Design of weirs and spillways 19

19.1 INTRODUCTION

19.1.1 Definitions: dams and weirs

Dams and weirs are hydraulic structures built across a stream to facilitate the storage of water.

A *dam* is defined as a large structure built across a valley to store water in the upstream reservoir. All flows up to the probable maximum flood must be confined to the designed spillway. The upstream water level should not overtop the dam wall. Dam overtopping may indeed lead to dam erosion and possibly destruction.

A conventional *weir* is a structure designed to rise the upstream water level: e.g. for feeding a diversion channel. Small flow rates are confined to a spillway channel. Larger flows are allowed to pass over the top of the full length of the weir. At the downstream end of the weir, the kinetic energy of the flow is dissipated in a dissipator structure (Figs 19.1a and 19.2a and Plate 30).

Another type of weirs is the *minimum energy loss (MEL) weir* (Figs 19.1b and 19.2b). MEL weirs are designed to minimize the total head loss of the overflow and hence to induce (ideally) zero afflux. MEL weirs are used in flat areas and near estuaries (see Appendix A4.2).

Practically, the differences between a small dam and a conventional weir are small, and the terms 'weir' or 'small dam' are often interchanged.

19.1.2 Overflow spillway

During large rainfall events, a large amount of water flows into the reservoir, and the reservoir level may rise above the dam crest. A *spillway* is a structure designed to 'spill' flood waters under controlled (i.e. *safe*) conditions. Flood waters can be discharged beneath the dam (e.g. culvert and bottom outlet), through the dam (e.g. rockfill dam) or above the dam (i.e. overflow spillway).

Most small dams are equipped with an overflow structure (called spillway) (e.g. Fig. 19.3). An overflow spillway includes typically three sections: a crest, a chute and an energy dissipator at the downstream end. The *crest* is designed to maximize the discharge capacity of the spillway. The *chute* is designed to pass (i.e. to carry) the flood waters above (or away from) the dam, and the *energy dissipator* is designed to dissipate (i.e. 'break down') the kinetic energy of the flow at the downstream end of the chute (Figs 19.1a and 19.2).

A related type of spillway is the drop structure. As its hydraulic characteristics differ significantly from that of standard overflow weirs, it will be presented in another chapter.

Notes

1. Other types of spillways include the Morning-Glory spillway (or bellmouth spillway) (see Plate 31, the Chaffey dam). It is a vertical discharge shaft, more particularly the circular hole form of a drop inlet spillway, leading to a conduit underneath the dam (or abutment). The shape of the intake is similar to a Morning-Glory flower. It is sometimes called a Tulip intake. The Morning-Glory spillway is not recommended for discharges usually greater than 80 m^3/s.

2. Examples of energy dissipators include stilling basin, dissipation basin, flip bucket followed by downstream pool and plunge pool (Fig. 19.3).
3. At an MEL weir, the amount of energy dissipation is always small (if the weir is properly designed) and no stilling basin is usually required. The weir spillway is curved in plan, to concentrate the energy dissipation near the channel centreline and to avoid bank erosion (Fig. 19.2b and Appendix A4.2).
4. MEL weirs may be combined with culvert design, especially near the coastline to prevent salt intrusion into freshwater waterways without upstream flooding effect. An example is the MEL weir built as the inlet of the Redcliffe MEL structure (Appendix A4.3).

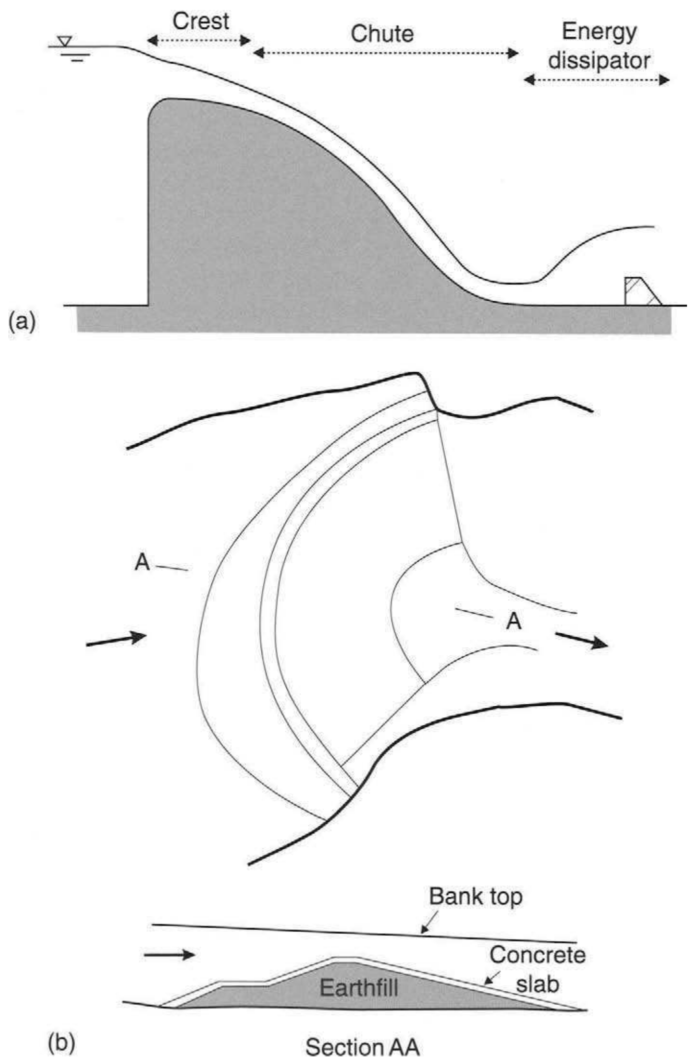


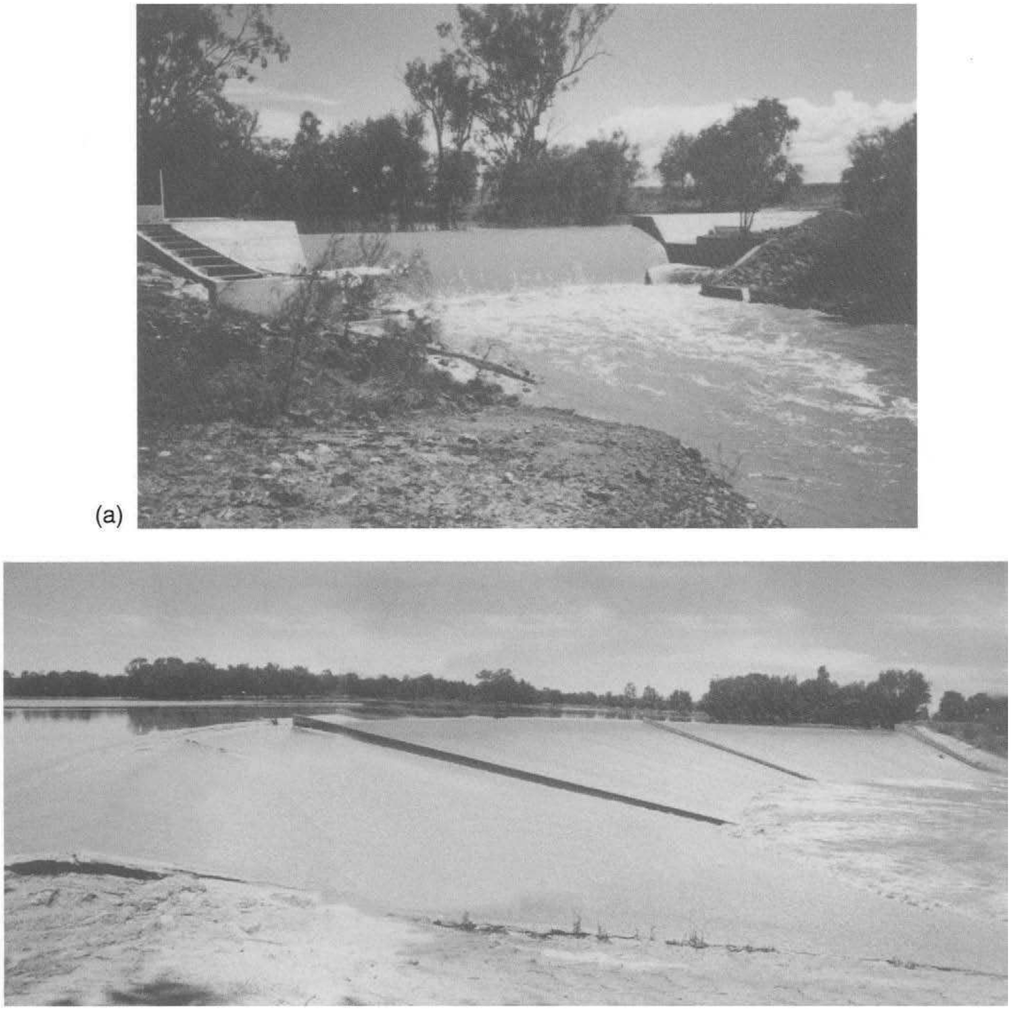
Fig. 19.1 Sketch of weirs: (a) conventional weir and (b) MEL weir.

19.1.3 Discussion

Although a spillway is designed for specific conditions (i.e. design conditions: Q_{des} and H_{des}), it must operate safely and efficiently for a range of flow conditions.

Design engineers typically select the optimum spillway shape for the design flow conditions. They must then verify the safe operation of the spillway for a range of operating flow conditions (e.g. from $0.1Q_{des}$ to Q_{des}) and for the emergency situations (i.e. $Q > Q_{des}$).

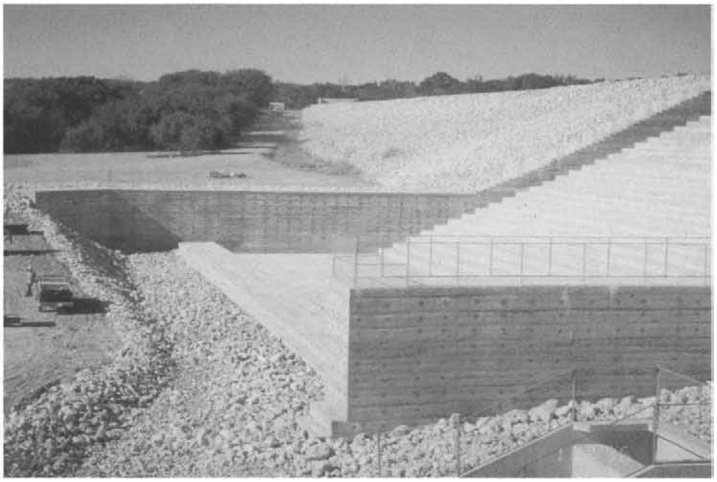
In the following sections, we present first the crest calculations, then the chute calculations followed by the energy dissipator calculations. Later the complete design procedure is described.



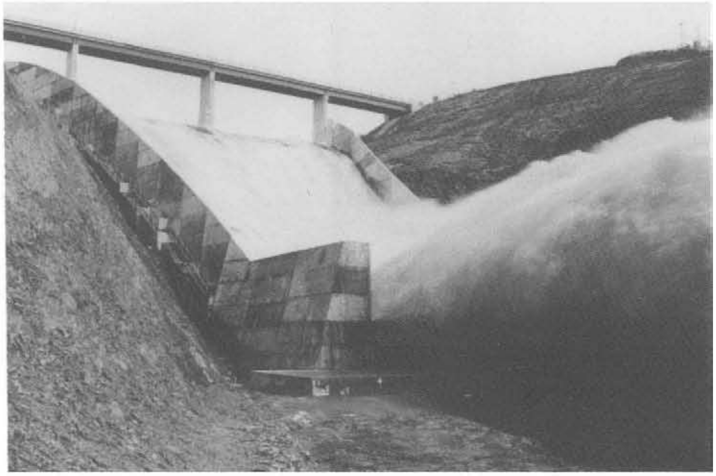
(a) (b)

Fig. 19.2 Examples of spillway operation: (a) Diversion weir at Dalby QLD, Australia on 8 November 1997. Ogee crest followed by smooth chute and energy dissipator (note fishway next to right bank). (b) Overflow above an MEL weir: Chinchilla weir at low overflow on 8 November 1997. Design flow conditions: 850 m^3/s, weir height: 14 m and reservoir capacity: $9.78 \times 10^6 m^3$.

