_c}{h} = 3.33$. Using this criteria, no appreciable increase in energy dissipation is achieved by further increasing the step height.

Frizell et al., (2000) discuss the roughness height or step height relating to the friction factor and show that, for embankment dam slopes, there appears to be a constant friction factor versus the non-dimensional parameter of relative roughness, h/D_h. Ward (2002) found the same result on a 2:1 slope with 1 and 2 ft (0.30 and 0.61 m) high steps in a near-prototype flume study. Rau (1994) observed no appreciable difference in friction factor or energy dissipation with step height after performing flume studies of scaled 1 and 2 ft (0.30 and 0.61 m) prototype step heights.

If the design specific discharge is large or the height of the dam is low, then uniform flow will most likely not be attained. This is common for many small dam applications and it is difficult to estimate the effect of step height versus energy dissipation under those conditions (Chanson, 2001).

Air Entrainment

Previous studies have detailed the process of self-aeration of free surface flows on smooth chutes and spillways (Straub & Anderson, 1958; Cain, 1978; Wood, 1983, 1991; Wilhelms & Gulliver, 1994, Chanson 1997). These studies include measured air concentration profiles as a function of discharge, depth, and spillway slope. They include both laboratory and field measurements. The majority of the measurements were taken with conductivity-type probes that detect the void fraction of air bubbles traveling in the flow and this is how the air concentration is defined.

Air concentration plays a large role in determining the flow depth, velocity, and energy dissipation on a stepped spillway. Much of the impact is based on which depth is used to compute the energy-related parameters; the equivalent clear water depth or the aerated mixture depth. Many researchers have chosen to use the depth at which the air concentration is equal to 90% as an important parameter in the correlation of many features describing stepped spillway performance. In addition, many have found to compare stepped spillway data to that of smooth chutes, one must determine the equivalent clear water depth, essentially removing any bulking or increase in flow depth that is due to aeration.

Length to Inception of Aeration

The presence of steps on the chute or spillway greatly increases the roughness and in turn triggers self-aeration more quickly than on smooth surfaces. One of the most important design factors is the length down the chute until the inception of air entrainment (L_i). Many researchers have shown this factor to be important in defining flow depths and energy dissipation on the chute. Physical models of a variety of scales have been used to develop various formulas to predict this inception length. Much of the first work on this concept was based on Wood et al. (1983) and their relationship for smooth chute spillways with ogee crest entrances. Chanson (1994a, 2002) used this concept and developed a relationship for stepped spillways with ogee crest and steep slopes (≥26.6-degrees). Meireles & Matos (2009) discovered that Chanson (1994a, 2002) overestimates the inception length for $F_r \leq 10$ and optimized Chanson's relationship for spillways more characteristic of embankment dams. These predictions were only valid for a range of the roughness Froude Number of 1.9 to 10. Hunt & Kadavy (2011) further developed this work for stepped spillways as flat as 14-degrees with a range of F_r between 1 and 100. Hunt & Kadavy (2013) effectively extended the applicable range of inception point predictors for F_r up to 10^5 and also interestingly showed that Wood et al.'s (1983) relationship predicts L_i/k_s for stepped spillways with $28<F_r<10^5$ even though it was developed for ogee-crested smooth chute spillways. Using these latest tools along with data from large scale models (≥10:1) design guidance for embankment dams with or retrofitted with stepped spillways can be achieved.

All methods and equations show that the primary factor in the distance along the chute until the point of air inception is the discharge, with less dependence on the channel slope and step height. The steeper the slope the shorter the distance to air inception and obviously the steps will cause aeration sooner than on a smooth spillway

The mean air concentration at the point of inception has been investigated for both embankment dam and steeper gravity dam slopes. Matos (2000) developed the equation

$$C_{meani} = 0.163F_r^{0.154} \tag{6}$$

and determined that the average value of $C_{meani} \approx 0.2$ for steep slopes. Matos determined the point of inception as where the boundary layer reached the free surface. Boes & Hager (2003a) found that the mean air concentration at the point of inception, defined as the point with 0.01% air, for slopes of $26° \leq \theta \leq 55°$ was

$$C_{meani} = 1.2x10^{-3}(240° - \theta) \qquad (7)$$

Both equations show that the mean air concentration at inception is not dependent on step height, but only on the stepped spillway slope. For a slope of 53°, equal to the slope of experiments of Matos, equation 6 returns a mean air concentration at inception of 0.22 which agrees well with 0.20 found by Matos et al. (2000).

The mean air concentration at the point of inception may not be a significant parameter for estimated cavitation risk on a stepped spillway of significant height and design specific discharge as most of the air is entrapped and near the surface. Matos et al. (2000) and Boes & Minor (2000) provide a more reasonable estimate of safety against cavitation damage shortly downstream of the point of air inception.

Air Concentration Distribution

The distribution of air over a smooth spillway surface has been well documented by Straub & Anderson (1958), Cain (1978), Wood (1983, 1991), and Chanson (1997). Wood (1991) developed an equation for determining the mixture density or air concentration as a function of the slope, air concentration, turbulent diffusivity, and fall velocity of water droplets. Wilhelms & Gulliver (1994) presented a method to view the total conveyed air as the sum of the entrained air and the entrapped air. The entrained air is in the form of bubbles within the flow and the entrapped air is that carried along with the rough free surface of turbulent flow. Interestingly, the entrapped air is a constant amount of 23-percent for developed self-aerated flow.