(a)



(b)



(c)

Fig. 19.3 Examples of spillway design: (a) Overflow spillway with downstream stilling basin (Salado 10 Auxiliary spillway) (courtesy of USDA natural resources conservation service). (b) Overflow spillway with downstream flip bucket (Reece dam TAS, 1986) (courtesy of Hydro-Electric Commission Tasmania). Design spillway capacity: 4740 m^3/s, overflow event: 365 m^3/s. (c) Stepped spillway chute (Loyalty Road, Australia 1996) (courtesy of Mr Patrick James). Design spillway capacity: 1040 m^3/s, step height: 0.9 m, chute slope: 51° and chute width: 30 m.

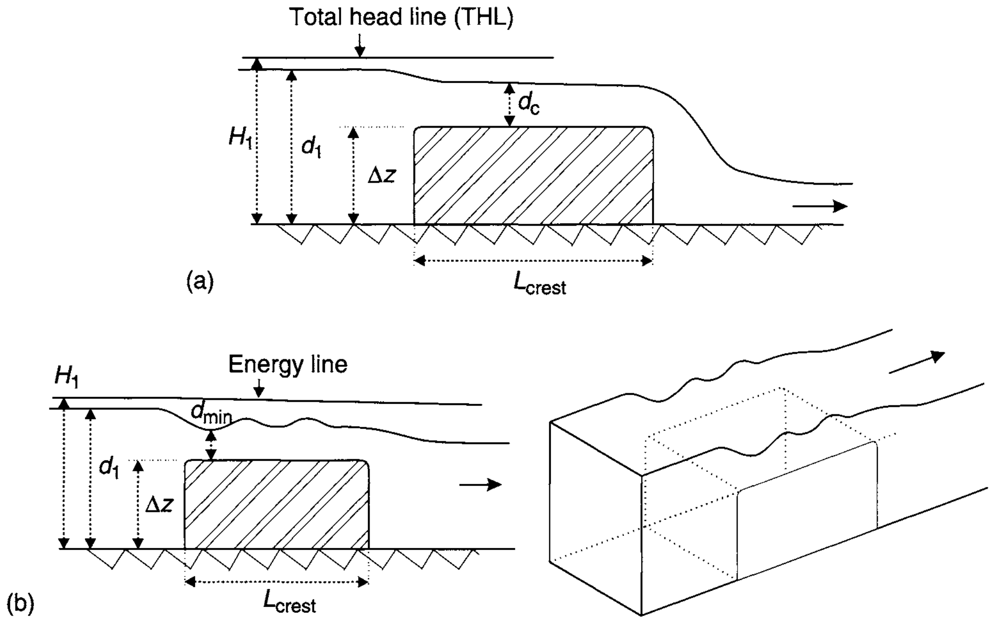


Fig. 19.4 Flow pattern above a broad-crested weir: (a) broad-crested weir flow and (b) undular weir flow.

19.2 CREST DESIGN

19.2.1 Introduction

The crest of an overflow spillway is usually designed to maximize the discharge capacity of the structure: i.e. to pass safely the design discharge at the lowest cost.

In open channels and for a given specific energy, maximum flow rate is achieved for critical flow conditions[1] (Bélanger, 1828). For an ideal fluid overflowing a weir (rectangular cross-section) and assuming hydrostatic pressure distribution, the maximum discharge per unit width may be deduced from the continuity and Bernoulli equations:

$$q = \sqrt{g}\left(\frac{2}{3}(H_1 - \Delta z)\right)^{3/2} \qquad \text{Ideal fluid flow} \tag{19.1}$$

where g is the gravity acceleration, H_1 is the upstream total head and Δz is the weir height (e.g. Fig. 19.4). In practice the observed discharge differs from equation (19.1) because the pressure distribution on the crest may not be hydrostatic. Furthermore, the weir geometry, roughness and inflow conditions affect the discharge characteristics (e.g. Miller, 1994). The real flow rate is expressed as:

$$q = C_D\sqrt{g}\left(\frac{2}{3}(H_1 - \Delta z)\right)^{3/2} \tag{19.2}$$

where C_D is the discharge coefficient.

[1]Flow conditions for which the specific energy (of the mean flow) is minimum are called critical flow conditions.

The most common types of overflow weirs are the broad-crested weir, the sharp-crested weir and the ogee-crest weir. Their respective discharge characteristics are described below.

Notes

1. Jean-Baptiste Bélanger (1789–1874) was a French professor at the Ecole Nationale Supérieure des Ponts et Chaussées (Paris). In his book (Bélanger, 1828), he first presented the basics of hydraulic jump calculations, backwater calculations and discharge characteristics of weirs.
2. C_D is a dimensionless coefficient. Typically $C_D = 1$ for a broad-crested weir. When $C_D > 1$, the discharge capacity of a weir is greater than that of a broad-crested weir for identical upstream head above crest $(H_1 - \Delta z)$.

19.2.2 Broad-crested weir

A broad-crested weir is a flat-crested structure with a crest length large compared to the flow thickness (Fig. 19.4). The ratio of crest length to upstream head over crest must be typically greater than 1.5–3 (e.g. Chow, 1973; Henderson, 1966):

$$\frac{L_{\text{crest}}}{H_1 - \Delta z} > 1.5\text{–}3 \qquad (19.3)$$

When the crest is 'broad' enough for the flow streamlines to be parallel to the crest, the pressure distribution above the crest is hydrostatic and the critical flow depth is observed on the weir crest. Broad-crested weirs are sometimes used as critical depth meters (i.e. to measure stream discharges). The hydraulic characteristics of broad-crested weirs were studied during the 19th and 20th centuries. Hager and Schwalt (1994) recently presented an authoritative work on the topic.

The discharge above the weir equals:

$$q = \frac{2}{3}\sqrt{\frac{2}{3}g(H_1 - \Delta z)^3} \qquad \text{Ideal fluid flow calculations} \qquad (19.1a)$$

where H_1 is the upstream total head and Δz is the weir height above channel bed (Fig. 19.4). Equation (19.1a) may be rewritten conveniently as:

$$q = 1.704(H_1 - \Delta z)^{3/2} \qquad \text{Ideal fluid flow calculations} \qquad (19.1b)$$

Notes

1. In a horizontal rectangular channel and assuming hydrostatic pressure distribution, the critical flow depth equals:

$$d_c = \frac{2}{3}E \qquad \text{Horizontal rectangular channel}$$

where E is the specific energy. The critical depth and discharge per unit width are related by:

$$d_c = \sqrt[3]{\frac{q^2}{g}} \qquad \text{Rectangular channel}$$

$$q = \sqrt{g d_c^{\,3}} \qquad \text{Rectangular channel}$$

2. At the crest of a broad-crested weir, the continuity and Bernoulli equations yield:

$$H_1 - \Delta z = \frac{3}{2} d_c$$

Note that equation (19.1) derives from the continuity and Bernoulli equations, hence from the above equation.

Discussion

(A) *Undular weir flow*

For low discharges (i.e. $(d_1 - \Delta z)/\Delta z \ll 1$), several researchers observed free-surface undulations above the crest of broad-crested weir (Fig. 19.4b). Model studies suggest that undular weir flow occurs for:

$$\frac{q}{\sqrt{g d_{\min}^3}} < 1.5 \qquad \text{Undular weir flow}$$

where $d_{\min}$ is the minimum flow depth upstream of the first wave crest (Fig. 19.4). Another criterion is:

$$\frac{H_1 - \Delta z}{L_{\text{crest}}} < 0.1 \qquad \text{Undular weir flow}$$

where L_{crest} is the crest length in the flow direction. The second equation is a practical criterion based on the ratio head on crest to weir length.

In practice, design engineers should avoid flow conditions leading to undular weir flow. In the presence of free-surface undulations above the crest, the weir cannot be used as a discharge meter, and waves may propagate in the downstream channel.

(B) *Discharge coefficients*

Experimental measurements indicate that the discharge versus total head relationship departs slightly from equation (19.1) depending upon the weir geometry and flow conditions. Equation (19.1) is usually rearranged as:

$$q = C_D \frac{2}{3} \sqrt{\frac{2}{3} g (H_1 - \Delta z)^3}$$

where the discharge coefficient C_D is a function of the weir height, crest length, crest width, upstream corner shape and upstream total head (Table 19.1).

19.2.3 Sharp-crested weir

A sharp-crested weir is characterized by a thin sharp-edged crest (Fig. 19.5). In the absence of sidewall contraction, the flow is basically two dimensional and the flow field can be solved by analytical and graphical methods: i.e. ideal fluid flow theory (e.g. Vallentine, 1969; p. 79).

For an aerated nappe, the discharge per unit width is usually expressed as:

$$q = \frac{2}{3} C \sqrt{2g(d_1 - \Delta z)^3} \tag{19.4}$$

where d_1 is the upstream water depth, Δz is the crest height above channel bed (Fig. 19.5) and C is a dimensionless discharge coefficient. Numerous correlations were proposed for C

Table 19.1 Discharge coefficient for broad-crested weirs

Reference (1)	Discharge coefficient C_D (2)	Range (3)	Remarks (4)
Sharp-corner weir Hager and Schwalt (1994)	$0.85\frac{9}{7}\left(1-\frac{2/9}{1+\left((H_1-\Delta z)/L_{crest}\right)^4}\right)$	$0.1<\frac{H_1-\Delta z}{L_{crest}}<1.5$	Deduced from laboratory experiments
Rounded-corner weir Bos (1976)	$\left(1-0.01\frac{L_{crest}-r}{W}\right)\left(1-0.01\frac{L_{crest}-r}{d_1-\Delta z}\right)$	$\frac{d_1-Dz}{L_{crest}}>0.05$ $d_1-\Delta z>0.06\,m$ $\frac{H_1-\Delta z}{\Delta z}<1.5$ $\Delta z>0.15\,m$	Based upon laboratory and field tests
Ackers *et al.* (1978)	0.95[a]	$0.15<\frac{H_1-\Delta z}{\Delta z}<0.6$	

Notes: [a]Re-analysis of experimental data presented by Ackers *et al.* (1978); *r*: curvature radius of upstream corner.

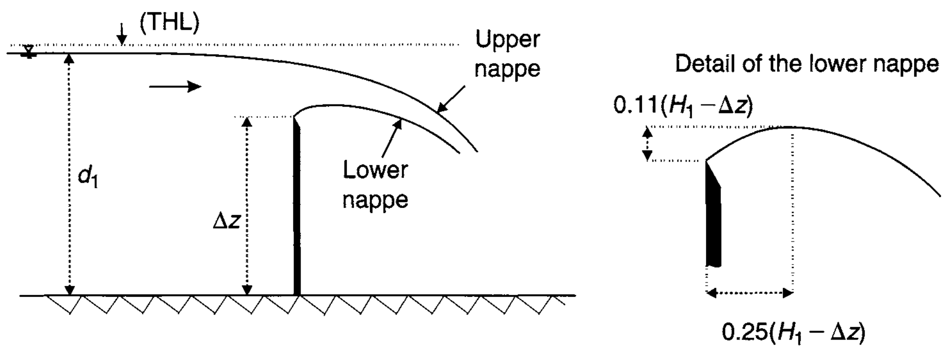


Fig. 19.5 Sharp-crested weir.

(Tables 19.2 and 19.3). In practice, the following expression is recommended:

$$C=0.611+0.08\frac{d_1-\Delta z}{\Delta z} \tag{19.5}$$

Notes

1. Sharp-crested weirs are very accurate discharge meters. They are commonly used for small flow rates.
2. For a vertical sharp-crested weir, the lower nappe is deflected upwards immediately downstream of the sharp edge. The maximum elevation of the lower nappe location occurs at about $0.11(H_1-\Delta z)$ above the crest level in the vertical direction and at about $0.25(H_1-\Delta z)$ from the crest in the horizontal direction (e.g. Miller, 1994).

Table 19.2 Discharge coefficient for sharp-crested weirs (full-width weir in rectangular channel)

Reference (1)	Discharge coefficient C (2)	Range (3)	Remarks (4)
von Mises (1917)	$\frac{\pi}{\pi+2}$	$\frac{d_1-\Delta z}{\Delta z}$ very large	Ideal fluid flow calculations of orifice flow
Henderson (1966)	$0.611+0.08\frac{d_1-\Delta z}{\Delta z}$	$0 \leqslant \frac{d_1-\Delta z}{\Delta z} < 5$	Experimental work by Rehbock (1929)
	1.135	$\frac{d_1-\Delta z}{\Delta z} = 10$	
	$1.06\left(1+\frac{\Delta z}{d_1-\Delta z}\right)^{3/2}$	$20 < \frac{d_1-\Delta z}{\Delta z}$	
Bos (1976)	$0.602+0.075\frac{d_1-\Delta z}{\Delta z}$	$d_1-\Delta z > 0.03\,\mathrm{m}$ $\frac{d_1-\Delta z}{\Delta z} < 2$ $\Delta z > 0.40\,\mathrm{m}$	Based on experiments performed at Georgia Institute of Technology
Chanson (1999)	1.0607	$\Delta z = 0$	Ideal flow at overfall

Table 19.3 Discharge correlations for sharp-crested weirs (full width in rectangular channel)

Reference (1)	Discharge per unit width q ($\mathrm{m^2/s}$) (2)	Comments (3)
Ackers *et al.* (1978)	$0.564\left(1+0.150\frac{d_1-\Delta z}{\Delta z}\right)\sqrt{g}(d_1-\Delta z+0.0001)^{3/2}$	Range of applications $d_1-\Delta z > 0.02\,\mathrm{m}$ $\Delta z > 0.15$ $(d_1-\Delta z)/\Delta z < 2.2$
Herschy (1995)	$1.85(d_1-\Delta z)^{3/2}$	Approximate correlation ($\pm 3\%$): $(d_1-\Delta z)/\Delta z < 0.5$

3. At very low flow rates, $(d_1 - \Delta z)/\Delta z$ is very small and equation (19.4) tends to:

$$q \sim 1.803(d_1 - \Delta z)^{3/2} \qquad \text{Very small discharge}$$

4. Nappe aeration is extremely important. If the nappe is not properly ventilated, the discharge characteristics of the weir are substantially affected, and the weir might not operate safely. Sometimes, the crest can be contracted at the sidewalls to facilitate nappe ventilation (e.g. Henderson, 1966; pp. 177–178).

19.2.4 Ogee-crest weir

The basic shape of an ogee crest is that of the lower nappe trajectory of a sharp-crested weir flow for the design flow conditions (discharge Q_{des} and upstream head H_{des}) (Figs 19.2a and 19.5–19.8).

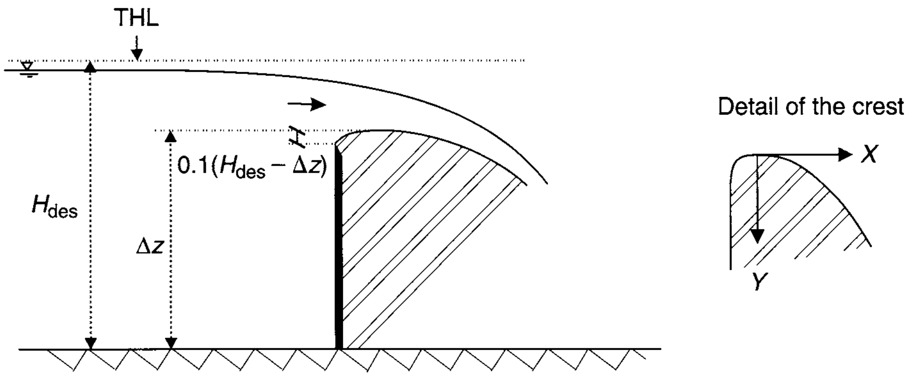


Fig. 19.6 Sketch of a nappe-shaped ogee crest.

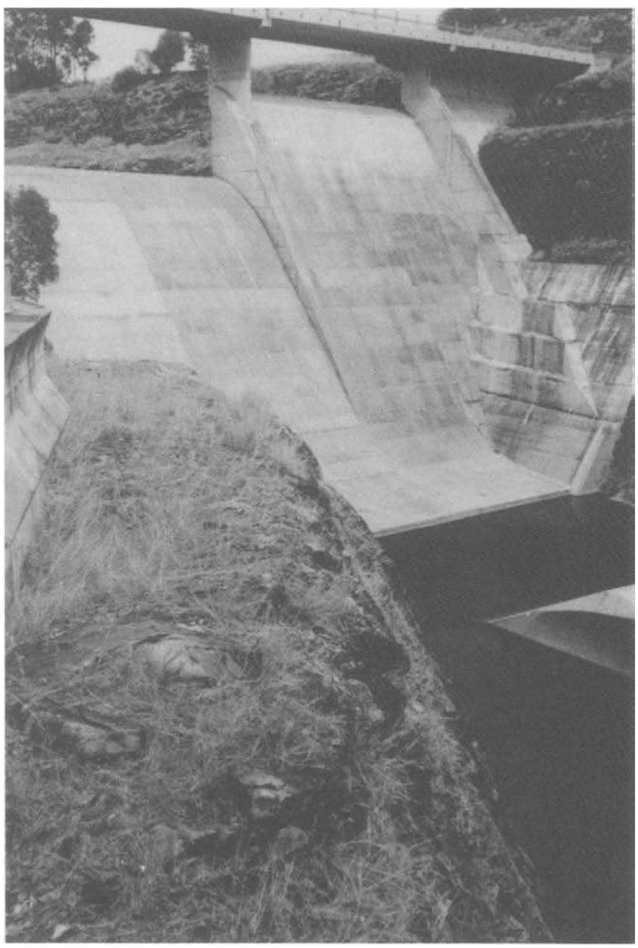
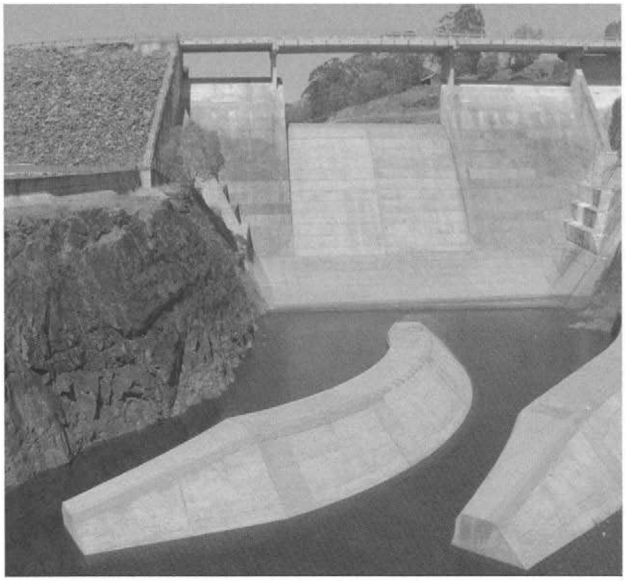
(a)


Fig. 19.7 Spillway chute of Hinze dam (Gold Coast QLD, 1976). Dam height: 44 m and design spillway capacity: 1700 m^3/s. (a) Ogee-crest chute in September 1997 (view from downstream, on right bank).



(b)

(c)

Fig. 19.7 (b) Concrete veins at downstream end of chute, directing the supercritical chute flow into the energy dissipator (looking upstream on September 2002). (c) Spillway in operation in the early 1990s for $Q \sim 300\text{–}400\,\mathrm{m^3/s}$ (courtesy of Gold Coast City Council). View from the left bank, with turning veins in foreground (underwater). Flow from right to left.

Nappe-shaped overflow weir

The characteristics of a nappe-shaped overflow can be deduced from the equivalent 'sharp-crested weir' design. For the design head H_{des}, equation (19.4) leads to:

$$q_{des} = \frac{2}{3} C \sqrt{g\left(\frac{H_{des} - \Delta z}{0.89}\right)^3} \qquad (19.6)$$

where Δz is the crest elevation (Fig. 19.6). For a high weir (i.e. $(H_{des} - \Delta z)/\Delta z \ll 1$), the discharge above the crest could be deduced from equations (19.5) and (19.6):

$$q_{des} = 2.148(H_{des} - \Delta z)^{3/2} \qquad \text{Ideal flow conditions} \qquad (19.7)$$

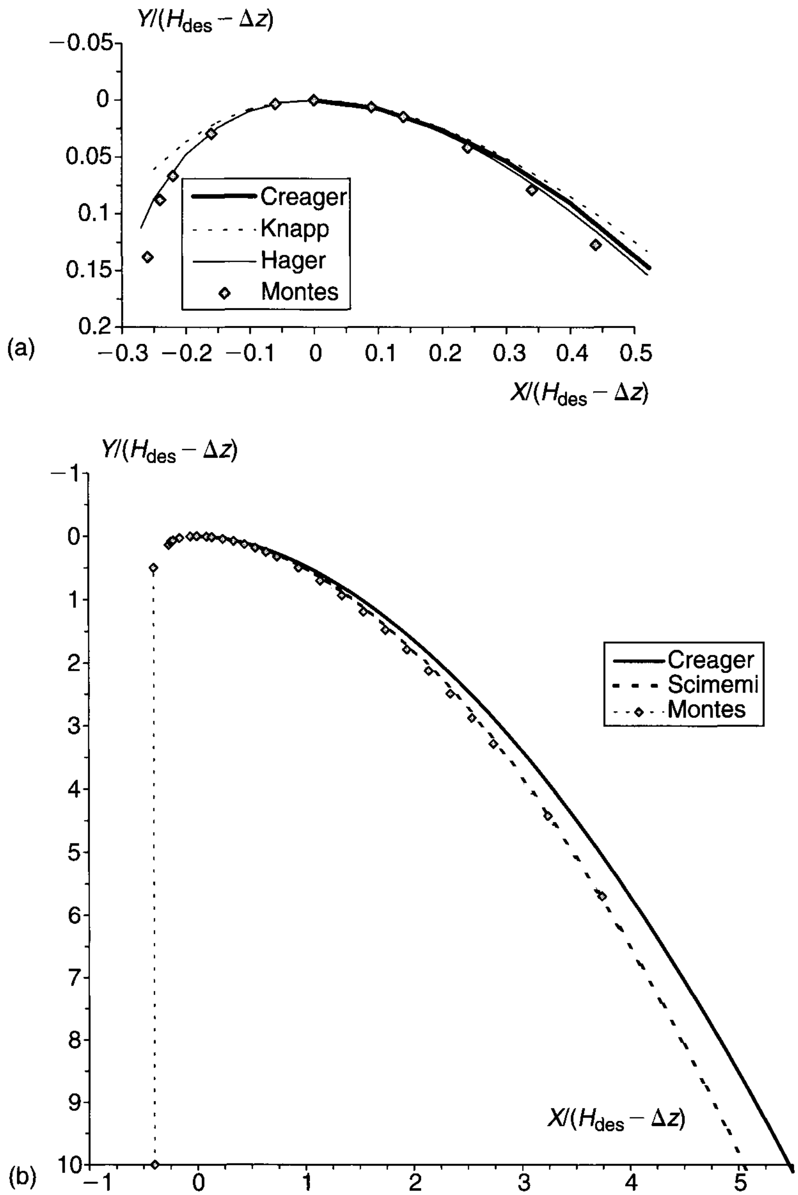


Fig.19.8 Profiles of ogee crest: (a) crest details and (b) Creager and Scimemi profiles.

Discharge characteristics of an ogee crest

In practice, the discharge–head relationship differs from ideal flow conditions (equation (19.7)). For the design flow conditions, the flow rate per unit width is often presented as:

$$q_{\mathrm{des}} = C_{\mathrm{des}}(H_{\mathrm{des}} - \Delta z)^{3/2} \qquad \{\text{Design flow conditions: } q = q_{\mathrm{des}}\} \qquad (19.8)$$

where the design discharge coefficient C_{des} is primarily a function of the ogee-crest shape. It can also be deduced from model tests.

Table 19.4 Discharge coefficient of ogee crest (vertical-faced ogee crest)

$\Delta z/(H_{des} - \Delta z)$ (1)	C_{des} ($m^{1/2}/s$) (2)	$(C_D)_{des}$ (3)	Comments (4)
0	1.7	1.0	Broad-crested weir
0.04	1.77	1.04	
0.14	1.92	1.13	
0.36	2.07	1.21	
0.50	2.10	1.23	
1.0	2.14	1.257	
1.5	2.16	1.268	
3.0	2.18	1.28	
>3.0	2.19	1.28	Large weir height

Reference: US Bureau of Reclamation (1987).
Notes: $C_{des} = q_{des}/(H_{des} - \Delta z)^{3/2}$; $(C_D)_{des} = q_{des}/(\sqrt{g}\,(2/3)^{3/2}(H_{des} - \Delta z)^{3/2}$.