The distribution of air over the pseudo-bottom of stepped spillways has been measured by many investigators (Ruff & Frizell 1994, Matos & Frizell 1997, Ohtsu & Yasuda 1997, Chamani & Rajaratnam 1999a, and Chanson et al. 2000). The distribution in terms of air concentration profiles have been compared to air concentration profiles over smooth spillways by Matos and Chanson with relatively good agreement. Chanson (1995, 2001) proposed an advective diffusion model for stepped spillways that is very similar to that of Wood (1983) for smooth chutes.

Renna et al. (2005) showed that in gradually and rapidly varied flow regions above the dimensionless depth $y/Y_{90}> 0.4$, there is good agreement with all proposed equations. The profiles also show the presence of an air concentration boundary layer of about 0.05 ft (0.015 m) near the step tips (Matos, 1999; Chanson, 1997). Chanson & Toombes (2002) developed an equation for skimming flow which is reported (Gonzalez & Chanson, 2004) to describe the air distribution of steeply sloping stepped spillways. In addition, Boes & Hager (2003a) showed good agreement with Chanson (1995) for spillway slopes from 30-50 degrees except very near the pseudo-bottom where $y/Y_{90} < 0.3$. In the region nearer the step tips the models of Wood (1983) and Chanson (1995) provide a better fit to the data than does Chanson & Toombes (2002) which tends to under predict the mean air concentration in this region.

Mean Air Concentration

Uniform or equilibrium flow conditions are important in regards to predicting the mean air concentration in skimming flow down a stepped spillway. Uniform flow is attained at the point on the spillway chute where the incremental flow depth, velocity, and air concentration are essentially constant. Matos (2000) presented data and developed equations to predict the mean air concentration in the rapidly and gradually varied flow regions but showed that in the uniform flow region; one could use the conventional smooth chute equations. Straub & Anderson (1958) proposed that the uniform mean air concentration was a function of slope and flow rate. Wood (1983, 1985) reanalyzed Straub & Anderson's (1958) data and found the equilibrium mean air concentration was a function of the slope only. Several researchers have shown in model and prototype flume studies that the mean air concentration of a stepped spillway is very similar to a smooth spillway once uniform flow has been attained. Hager (1991) presented a simplified equation for predicting mean air concentration in uniform flow:

$$C_{mean} = 0.75(\sin\theta)^{3/4} \qquad (8)$$

This mean air concentration can then be used to determine the equivalent clear water depth and the bulked depth for energy dissipation characteristics and training wall heights. Others have developed formulae for predicting the mean air concentration in uniform flow over smooth chutes that include both model and prototype data (Chanson 1997, Matos 1999, or model data in skimming flow over stepped chutes (Takahashi & Ohtsu 2012); however, the relative differences between these and Hager's simple formula are in general not significant for typical applications on steep slopes.

Uniform Flow Determination

Uniform flow can be an important and helpful condition to achieve in the design of a stepped spillway. Many researchers have proposed equations to predict whether flow has reached the equilibrium state in stepped spillways. While there is some variation between researchers, Boes & Hager (2003b) presented a simplified equation dependent only on dam height and critical flow depth, with uniform flow occurring when:

$$\frac{H_{dam}}{d_c} \geq (15\ to\ 20) \qquad (9)$$

Note that the step height is not included in the determination of uniform flow. In addition, Boes & Hager (2003b) present an equation to estimate the uniform equivalent clear water depth as a function of critical depth and spillway slope:

$$\frac{d_{cw}}{d_c} = 0.215(\sin\theta)^{-\frac{1}{3}} \qquad (10)$$

Matos et al. (2001a) used a non-dimensional length to inception parameter, $s' = (L - L_i)/d_i$, to describe when uniform flow had been achieved. They state when s' > 100, a uniform self-aerated flow regime is attained for both embankment and steep dam slopes. Once the designer has determined that uniform flow will occur, the mean air concentration for a stepped spillway of a given slope is equivalent to that of a smooth spillway and may be determined from equation 7.

If the design unit discharge is large or the height of the dam is low, then uniform flow will most likely not be attained. This is common for many small dam applications and it is difficult to estimate the effect of step height versus energy dissipation under those conditions (Chanson, 2001).

Energy Dissipation

One of the most attractive features of the selection of a stepped spillway over a smooth chute is the enhanced energy dissipation that takes place on the spillway chute itself. The benefit arises in a reduced amount of kinetic energy at the toe of the spillway and thus a smaller (shorter) stilling basin is required. Excavation costs and quantity of concrete and reinforcement are the major cost saving features of a smaller stilling basin. Flow regime is also an important feature in predicting energy dissipation. With nappe flow conditions, Pinheiro & Fael (2000) summarized and compared the energy dissipation theories developed by many researchers and found the equation presented by Chamani & Rajaratnam (1994) provided the best agreement among those evaluated:

$$\frac{\Delta E}{E} = \frac{\left\{(1-A)^N\left[1+1.5\left(\frac{d_c}{h}\right)\right]+\sum_{i=1}^{N-1}(1-A)^i\right\}}{N+1.5\left(\frac{d_c}{h}\right)} \tag{11}$$

where

$$A = \left[0.30 - 0.35\left(\frac{h}{l}\right)\right] - \left[0.54 - 0.27\left(\frac{h}{l}\right)\right] log\left(\frac{d_c}{h}\right)$$

In the nappe flow regime, the total energy loss is highly dependent on the number of steps. The total energy loss in nappe flow decreases as the number of steps increases for the same slope, dam height and discharge (Chanson, 1994a; Matos & Quintela, 1995b; Peruginelli & Pagliara, 2000). Therefore, for low dams with large steps in the non-uniform flow region, nappe flow produces more energy loss than skimming flow (André, 2004). As the dam height increases, the chute slopes to support nappe flow or skimming flow must be much different and although uniform flow and similar energy dissipation could be achieved with either arrangement, typically the steeper chute with skimming flow is chosen due to construction costs and space requirements.