For vertical-shaped ogee crest, typical values of the discharge coefficient are reported in Table 19.4 and Fig. 19.9a. In Table 19.4, the broad-crested weir case corresponds to $\Delta z/(H_{des} - \Delta z) = 0$ and it is found $(C_D)_{des} = 1$ (equation (19.1)). For a high weir (i.e. $\Delta z/(H_{des} - \Delta z) > 3$), the discharge coefficient C_{des} tends to $2.19\,m^{1/2}/s$. Such a value differs slightly from equation (19.7) but it is more reliable because it is based upon experimental results.

For a given crest profile, the overflow conditions may differ from the design flow conditions. The discharge versus upstream head relationship then becomes:

$$q = C(H_1 - \Delta z)^{3/2} \qquad \{\text{Non-design flow conditions: } q \neq q_{des}\} \tag{19.9}$$

in which the discharge coefficient C differs from the design discharge coefficient C_{des}.

Generally, the relative discharge coefficient C/C_{des} is a function of the relative total head $(H_1 - \Delta z)/(H_{des} - \Delta z)$ and of the ogee-crest shape (e.g. Fig. 19.9b).

Discussion

For $H_1 = H_{des}$ the pressure on the crest invert is atmospheric (because the shape of the invert is based on the lower nappe trajectory of the sharp-crested weir overflow).

When the upstream head H_1 is larger than the design head H_{des}, the pressures on the crest are less than atmospheric and the discharge coefficient C (equation (19.9)) is larger than the design discharge coefficient C_{des} (typically $2.19\,m^{1/2}/s$). For $H_1 < H_{des}$, the pressures on the crest are larger than atmospheric and the discharge coefficient is smaller. At the limit, the discharge coefficient tends to the value of $1.704\,m^{1/2}/s$ corresponding to the broad-crested weir case (equation (19.1b)) (Table 19.5).

Standard crest shapes

With nappe-shaped overflow weirs, the pressure on the crest invert should be atmospheric at design head. In practice, small deviations occur because of bottom friction and developing boundary layer. Design engineers must select the shape of the ogee crest such that sub-atmospheric pressures are avoided on the crest invert: i.e. to prevent separation and cavitation-related problems. Several ogee-crest profiles were developed (Table 19.6).

The most usual profiles are the WES profile and the Creager profile. The Creager design is a mathematical extension of the original data of Bazin in 1886–1888 (Creager, 1917). The WES-standard ogee shape is based upon detailed observations of the lower nappe of sharp-crested weir flows (Scimemi, 1930) (Figs 19.5 and 19.8).

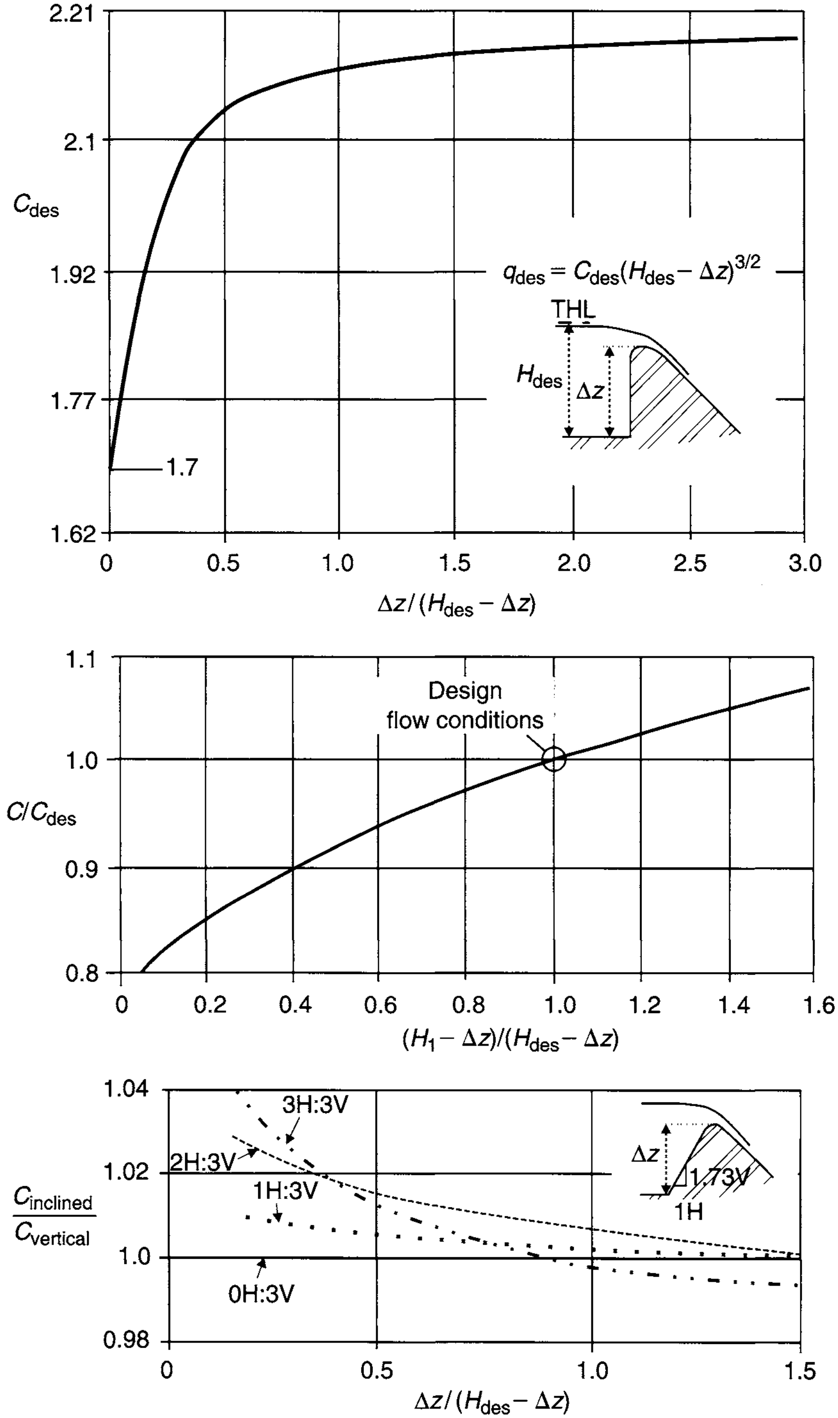


Fig. 19.9 Discharge coefficient of a USBR-profile ogee crest (data: US Bureau of Reclamation, 1987).

Table 19.5 Pressures on an ogee crest invert for design and non-design flow conditions

Upstream head (1)	Pressure on crest (2)	Discharge coefficient (3)
$H_1 = H_{des}$	Quasi-atmospheric	$C = C_{des}$
$H_1 > H_{des}$	Less than atmospheric	$C > C_{des}$
$H_1 < H_{des}$	Larger than atmospheric	$C < C_{des}$
$H1 \ll H_{des}$	Larger than atmospheric	$C \approx 1.704\,\mathrm{m^{1/2}/s}$

Table 19.6 Examples of spillway profiles (vertical-faced ogee crest)

Profile (1)	Equations (2)	Comments (3)
Creager (1917) profile	$Y = 0.47\dfrac{X^{1.80}}{(H_{des} - \Delta z)^{0.80}}$	For $X \geqslant 0$; derived from Bazin's (1888–1898) experiments
Scimemi (1930) profile	$Y = 0.50\dfrac{X^{1.85}}{(H_{des} - \Delta z)^{0.85}}$	For $X \geqslant 0$; also called WES profile
Knapp (1960)	$\dfrac{Y}{H_{des} - \Delta z} = \dfrac{X}{H_{des} - \Delta z} - \ln\left(1 + \dfrac{X}{0.689(H_{des} - \Delta z)}\right)$	Continuous spillway profile for crest region only (as given by Montes, 1992a)
Hager (1991)	$\dfrac{Y}{H_{des} - \Delta z} = 0.1360 + 0.482625\left(\dfrac{X}{H_{des} - \Delta z} + 0.2818\right) \times \ln\left(1.3055\left(\dfrac{X}{H_{des}\Delta z} + 0.2818\right)\right)$	Continuous spillway profile with continuous curvature radius: $-0.498 < \dfrac{X}{H_{des} - \Delta z} < 0.484$
Montes (1992a)	$\dfrac{R_1}{H_{des} - \Delta z} = 0.05 + 1.47\dfrac{s}{H_{des} - \Delta z}$	Continuous spillway profile with continuous curvature radius R Lower asymptote: i.e. for small values of $s/(H_{des} - \Delta z)$
	$\dfrac{R}{H_{des} - \Delta z} = \dfrac{R_1}{H_{des} - \Delta z}\left(1 + \left(\dfrac{R_u}{R_1}\right)^{2.625}\right)^{1/2.625}$	Smooth variation between the asymptotes
	$\dfrac{R_u}{H_{des} - \Delta z} = 1.68\left(\dfrac{s}{H_{des} - \Delta z}\right)^{1.625}$	Upper asymptote: i.e. for large values of $s/(H_{des} - \Delta z)$

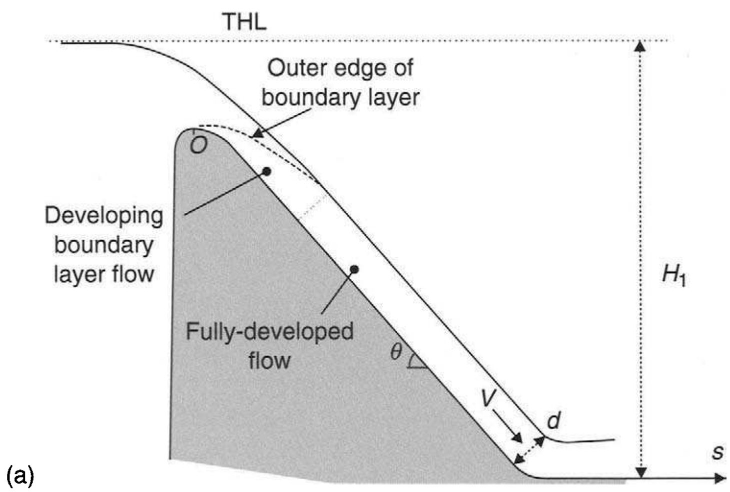
Notes: X, Y: horizontal and vertical co-ordinates with dam crest as origin, Y measured positive downwards (Fig. 19.8); R: radius of curvature of the crest; s: curvilinear co-ordinate along the crest shape.

Montes (1992a) stressed out that the ogee-crest profile must be continuous and smooth, and sudden variation of the crest curvature must be avoided to prevent unwanted aeration or cavitation. Ideally, the crest profile should start tangentially to the upstream apron, with a smooth and continuous variation of the radius of curvature.

19.3 CHUTE DESIGN

19.3.1 Presentation

Once the water flows past the crest, the fluid is accelerated by gravity along the chute. At the upstream end of the chute, a turbulent boundary layer is generated by bottom friction and develops in the flow direction. When the outer edge of the boundary layer reaches the free surface, the flow becomes fully developed (Fig. 19.10).



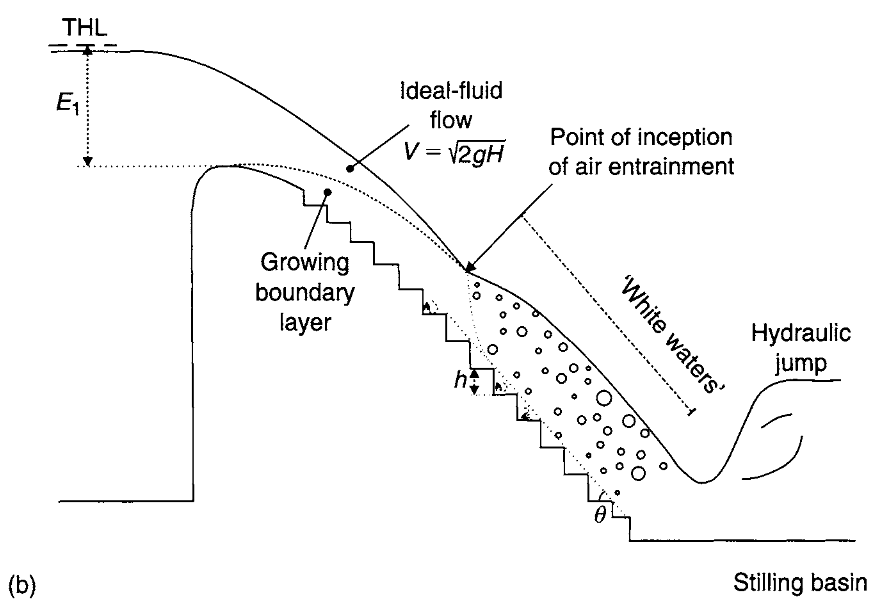


Fig. 19.10 Sketch of a steep chute: (a) smooth chute and (b) stepped chute.

In the developing flow region, the boundary layer thickness δ increases with distance along the chute. Empirical correlations may be used to estimate the boundary layer growth:

$$\frac{\delta}{s} = 0.0212(\sin\theta)^{0.11}\left(\frac{s}{k_s}\right)^{-0.10} \quad \text{Smooth concrete chute } (\theta > 30°) \qquad (19.10)$$

$$\frac{\delta}{s} = 0.06106(\sin\theta)^{0.133}\left(\frac{s}{h\cos\theta}\right)^{-0.17} \quad \text{Stepped chute (skimming flow)} \qquad (19.11)$$

where s is the distance measured from the crest origin, k_s is the equivalent roughness height, θ is the chute slope and h is the step height. Equations (19.10) and (19.11) are semi-empirical formulations which fit well model and prototype data (Wood *et al.* 1983; Chanson, 1995b, 2001). They apply to steep concrete chutes (i.e. $\theta > 30°$).

In the fully developed flow region, the flow is gradually varied until it reaches equilibrium (i.e. normal flow conditions). The gradually varied flow properties can be deduced by integration of the backwater equation. The normal flow conditions are deduced from the momentum principle (Appendix A4.1).

Note

On steep chutes, free-surface aeration[2] may take place downstream of the intersection of the outer edge of the developing boundary layer with the free surface. Air entrainment can be clearly identified by the 'white water' appearance of the free-surface flow. Wood (1991) and Chanson (1997) presented comprehensive studies of free-surface aeration on smooth chutes while Chanson (1995b, 2001) reviewed the effects of free-surface aeration on stepped channels.

The effects of free-surface aeration include flow bulking, some drag reduction effect and air–water gas transfer. The topic is still under active research (e.g. Chanson, 1997).

19.3.2 Application

For steep chutes (typically $\theta \sim 45–55°$), both the flow acceleration and boundary layer development affect the flow properties significantly. The complete flow calculations can be tedious and most backwater calculations are not suitable[3]. Complete calculations of developing flow and uniform equilibrium flow (see Appendix A4.1) may be combined to provide a general trend which may be used for a preliminary design (Fig. 19.11). Ideally, the maximum velocity at the downstream chute end is:

$$V_{\max} = \sqrt{2g(H_1 - d\cos\theta)} \quad \text{Ideal fluid flow} \qquad (19.12)$$

where H_1 is the upstream total head and d is the downstream flow depth: i.e. $d = q/V_{\max}$ (Fig. 19.10). In practice the downstream flow velocity V is smaller than the theoretical velocity $V_{\max}$ because of friction losses.

[2] Air entrainment in open channels is also called free-surface aeration, self-aeration, insufflation or white waters.

[3] Because backwater calculations are valid only for fully developed flows. Furthermore, most softwares assume hydrostatic pressure distributions and neglect the effects of free-surface aeration.

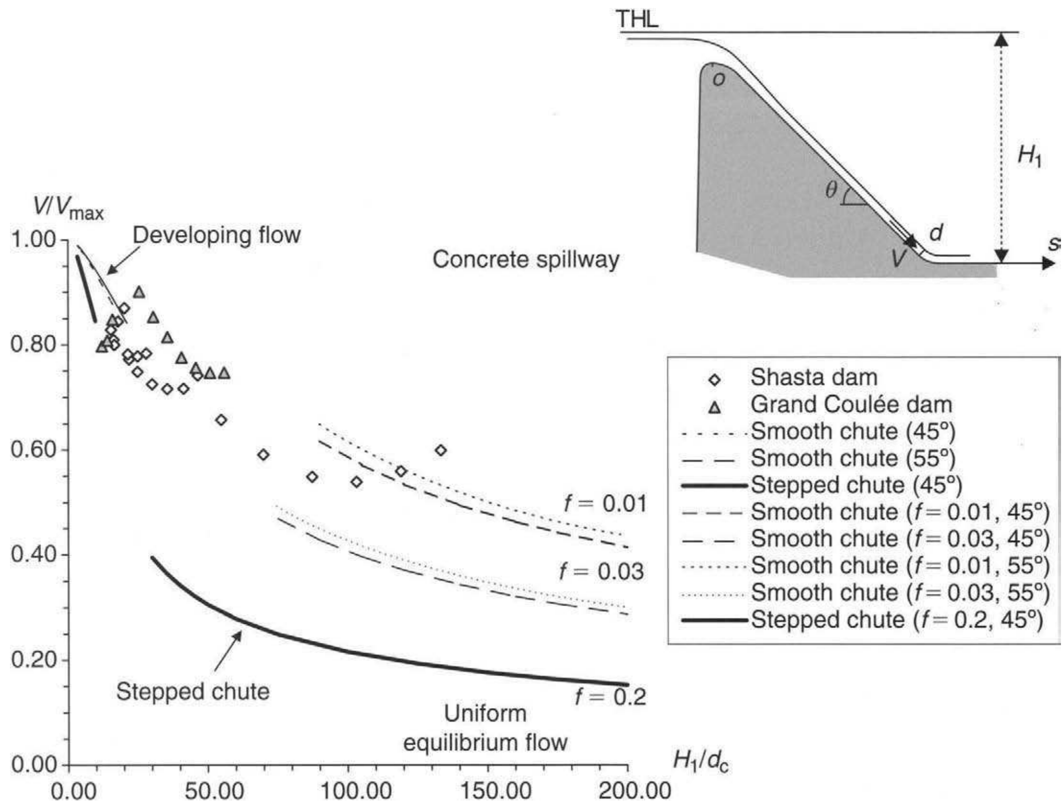


Fig. 19.11 Flow velocity at the downstream end of a steep chute.

In Fig. 19.11, the mean flow velocity at the end of the chute is plotted as V/V_{max} versus H_1/d_c where V_{max} is the theoretical velocity (equation (19.12)), H_1 is the upstream total head (above spillway toe) and d_c is the critical depth. Both developing flow calculations (equations (19.10) and (19.11)) and uniform equilibrium flow calculations are shown. Fitting curves are plotted to connect these lines (Fig. 19.11).

The semi-empirical curves (Fig. 19.11) are compared with experimental results obtained on prototype spillways (Grand Coulée dam and Shasta dam). The curves are valid for smooth and stepped spillways (concrete chutes), with slopes ranging from 45° to 55° (i.e. 1V:1H to 0.7V:1H).

19.3.3 Discussion

It is worth comparing the performances of stepped and smooth chutes. The larger mean bottom shear stress, observed with stepped chute flows, implies larger hydrodynamic loads on the steps than on a smooth invert. Stepped chutes required reinforced stepped profile compared to a smooth chute for identical inflow conditions. On the other hand, Fig. 19.11 also shows that larger energy dissipation takes place along a stepped spillway compared with a smooth chute. Hence, the size of the downstream stilling basin can be reduced with a stepped chute.

Note

Stepped chute flows are subjected to strong free-surface aeration (Chanson, 1995b, 2001). As a result, flow bulking and gas transfer are enhanced compared to a smooth channel.

Discussion

The energy dissipation characteristics of stepped channels were well known to ancient engineers. During the Renaissance period, Leonardo da Vinci realized that the flow, '*the more rapid it is, the more it wears away its channel*'; if a waterfall '*is made in deep and wide steps, after the manner of stairs, the waters (...) can no longer descend with a blow of too great a force*'. He illustrated his conclusion with a staircase waterfall 'down which the water falls so as not to wear away anything' (Richter, 1939).

During the 19th century, stepped (or staircase) weirs and channels were quite common: e.g. in USA, nearly one third of the masonry dams built during the 19th century were equipped with a stepped spillway. A well-known 19th century textbook stated: '*The byewash*[4] *will generally have to be made with a very steep mean gradient, and to avoid the excessive scour which could result if an uniform*[5] *channel were constructed, it is in most cases advisable to carry the byewash down by a series of steps, by which the velocity will be reduced*' (Humber, 1876: p. 133).

19.4 STILLING BASINS AND ENERGY DISSIPATORS

19.4.1 Presentation

Energy dissipators are designed to dissipate the excess in kinetic energy at the end of the chute before it re-enters the natural stream. Energy dissipation on dam spillways is achieved usually by (1) a standard stilling basin downstream of a steep spillway in which a hydraulic jump is created to dissipate a large amount of flow energy and to convert the flow from supercritical to subcritical conditions, (2) a high velocity water jet taking off from a flip bucket and impinging into a downstream plunge pool or (3) a plunging jet pool in which the spillway flow impinges and the kinetic energy is dissipated in turbulent recirculation (Fig. 19.12). The construction of steps on the spillway chute may also assist in energy dissipation.

The stilling basin is the common type of dissipators for weirs and small dams. Most energy is dissipated in a hydraulic jump assisted by appurtenances (e.g. step and baffle blocks) to increase the turbulence.

Notes

1. Other forms of energy dissipator include the drop structure and the impact-type stilling basin. The drop structure is detailed in another section because the flow pattern differs substantially from chute spillways. With impact-type dissipators, dissipation takes place by impact of the inflow on a vertical baffle (e.g. US Bureau of reclamation, 1987: p. 463).
2. On stepped chutes, the channel roughness (i.e. steps) contributes to the energy dissipation. In practice, a stilling basin is often added at the downstream end, but its size is smaller than that required for a smooth chute with identical flow conditions (see Section 19.3.2).

[4] Channel to carry waste waters: i.e. spillway.
[5] In the meaning of an uniform smooth channel bed (i.e. not stepped).

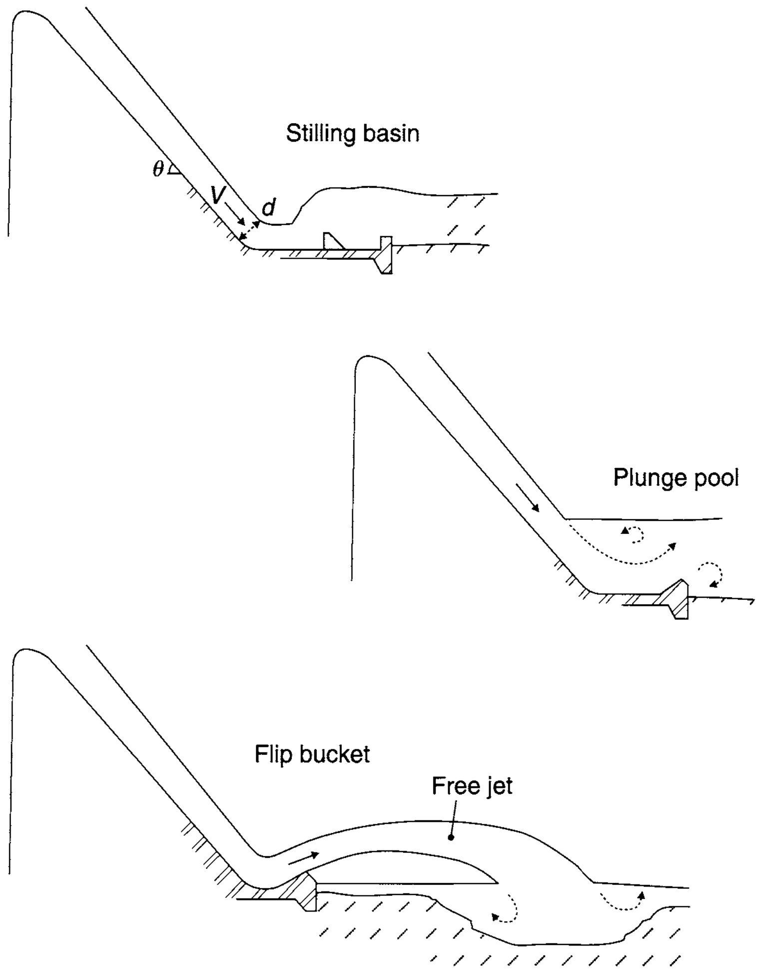


Fig. 19.12 Types of energy dissipators.