Generally, designing a spillway specifically for nappe flow requires larger steps and flatter slopes. The construction costs are likely to be large and the more cost effective option would be to design for the skimming flow regime. Larger specific discharges are also becoming more common as a cost savings option. Nappe flow characteristics are then only significant because nappe flow will occur at lower flow rates that can occur frequently at a project, and might lead to excessive splashing and spraying. Some investigators have studied various structures attached to the steps to manipulate the turbulent flow field, using non-uniform step patterns or adverse slopes on the steps to force hydraulic jumps on the steps (André, 2004; Felder & Chanson, 2014; Gonzalez & Chanson, 2008; Essery & Horner, 1978; and

Peruginelli & Pagliara, 2000). These efforts have largely been untried, due to practicality and cost effectiveness from a construction standpoint.

Residual Energy and Friction Factor

One of the main reasons for designing a stepped spillway is to take advantage of the energy dissipation expected with flow over the steps. The initial investigations of stepped spillways showed so much promise for dissipating energy that designers wanted to pass larger unit discharges thus driving the research into the skimming flow regime over high dams.

Energy dissipation was reported in initial studies of stepped spillways as a function of the relative energy loss compared to smooth spillways or the theoretical available head such as Houston (1988), Stephenson (1991), Frizell (1992), and Chamani & Rajaratnam (1999b) to list a few. This trend continued for many years including discussions by Chanson (1994a,b), and Matos & Quintela (1995a). The preferred method of presentation for energy dissipation, which has evolved from these early comparisons to the theoretical available energy head, is to discuss energy dissipation in terms of residual energy head at the toe of the chute.

Residual energy is a function of the unit discharge or velocity and depth at the chute toe, and the chute slope. The velocity is a function of the chute roughness and dam height as well as the unit discharge. Most stepped spillway model studies have been conducted with energy dissipation or the amount of energy remaining in the flow, one of the most desired outcomes. Herein lays the problem – determining which studies have produced valid information. The friction factor is greatly influenced, and reduced, by air entrainment. The friction factor must be calculated with a non-aerated flow depth or clear water flow depth. Many studies have been conducted without aeration in the flow, thus over predicting the energy dissipation. Many studies have been conducted with aeration but not making proper measurements in the aerated flow, thus reporting aerated or mixture depths and generally over predicting the energy dissipation. Many studies have been conducted where uniform flow or quasi-uniform flow was not attained, but uniform flow equations were used to determine important parameters. As one might expect, results varied widely among the many model studies that have been performed, especially during the 1980-2000 time period.

Friction Factor

Critical to determining the residual energy below a stepped spillway is to understand the variables involved, namely the friction, drag, and aeration effects.

Chanson et al. (2000) state that the difference between smooth spillways and stepped spillways is that, on smooth spillways skin friction predominates and on stepped spillways form drag predominates. In skimming flow, the step tips form a pseudo-bottom parallel to the chute bottom over which the flow passes. Water skims over the step edges with formation of recirculating vortices between the main stream and the step corners. Momentum transfer occurs between the recirculating eddies in the step offsets and the main skimming flow increasing energy dissipation. On steep slopes, the recirculating eddies are the primary mechanism for form drag and energy dissipation, whereas on flatter slopes, the recirculating eddies produce form drag in addition to skin friction on the step surface and free-surface wakes.

Even though form drag predominates, the Darcy-Weisbach friction factor is the accepted parameter used to determine energy dissipation. The Darcy-Weisbach friction factor for air-water flow is given by:

$$f_e = \frac{8gS_f d_{cw}^2}{q_{cw}^2}\left[\frac{D_h}{4}\right] \tag{12}$$

where for a hydraulically wide chute, $D_h/4 \cong d_{cw}$, the equivalent clear water depth. This results in:

$$f_e = \frac{8gS_f d_{cw}^3}{q_{cw}^2} \tag{13}$$

for non-uniform gradually-varied flow. With uniform flow conditions, the friction slope equals the bottom slope so the following equation applies for skimming flow over hydraulically wide stepped spillway channels:

$$f_e = \frac{8gd_{cw}^3 \sin\theta}{q_{cw}^2} \tag{14}$$

The friction factor and residual energy in the flow must be determined using the clear water values for the depth and velocity even though the flow is highly turbulent and aerated.

The mean flow depth as measured by visual observation is of the same order of magnitude as the characteristic depth Y_{90}. The equivalent clear water depth d_{cw} is much lower than Y_{90}, confirming the significant overestimation of the friction factor and the energy dissipation based on the aerated flow depth measurement or visual observation, as suggested by Matos & Quintela (1995a,b). Near the downstream end of a long spillway chute, where uniform flow is attained, both Y_{90} and d_{cw} become practically constant.

However, making measurements in highly aerated flow is difficult and results can vary depending upon the techniques and methods used to perform the depth, air concentration, and velocity measurements. Many early investigations did not measure the air concentration, and determined depths that were most likely mixture depths or aerated flow depths. Using aerated flow depths to determine a friction factor produced greatly overestimated friction factor results in Christodoulou (1993), Rice & Kadavy (1996, 1997), and Stephenson (1991). Other times model scale factors were not adequate to produce aeration that would normally occur and friction factor results were again overestimated (Yasuda & Othsu, 1999). In addition, if the length or height of the facility was not adequate for uniform flow to be attained, the friction factors were generally also overestimated.

Residual Energy Head

Chow (1959) provides the following equation for residual energy for uniform and non-uniform flow over smooth chutes:

$$H_{res} = d_{cw}\cos\theta + \alpha\frac{q_{cw}^2}{2gd_{cw}^2} \tag{15}$$

with Chanson (1994b), Boes & Hager (2003a,b), Matos (2000), and Matos et al. (2001) developing the kinetic energy coefficient for skimming flow over stepped chutes α = 1.1 to 1.16.

In the uniform flow region, a direct computation of the residual energy head remaining under uniform flow conditions may be based upon the simpler computation of the uniform equivalent clear water depth provided by Boes & Hager (2003b):

$$\frac{d_{cw}}{d_c} = 0.215\ (\sin\theta)^{-1/3} \tag{16}$$

Equation 16 requires no direct knowledge of the friction factor. Figure 12 shows a relationship developed by Boes & Minor (2002) and Boes & Hager (2003b) in both the uniform and gradually varied flow regions using air concentration data and considering α=1.1, (Boes & Hager (2003b) equations 24a and 24b along with 4, 20 or 21 are the basis of figure 12).

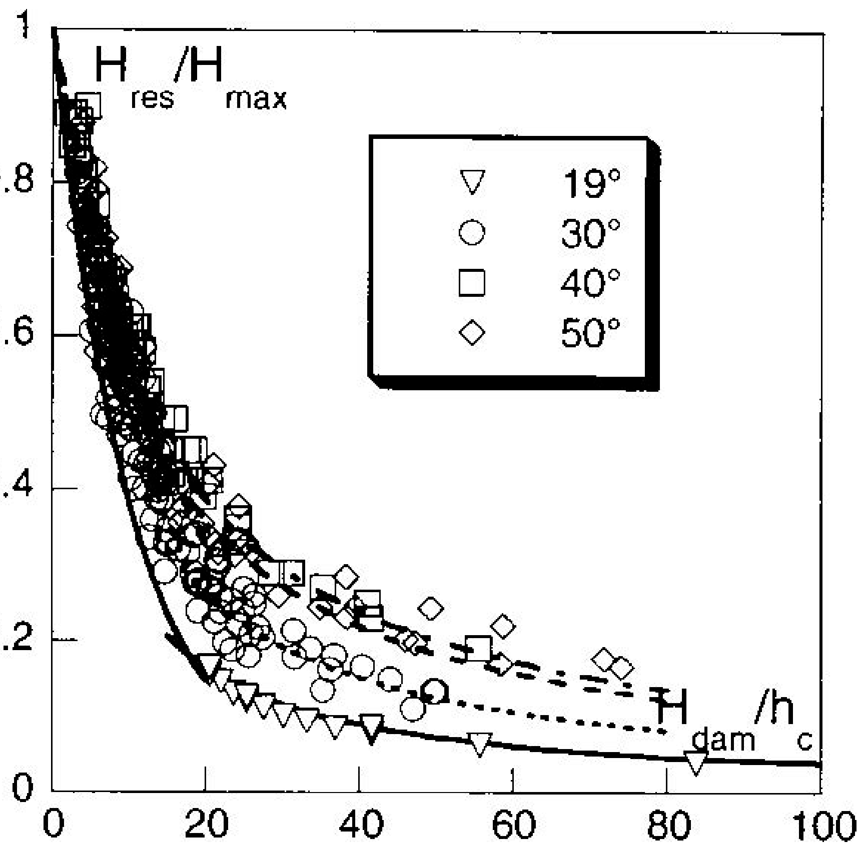

Figure 12: Relative residual energy head ratio H_{res}/H_{max} as a function of relative spillway height, H_{dam}/h_c. Data courtesy of Yasuda & Ohtsu, and Boes & Hager. (Courtesy of Boes & Hager (2003b) $h_c = d_c$).

While figure 12 shows the main parameters involved in determining residual energy head, use of Eq. 16 for a uniform mixture may be simpler. Additional hydraulic guidelines have been proposed in the last couple of decades (e.g. Matos 2000 and Chanson 2001, for concrete gravity dams, Chanson et al. 2015) and may warrant further review by the designer.

Non-uniform flow conditions often exist when small dams are considered. Non- uniform flow will exist when terminal velocity and fully aerated flow conditions are not attained. These flow conditions often exist when a large unit discharge is passed by a low dam or even a relatively high dam if the unit discharge is very large. Most general lab and near-prototype studies have determined flow velocities, depths, and air concentration for uniform or quasi-uniform flow conditions. In addition, many site specific models have not had an adequate model scale to allow correct modeling of aeration characteristics (Ohtsu & Yasuda, 1997, Yasuda & Ohtsu, 1999). There have been several site specific studies performed with no aeration under the design unit discharge, such as those by Rice & Kadavy (1996, 1997) for Salado Creek and Cedar Run 6 dams where uniform flow was also not attained as indicated by the velocity profiles. Also, studies by Christodoulou (1993), and Chamani (1997) and Chamani & Rajaratnam (1999b) most likely did not reach uniform flow as indicted by large friction factors and low spillway heights.