19.4.2 Energy dissipation at hydraulic jumps

Introduction

A hydraulic jump is the rapid transition from a supercritical to subcritical flow. It is an extremely turbulent process, characterized by large-scale turbulence, surface waves and spray, energy dissipation and air entrainment (Fig. 19.13). The large-scale turbulence region is usually called the 'roller'.

The downstream flow properties and energy loss in a hydraulic jump can be deduced from the momentum principle as a function of the upstream Froude number Fr and upstream flow depth d. For a horizontal flat rectangular channel, the downstream flow depth equals:

$$\frac{d_{\mathrm{conj}}}{d} = \frac{1}{2}\left(\sqrt{1 + 8Fr^2} - 1\right) \tag{19.13}$$

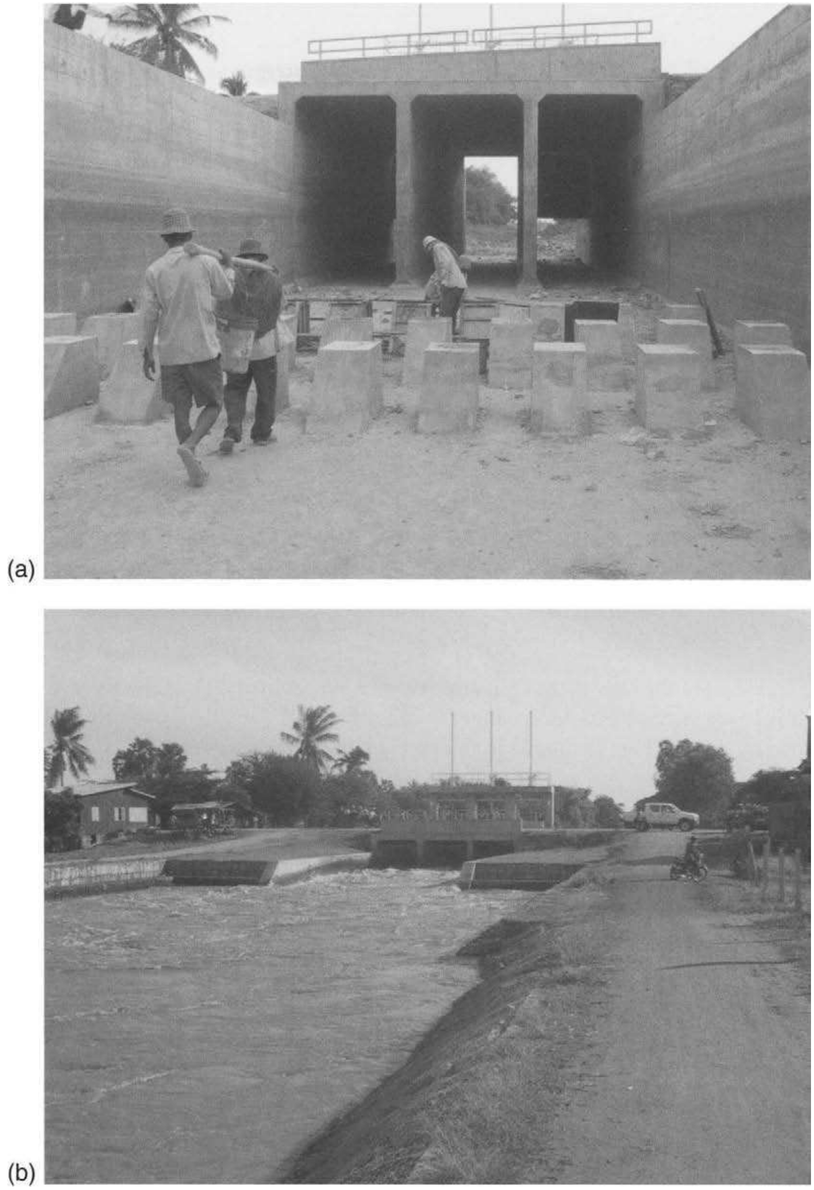


Fig. 19.13 Hydraulic jump stilling basin downstream of sluice of gates, Large Colmitage, Cambodia (courtesy of Peter Ward): (a) energy dissipation blocks in foreground and gates in background, looking upstream and (b) operation during the flood season, looking upstream.

where $Fr = V/\sqrt{gd}$. The energy equation gives the head loss:

$$\frac{\Delta H}{d} = \frac{\left(\sqrt{1 + 8Fr^2} - 3\right)^3}{16\left(\sqrt{1 + 8Fr^2} - 1\right)} \tag{19.14}$$

Notes

1. The upstream and downstream depth d and d_{conj} are referred to as conjugate or sequent depths.
2. The upstream Froude number must be greater than unity: $Fr > 1$.
3. Jean-Baptiste Bélanger (1789–1874) was the first to suggest the application of the momentum principle to the hydraulic jump flow (Bélanger, 1828). The momentum equation applied across a hydraulic jump is called the Bélanger equation. Equation (19.13) is sometimes called (improperly) the Bélanger equation.

Application

In a horizontal rectangular stilling basin with baffle blocks, the inflow conditions are $d = 0.95$ m and $V = 16.8$ m/s. The observed downstream flow depth is 6.1 m. The channel is 12.5 m wide. Calculate the total force exerted on the baffle blocks. In which direction does the force applied by the flow onto the blocks act?

Solution

First, we select a control volume for which the upstream and downstream cross-sections are located far enough from the roller for the velocity to be essentially horizontal and uniform. The application of the momentum equation to the control volume yields:

$$\rho q(V_{tw} - V) = \frac{1}{2}\rho g(d^2 - d_{tw}^2) - \frac{F_B}{B}$$

where d_{tw} and V_{tw} are the tailwater depth and velocity, respectively, F_B is the force from the blocks on the control volume and B is the channel width. The total force is $F_B = +605$ kN. That is, the force applied by the fluid on the blocks acts in the downstream direction.

Remark

Equation (19.13) is not applicable. It is valid only for flat horizontal rectangular channels. In the present application, the baffle blocks significantly modify the jump properties.

Types of hydraulic jump

Hydraulic jump flows may exhibit different flow patterns depending upon the upstream flow conditions. They are usually classified as functions of the upstream Froude number Fr (Table 19.7). In practice, it is recommended to design energy dissipators with a steady jump type.

Table 19.8 summarizes the basic flow properties of hydraulic jump in rectangular horizontal channels.

Notes

1. The classification of hydraulic jumps (Table 19.7) must be considered as *rough* guidelines. It applies to hydraulic jumps in rectangular horizontal channels.
2. Recent investigations (e.g. Chanson and Montes, 1995; Montes and Chanson, 1998) showed that undular hydraulic jumps (Fawer jumps) might take place for upstream Froude numbers up to 4 depending upon the inflow conditions. The topic is still actively studied.
3. A hydraulic jump is a very unsteady flow. Experimental measurements of bottom pressure fluctuations indicated that the mean pressure is quasi-hydrostatic below the jump but large pressure fluctuations are observed (e.g. Hager, 1992b).

Table 19.7 Classification of hydraulic jump in rectangular horizontal channels (Chow, 1973)

Fr (1)	Definition (2)	Remarks (3)
1	Critical flow	No hydraulic jump
1–1.7	Undular jump (Fawer jump)	Free-surface undulations developing downstream of jump over considerable distances; *negligible* energy losses
1.7–2.5	Weak jump	Low energy loss
2.5–4.5	Oscillating jump	Wavy free surface, production of large waves of irregular period, unstable oscillating jump, each irregular oscillation produces a large wave which may travel far downstream, damaging and eroding the banks; *to be avoided* if possible
4.5–9	Steady jump	45–70 % of energy dissipation, steady jump, insensitive to downstream conditions (i.e. tailwater depth); *best economical design*
>9	Strong jump	Rough jump, up to 85% of energy dissipation, *risk of channel bed erosion*; to be avoided

Table 19.8 Dimensionless characteristics of hydraulic jump in horizontal rectangular channels

Fr (1)	d_{conj}/d (2)	$\Delta H/H$ (3)	L_r/d (4)
3	3.77	0.26	11.8
3.5	4.47	0.33	15.7
4.5	5.88	0.44	23.4
5.5	7.29	0.53	30.9
6.5	8.71	0.59	38.2
7.5	10.12	0.64	45.3
9	12.24	0.70	55.5
12	16.48	0.77	73.9
15	20.72	0.82	89.6

Length of the roller

The roller length of the jump may be estimated as (Hager *et al.* 1990):

$$\frac{L_r}{d} = 160\tanh\left(\frac{Fr}{20}\right) - 12 \qquad 2 < Fr < 16 \tag{19.15}$$

where tanh is the hyperbolic tangent function and L_r is the length of the roller. Equation (19.15) is valid for rectangular horizontal channels ($d/B < 0.1$) and it may be used to predict the length of horizontal dissipation basins.

Practically, the length of a hydraulic jump stilling basin must be greater than the roller length for all flow conditions.

19.4.3 Stilling basins

Basically, the hydraulic design of a stilling basin must ensure a safe dissipation of the flow kinetic energy, to maximize the rate of energy dissipation and to minimize the size (and cost) of the structure. In practice, energy dissipation by hydraulic jump in a stilling basin is assisted with elements (e.g. baffle blocks and sill) placed on the stilling basin apron.

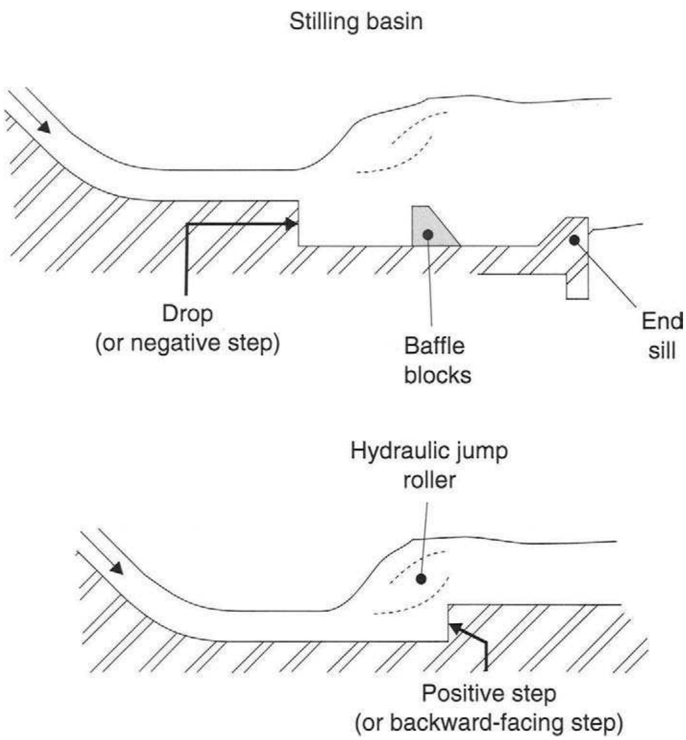


Fig. 19.14 Sketch of stilling basins.

Basic shapes

The basic features of stilling basins include drop, backward-facing step (or sill), baffle block(s) and sudden expansion (Fig. 19.14). Hager (1992b) reviewed the advantages of each type. A summary is given below.

Drops and backward-facing steps are simple elements used to stabilize the hydraulic jump. Drops (also called negative steps) are advised when the downstream tailwater level may vary significantly (Fig. 19.14a). Backward-facing steps (also called positive steps) are usually located near the toe of the jump (Fig. 19.14b).

Baffle blocks (or dentated sills) can be placed in one or several rows.[6] The blocks force the flow above them (as a sill) and in between them. Baffle blocks must be designed with standard shapes. They are not recommended when the inflow velocity is larger than 20–30 m/s because of the risks of cavitation damage.

Sudden expansion (in the stilling basin) is another technique to enhance turbulent energy dissipation and to reduce the basin length. Physical modelling is, however, strongly recommended.

Standard stilling basins

Several standardized designs of stilling basins were developed in the 1950s and 1960s (Table 19.9 and Fig. 19.15). These basins were tested in models and prototypes over a considerable range of operating flow conditions. The prototype performances are well known, and they can be selected and designed without further model studies.

[6] The single-row arrangement is comparatively more efficient than the multiple-rows geometry.