Hunt & Kadavy (2010a) present a simple equation for determining the relative energy loss at any point upstream from the point of inception of air entrainment, developed for typical embankment dam slopes (4H:1V) showing energy loss increasing from 0 at the crest to 30-percent at the point of inception:

$$\frac{\Delta H}{H_o} = 0.3\frac{L}{L_i} \tag{17}$$

They expanded their work in Hunt & Kadavy (2010b) to include the loss predictions from the inception point towards the toe of the spillway:

$$\frac{\Delta H}{H_o} = 1 - \left[\frac{L}{L_i} + 0.51\right]^{-0.87} \tag{18}$$

This work shows the energy loss increased from 30-percent for L/L_i=1 to 73-percent for L/L_i=3.5. Meireles & Matos (2009) presented similar results for slightly steeper slopes (2H:1V) in the developing region prior to the inception point of air entrainment resulting in an energy loss of 26-percent at the inception point. Meireles et al. (2012) looked at the non-aerated portion on stepped spillways with a slope of 0.75H:1V more typical of RCC/concrete gravity dams and showed the normalized specific energy could be approximated by:

$$\frac{H}{H_{max}} = 1 - 0.315\left(\frac{L}{L_i}\right) \tag{19}$$

resulting in an energy loss of 32-percent at the inception point.

Stilling Basins

One of the major benefits of choosing a stepped spillway over a smooth chute is that the stilling basin size can generally be reduced given similar operating parameters. The size reduction of the stilling basin can often translate to large cost savings. These savings are usually due to reduced excavation and reinforced concrete required. The energy dissipation that takes place on the stepped chute can be considerable, up to 3 times larger than that of a smooth chute. Reclamation designed a standard Type I stilling basin for use on a stepped spillway for the Upper Stillwater Dam (Houston, 1987). The energy dissipation credited to the stepped chute allowed the 600-ft-wide (183 m) hydraulic jump stilling basin to be shortened from an estimated 200 ft (61 m) to 30 ft (9 m), resulting in a significant cost savings. Prior to that, Lahontan Dam, completed in 1915, utilized a small stilling basin located central to the embankment. Both spillways and the outlet works use this basin and the size is quite small as a result of the spillways operating in the nappe flow regime.

Design guidance for stilling basins on stepped spillways is still somewhat lacking. Much of the previous work has concentrated on the residual energy remaining in the flow at the end of the chute/or start of the basin. There have been several site specific related model studies that have looked a stilling basin type and size for stepped spillway design. The most anticipated question is whether prior research on stilling basins for smooth spillways can be used for stepped chutes and if so what are the appropriate design parameters that should be included. Frizell & Svoboda (2012) tested three slopes of smooth and stepped chutes terminating in a standard Type III stilling basin. The design parameters recommended in Reclamation's Engineering Monograph No. 25 (1978) suggest using the full sequent depth of the hydraulic jump, even with the placement of chute blocks and baffle blocks in the standard Type III basin. Using measured mean air concentration profiles from the model data, the incoming bulked depth was adjusted to an equivalent clear water depth. The mean velocity was calculated based on this depth and the measured discharge. The stilling basin with both smooth and stepped chutes performed adequately at 15- to 20-percent of the D_2/D_1 ratio when compared to when D_2 equals the sequent depth. In addition at incoming Froude numbers less than 6, the stepped channel maintained the hydraulic jump at significantly less tailwater than its smooth counterpart.

Simões, et al. (2010) reanalyzed numerous data to look at resistance effects on type I stilling basin lengths. They concentrated on steep stepped and smooth chutes with angles between 45- and 59-degrees. They used typical parameters of Bureau of Reclamation (1978) and presented a correlation of the Darcy-Weisbach friction factor as a function of the Froude number at the toe of the spillway (i.e. entering the stilling basin). Once again, clear water parameters for depth and velocity were used in this presentation, fig. 13. This figure shows the considerable difference in the friction factor for a stepped channel versus a smooth one for the same Froude number at the beginning of the stilling basin.

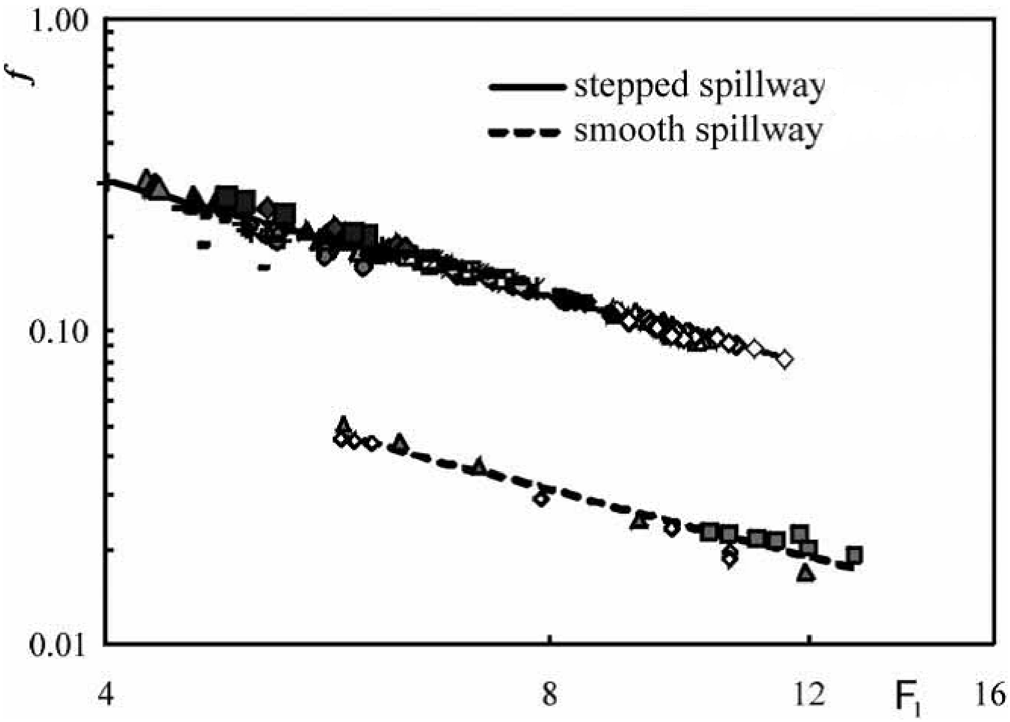

Figure 13: Friction factor as a function of Froude number entering the stilling basin. Clear water parameters were used in this analysis by Simões, et al. (2010). (Courtesy of Simões, et al. (2010)).

Others have investigated stilling basin design parameters with some interesting findings but without any true degree of standardization (Chanson 2001, Gonzalez & Chanson 2007, Cardoso et al. 2007). Some standardization regarding type III stilling basins has been achieved in the works of Frizell et al. (2009), Meireles et al. (2010), Bung et al. (2012) and Frizell & Svoboda (2012).

Sidewall Design

Chute dimensions are generally a product of constraints with the site geometry, discharge properties and economics. Specific discharges have generally been low for most stepped spillways designed in the past. This results in lower flow depths, even with the bulked flow depths caused by induced aeration. The option to narrow the spillway chute and increase the depth and specific discharge has received some attention but the uncertainties regarding possible cavitation damage in the zone prior to the inception point have resulted in only a handful of designs with large initial flow depths. In addition, there is the option to converge the sidewalls to allow for tighter terminal discharge dimensions and smaller width stilling basins. The converging sidewalls cause issues of their own resulting in increased depth of flow at the sidewalls and shockwaves or formation of oblique hydraulic jumps along the sidewalls.

Parallel walls

The design of parallel vertical sidewall on a stepped spillway chute is mainly an exercise in predicting flow depths along the chute so that flow will be contained by the training walls. Flow bulking or increased depth due to air entrainment occurs both on smooth or stepped chutes. Air entrainment is generally accelerated on a stepped chute versus a smooth chute due to the extremely rough boundary. The height of the chute sidewall (h_w) is usually based on a "safety factor" multiplier to the uniform mixture depth $Y_{90,u}$, $h_w = nY_{90}$. Ohtsu et al. (2004) suggested n-1.4 based on the experiments of Boes & Minor (2000) using the ratio Y_{99}/Y_{90}. The uniform mixture depth $Y_{90,u}$ is given by

$$\frac{Y_{90,u}}{h} = \frac{1}{2} F_*^{(0.1 tan\theta + 0.5)} \tag{20}$$

where h is the step height, F_* is the alternate roughness Froude number and θ is the angle of the chute to the horizontal. Boes & Hager (2003b) suggested using safety factors of 1.2 for concrete dams or locations not subject to possible erosion and 1.5 for emergency spillways and embankment dams or sites prone to erosion. Ward (2002), Meireles et al. (2007) and Matos & Meireles (2014) have shown that the maximum depths can be considerably larger than those proposed by others, reaching as high asY_{99}/Y_{90} of 1.9 for small normalized critical depths..

Converging walls

Converging sidewalls on a spillway chute with supercritical flow results in formation of an oblique hydraulic jump whether the chute is smooth or stepped (Chow 1959; Boes & Hager 2003b). Beginning in the 1990's researchers used physical models to evaluate converging sidewalls on stepped spillways (Frizell 1990; Hanna & Pugh 1997; Robinson et al. 1998). The studies generally showed that increased wall heights were required to contain the flow and velocities increased due to decreased energy dissipation. Hunt (2008) provides the most complete studies, generalizing design guidelines for stepped spillways with vertical training walls converging between 0 and 70-degrees. The models used in Hunt (2008) were relatively flat spillway slopes (3:1 H:V). Others also studied sloped training walls (André et al. 2005, Woolbright 2008) or sloped training walls with steps (Frizell 2006).

The most recent advances in the prediction of the minimum training wall height involve applying a simplified momentum analysis. Hunt et al. (2012) presented improvements to the method first presented by Hunt (2008) that should be applicable to a variety of chute slopes, step heights, and angles of convergence. The equations presented do not include the bulking effects of air-entrained flow, in addition the adjustments due to the force on the control volume associated with the weight of water have an empirical component and thus the extent of its application may need further testing.

Cavitation Potential

Cavitation damage to stepped spillways has not been reported. However, with increased use and the desire to pass larger specific discharges over the stepped surfaces there are still considerable unknowns about whether cavitation can form and at what conditions. For the stepped spillway designer an important question is can any of the traditional smooth chute spillway design principles regarding cavitation mitigation be applied? The uncertainty surrounding predicting cavitation potential on stepped spillways has perpetuated conservative design practices.

The flow regime that a stepped spillway operates in is an important parameter that determines whether there is a possibility for cavitation to occur. If a spillway operates in the nappe flow regime there is little chance for cavitation to form. The highly aerated flow conditions associated with nappe flow, even with large specific discharges, are not conducive to cavitation formation. Skimming flow forms a highly intense shear layer along the line connecting successive step tips also called the pseudo-bottom), figure 14.