Table 19.9 Standard types of hydraulic jump energy dissipators

Name (1)	Application (2)	Flow conditions (3)	Tailwater depth d_{tw}[a] (4)	Remarks (5)
USBR Type II	Large structures	$Fr > 4.5$ $q < 46.5\,m^2/s$ $H_1 < 61\,m$ Basin length $\sim 4.4d_{conj}$	$1.05d_{conj}$	Two rows of blocks; the last row is combined with an inclined end sill (i.e. dentated sill); block height $= d$
USBR Type III	Small structures	$Fr > 4.5$ $q < 18.6\,m^2/s$ $V < 15$–$18.3\,m/s$ Basin length $\sim 2.8d_{conj}$	$1.0d_{conj}$	Two rows of blocks and an end sill; block height $= d$
USBR Type IV	For oscillating jumps	$2.5 < Fr < 4.5$ Basin length $\sim 6d_{conj}$	$1.1d_{conj}$	One row of blocks and an end sill; block height $= 2d$ Wave suppressors may be added at downstream end
SAF	Small structures	$1.7 < Fr < 17$ Basin length $= 4.5d_{conj}Fr^{-0.76}$	$1.0d_{conj}$	Two rows of baffle blocks and an end sill; block height $= d$
USACE		Basin length $> 4d_{conj}$	$1.0d_{conj}$	Two rows of baffle blocks and end sill

References: Chow (1973), Hager (1992b), Henderson (1966), US Bureau of Reclamation (1987).
Notes: [a]Recommended tailwater depth for optimum stilling basin operation; d: upstream flow depth (inflow depth); d_{conj}: conjugate flow depth (equation (19.13)); d_{tw}: tailwater depth; Fr: inflow Froude number.

In practice, the following types are highly recommended:

- the USBR Type II basin for large structures and $Fr > 4.5$;
- the USBR Type III basin and the SAF basin for small structures;
- the USBR Type IV basin for oscillating jump flow conditions (Fig. 19.16).

> **Note**
> The requirements of the USBR for its dissipators are more stringent than other organizations. As a result, the USBR Type III basin is sometimes too conservative and the SAF basin may be preferred for small structures.

19.4.4 Discussion

In practice, design engineers must ensure that a stilling basin can operate safely for a wide range of flow conditions. Damage (scouring, cavitation) to the basin and to the downstream natural bed may occur in several cases:

- the apron is too short and/or too shallow for an optimum jump location (i.e. on the apron),
- poor shapes of the blocks, sill and drop resulting in cavitation damage,
- flow conditions larger than design flow conditions,
- unusual overflow during construction periods,
- poor construction of the apron and blocks,
- seepage underneath the apron, inadequate drainage and uplift pressure built-up,
- wrong conception of the stilling basin.

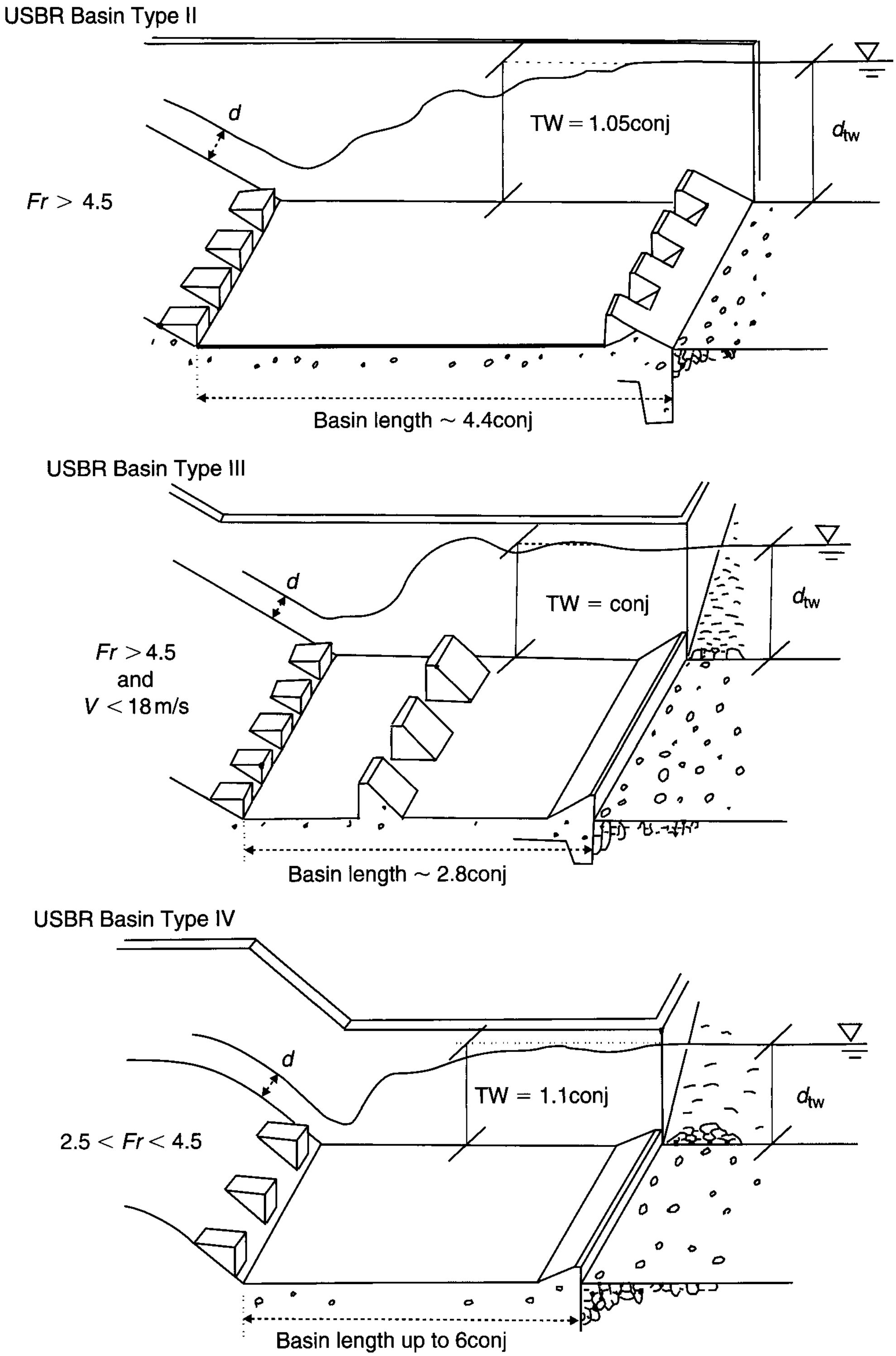


Fig. 19.15 Sketch of USBR Basins.

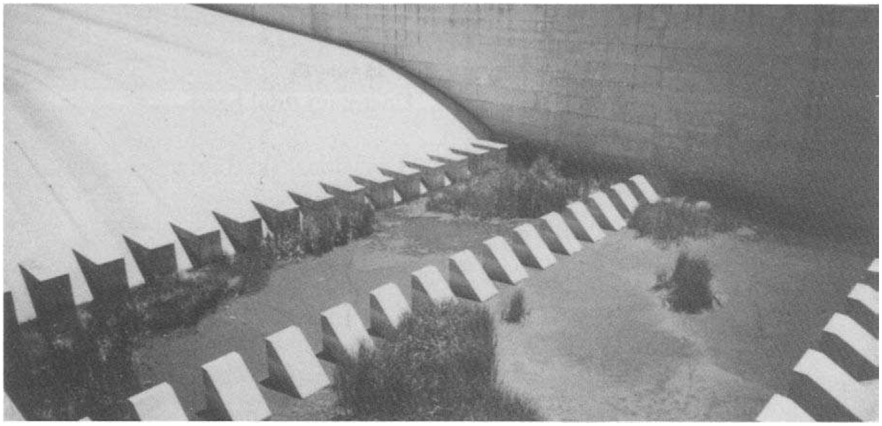


Fig. 19.16 Prototype stilling basin: USBR Basin Type IV at Bjelke-Petersen dam, Australia on 7 November 1997. Flow direction from left to right. Design conditions: 3660 m^3/s and dam height: 43 m.

19.5 DESIGN PROCEDURE

19.5.1 Introduction

The construction of a small dam across a stream will modify both the upstream and possibly downstream flow conditions. The dam crest elevation must be selected accurately to provide the required storage of water or upstream water level rise. Furthermore, the spillway and stilling basin must operate safely for a wide range of flow rates and tailwater flow conditions.

19.5.2 Dam spillway with hydraulic jump energy dissipator

Considering an overflow spillway with hydraulic jump energy dissipation at the toe, the basic steps in the design procedure are as follows:

Step 1. Select the crest elevation z_{crest} (bed topography and storage level).

Step 2. Choose the crest width B (site geometry, hydrology). The crest width may be smaller than the weir length (across the stream).

Step 3. Determine the design discharge Q_{des} from risk analysis and flood routing. Required informations include catchment area, average basin slope, degree of impermeability, vegetation cover, rainfall intensity, duration and inflow hydrograph. The peak spillway discharge is deduced from the combined analysis of storage capacity, and inflow and outflow hydrographs.

Step 4. Calculate the upstream head above spillway crest $(H_{\text{des}} - z_{\text{crest}})$ for the design flow rate Q_{des}:

$$\frac{Q_{\text{des}}}{B} = C_{\text{des}}(H_{\text{des}} - z_{\text{crest}})^{3/2}$$

Note that the discharge coefficient C_{des} varies with the head above crest $(H_{\text{des}} - z_{\text{crest}})$.

Step 5. Choose the chute toe elevation (i.e. apron level): $z_{\text{apron}} = z_{\text{crest}} - \Delta z$. The apron level may differ from the natural bed level (i.e. tailwater bed level).

Step 6. For the design flow conditions, calculate the flow properties d and V at the end of the chute toe (Fig. 19.10) using:

$$H_1 = H_{des} - z_{apron} \qquad \text{Upstream total head}$$

$$V_{max} = \sqrt{2g(H_1 - (q_{des}/V_{max})\cos\theta} \qquad \text{Ideal fluid flow velocity}$$

Step 7. Calculate the conjugate depth for the hydraulic jump:

$$\frac{d_{conj}}{d} = \frac{1}{2}\left(\sqrt{1 + 8Fr^2} - 1\right) \qquad \text{where } Fr = V/\sqrt{gd}$$

Step 8. Calculate the roller length L_r. The apron length must be greater than the jump length.

Step 9. Compare the *jump height rating level* (JHRL) and the natural downstream water level (i.e. the *tailwater rating level*, TWRL). If the jump height does not match the natural water level, the apron elevation, the crest width, the design discharge or the crest elevation must be altered (i.e. go back to Steps 5, 3, 2 or 1 respectively).

Practically, if the JHRL does not match the natural water level (TWRL), the hydraulic jump will not take place on the apron.

Figure 19.17 presents the basic definitions. Figure 19.18 illustrates two cases for which the stilling basin is designed to equal TWRL and JHRL at design flow conditions. In each sketch (Fig. 19.18), the real free-surface line is shown in solid line while the JHRL is shown in dashed line.

Figures 19.19–19.21 show prototype dissipators in operation for different flow rates, highlighting the tailwater effects on the hydraulic jump.

Notes

1. The JHRL is the free-surface elevation downstream of the stilling basin.
2. The TWRL is the natural free-surface elevation in the downstream flood plain. The downstream channel often flows as a subcritical flow, controlled by the downstream flow conditions (i.e. discharge and downstream flood plain geometry).

Discussion: calculations of the JHRL

(A) For a horizontal apron, the JHRL is deduced simply from the apron elevation and the conjugate depth:

$$\text{JHRL} = z_{apron} + d_{conj}$$

(B) For an apron with an end sill or end drop, the JHRL is calculated using the Bernoulli equation:

$$d_{conj} + z_{apron} + \frac{q^2}{2g(d_{conj})^2} = (\text{JHRL} - z_{tw}) + z_{tw} + \frac{q^2}{2g(\text{JHRL} - z_{tw})^2}$$

where z_{tw} is the downstream natural bed elevation. $(z_{tw} - z_{apron})$ is the drop/sill height.

Note that the above calculation assumes that the complete jump is located upstream of the drop/sill, and that no energy loss takes place at the sill/drop.