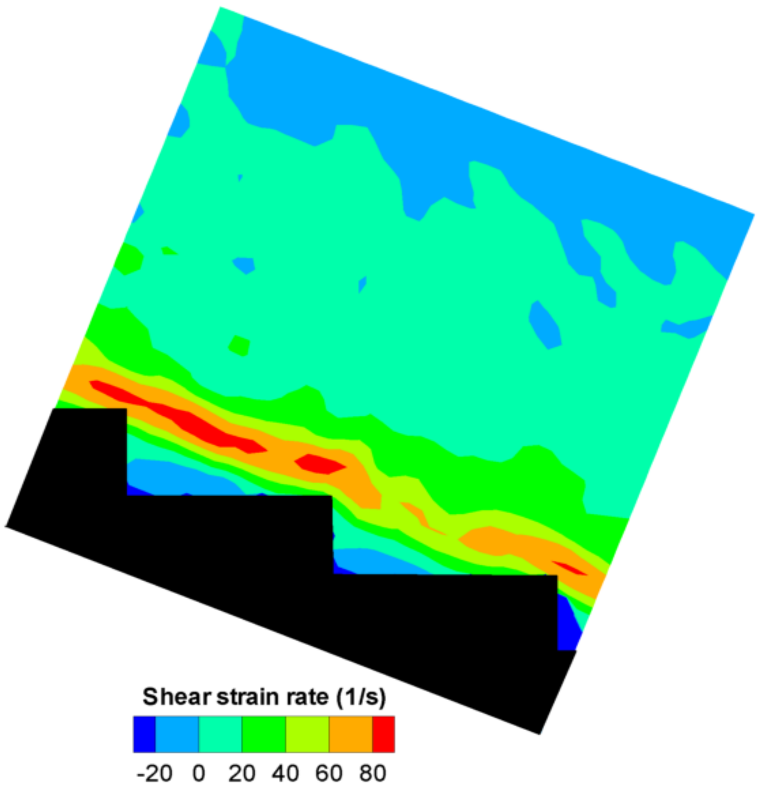

Figure 14: Shear strain rate from PIV measurements on a stepped channel.

Flow structure within this shear layer supports formation of cavitation along secondary flow features with many similarities to other types of cavitating shear flows, including plane shear layers (O'Hern 1987; Baur & Köngeter 1998; Iyer & Ceccio 2002) and submerged jets (Arndt & Taghavi 1984).

There have been several approaches over the years to address cavitation potential on stepped spillways. Matos et al. (2000) estimated the critical cavitation index at the air entrainment inception point on steeply sloping chutes based on the relation proposed by Arndt & Ippen (1968) for a uniformly distributed roughness, along with a reanalysis of friction factor data of Tozzi (1992). The resulting empirically based formula estimated the cavitation index at the inception point as a function of the step height and specific discharge. Boes & Hager (2003b) reported a critical velocity for cavitation inception in the flow prior to air entrainment of approximately 66 ft/s (20 m/s). Based on the possibility that this velocity could be reached prior to the initiation of air entrainment for a range of slopes and step heights, they recommended limiting design specific discharges to ~ 269 ft²/s (25 m^2/s). Pfister et al. (2006) proposed treating each step as a singular bottom irregularity with risk of cavitation. Based on this treatment ($\sigma_c \approx 1.0 \pm 0.1$) and pressure measurements on the first step, they presented scaled prototype (h = 4 ft (1.2 m)) results showing the cavitation index is below 0.9 on the first step when q >323 ft^2/s (30 m^2/s). Amador et al. (2009) recommended a mean velocity of 50 ft/s (15 m/s) at the inception point based on a 0.1% probability of extreme negative pressures measured near the edge of the vertical step faces on a steeply sloping chute. A similar negative pressures approach was adopted by Gomes (2006), on the basis of which a pseudocritical cavitation index was estimated as a function of the distance along the spillway. Critical unit discharges and mean velocities were also proposed for typical stepped spillway geometries (Gomes 2006; Gomes et al. 2007). This resulted in recommendations to limit the specific discharge from 124 ft^2/s to 150 ft^2/s (11.5 m^2/s to 14 m^2/s) for a steep (51.3°) stepped spillway with step heights of 2 ft and 4 ft (0.6 m and 1.2 m), respectively. Frizell et al. (2013) presented data for incipient cavitation indices for 2 slopes and 2 step heights based on tests performed at reduced ambient pressure in a specialized facility. The correlation of stepped chute data followed the existing work of Arndt & Ippen (1968), showing that the critical cavitation parameter was approximately 4 times the friction factor, figure 15. These tests were motivated by a desire to evaluate a new planned stepped spillway with specific discharges ranging from about 3 to 15 times the previous recommended values (a maximum of 1900 ft^2/s (176.5 m^2/s)). The use of specific discharge as a design recommendation seemed misguided without knowing the actual geometric and flow conditions of when and if cavitation will form. These tests provided the first direct measurements for the critical cavitation index for a stepped channel.

Using this correlation the designer only needs to specify basic parameters such as slope of the spillway and step height, calculate a friction factor from any number of methods presented in the literature then apply the correlation, $\sigma_c = 16\,C_f = 4f$, to predict the critical cavitation index. The cavitation indices of the flow can be computed along the spillway length, particularly in the nonaerated portion and compared to the critical index. If the flow index drops below the critical index prior to the length along the spillway where air entrainment begins then possible design features such as aeration ramps or specialized ramp/pier designs in a gated spillway (Guo et al. 2003, He & Zeng, 1995) should be considered.

Pfister et al. (2006) proposed placing a ramp on the first step in order to trigger aeration at the beginning of the spillway. This placement was largely the result of assuming the critical cavitation index was 1, which could occur near the beginning of the spillway. Zamora et al. (2008) further investigated step aerators, concentrating on reducing the spray created at low flows while still offering cavitation protection at high flows. Lesleighter et al. (2014) presented model study results of a large stepped spillway with an aerator installed to counter possible cavitation damage. Transient pressures measured on the steps during the model study indicate that vapor pressure could be reached making formation of cavitation a possibility. While transient step pressures can certainly support cavitation formation, the secondary flow characteristics within the shear layer traveling along the pseudo-bottom is likely the major contributing factor to the possibility of cavitation formation and any resulting damage.

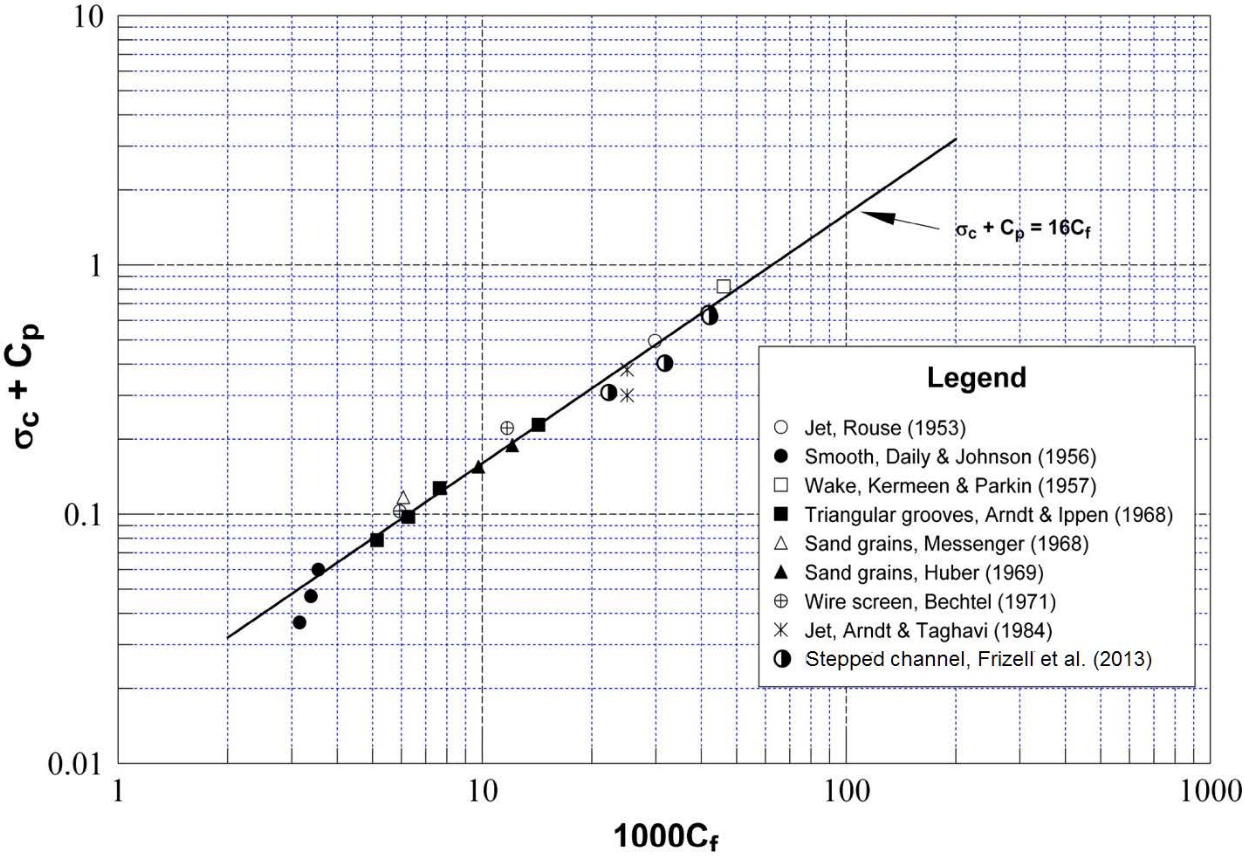

Figure 15: Correlation of coefficient of friction ($C_f = f/4$) as function of critical cavitation index; data for steps and other wall-bounded and free shear flows (Frizell et al. 2013).

Conclusions

This compilation of information is an attempt to present relevant design information in regards to step spillways of all kinds. Much of the research emphasis has been on skimming flow and thus, the most complete and broad design data and studies have been in this area. In recent years, a concentration of work has been on less steep spillways more characteristic of embankment dams. No matter the construction technique used to form the steps; RCC, slip-formed or curbing machines, traditional formed reinforced concrete, hand or machine placed concrete blocks, or even the masonry blocks of old, the design information contained herein should be applicable.

The past 50 years has likely been the most active time for the study and construction of stepped spillways and chutes. Many new structures have been completed and are successfully operating. New challenges, such as higher dams, much larger specific discharges, and need for gated control, continue to challenge spillway designers. A new class of stepped spillways awaits verification of new ideas through operation, especially the large specific discharge spillways that have recently been planned, are in construction, or have recently been completed and are now operating.

Design guidance is just that. This is not a cookbook on how to design the perfect stepped spillway, yet it should give the designer the insight to properly consider the many facets involved in a successful design. The information included is largely what has been used by Reclamation engineers and researchers over the last 40 years. However, as you can see from the list of references this work has been a global effort with many countries heavily involved in research and testing of stepped spillways as well as consulting on design and construction of new projects. As with any document of this kind, new research, new operational experience, new failures, all may have an impact on the usefulness and validity of certain portions of this guide and ultimately the designer is responsible for proper use of any and all tools at their disposal.

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