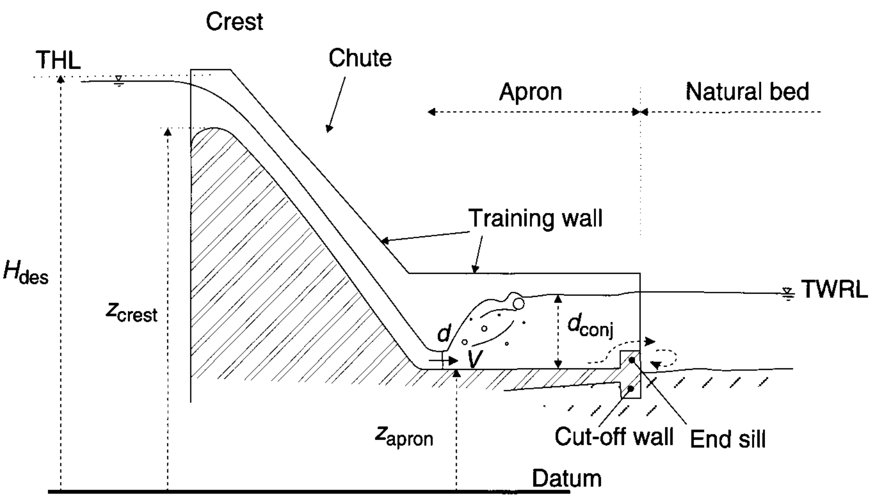


Fig. 19.17 Design of an overflow spillway with hydraulic jump stilling basin.

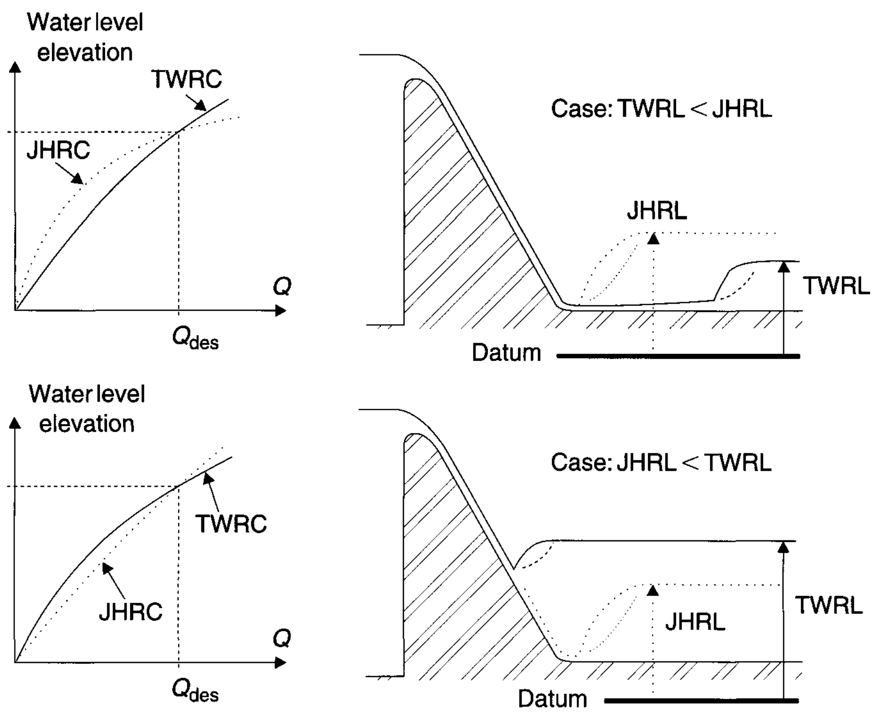
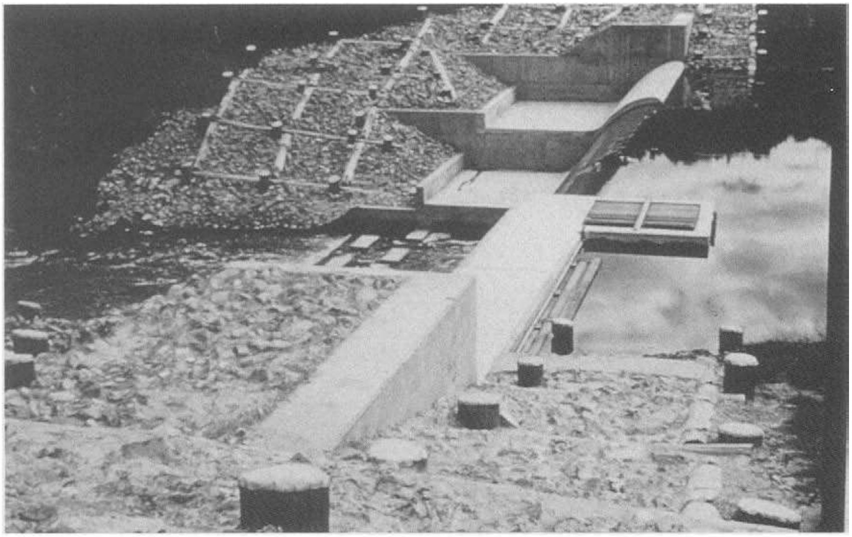


Fig. 19.18 Effect of tailwater level on the hydraulic jump location.

Matching the JHRC and the TWRC

During the design stages of hydraulic jump energy dissipator, engineers are required to compute the JHRL for all flow rates. The resulting curve, called the *jump height rating curve* (JHRC), must be compared with the variations of natural downstream water level with discharges (i.e. TWRC or *tailwater rating curve*).

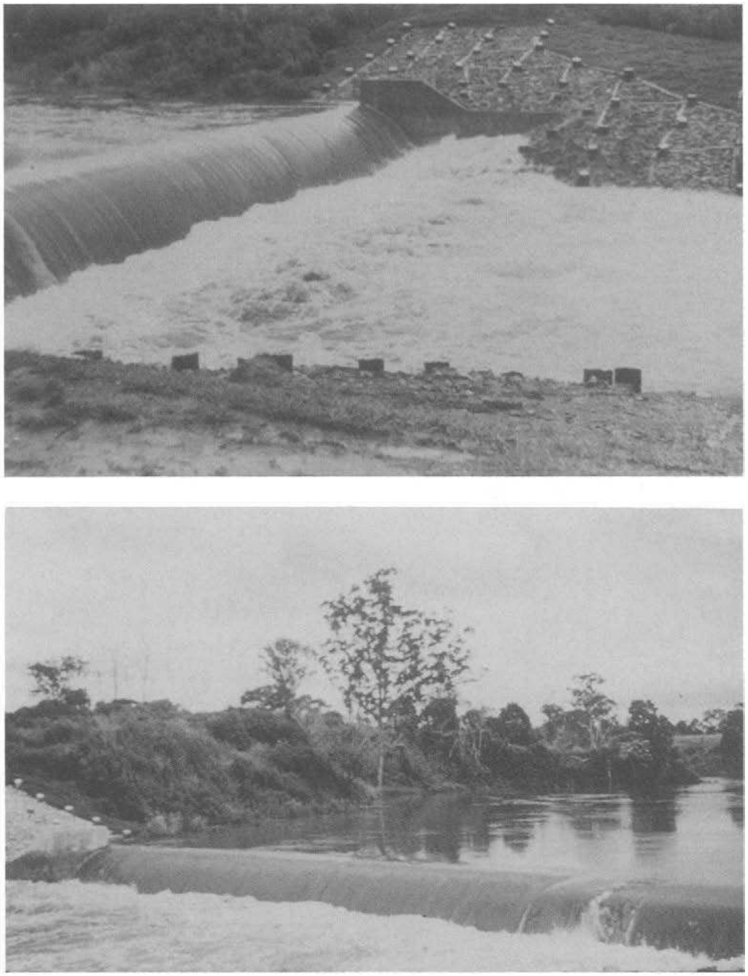
(a)



(b)

Fig. 19.19 Effect of tailwater levels on the Waraba Creek weir QLD, Australia (photographs from the collection of late Professor G.R. McKay, Australia). Smooth chute followed by a stilling basin with baffle blocks: (a) Operation at very low overflow Q_1. Flow from the right to the left. Note baffle blocks in the stilling basin. (b) Operation at low overflow Q_2 ($>Q_1$). Note the fully developed hydraulic jump.

First, let us remember that the flow downstream of the stilling basin jump is subcritical, and hence it is controlled by the downstream flow conditions: i.e. by the tailwater flow conditions. The location of the jump is determined by the upstream and downstream flow conditions. The upstream depth is the supercritical depth at the chute toe and the downstream depth is determined by the tailwater flow conditions. The upstream and downstream depths, called conjugate depths, must further satisfy the momentum equation: e.g. equation (19.13) for a horizontal apron in the absence of baffles.



(a)

(b)

Fig. 19.20 Effect of tailwater levels on the Waraba Creek weir QLD, Australia (photographs from the collection of late Professor G.R. McKay, Australia). (a) Operation at medium flow rate Q_3 ($Q_2 < Q_3 < Q_4$). Flow from the left to the right. Note the rising tailwater (compared to Fig. 19.19b). (b) Operation at large discharge Q_4 ($>Q_3$). View from downstream, flow from top right to bottom left. Note the plunge pool formed in the stilling basin caused by the rising tailwater level.

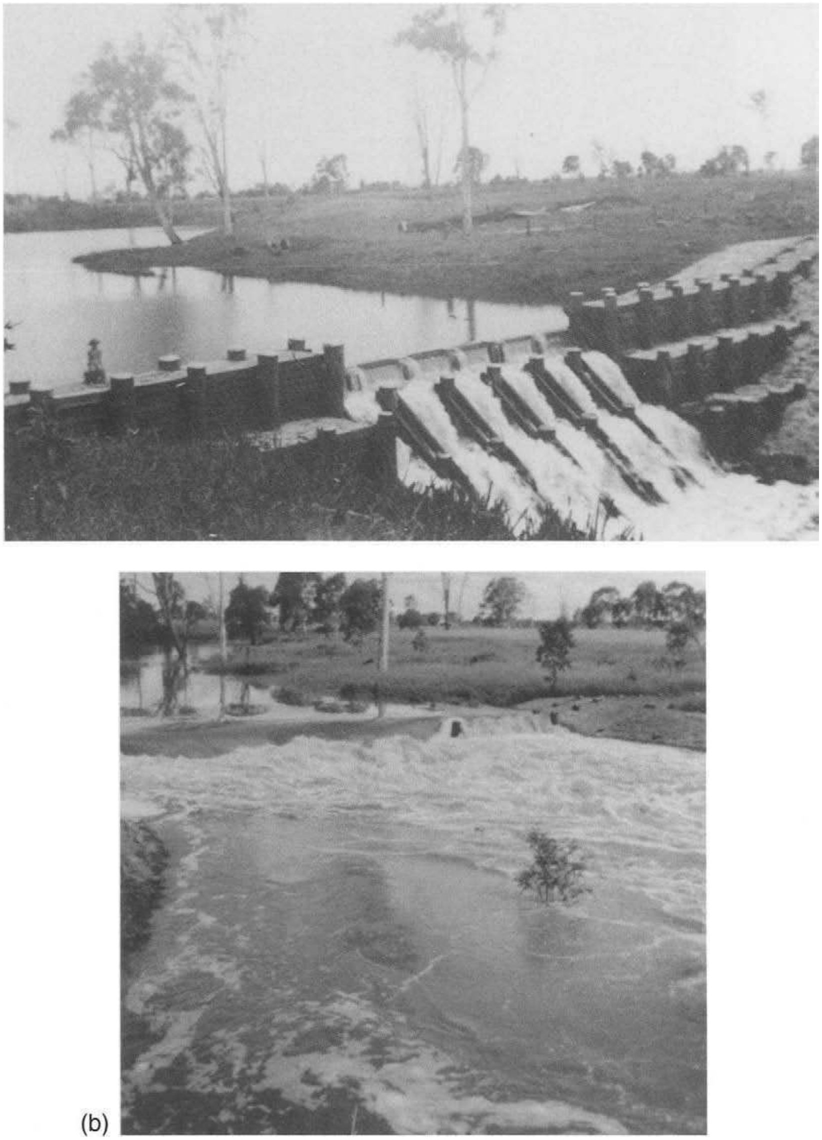
Discussion

In practice, the TWRC is set by the downstream flood plain characteristics for a range of flow rates. It may also be specified. Designers must select stilling basin dimensions such that the JHRC matches the TWRC: e.g. if the curves do not match, a sloping apron section may be used (e.g. Fig. 19.22). The height of the sloping apron, required to match the JHRC and the TWRC for all flows, is obtained by plotting the two curves, and measuring the greatest vertical separation of the curves. Note that if the maximum separation of the two curves occurs at very low flow rates, it is advised to set the height of the sloping apron to the separation of the curves at a particular discharge (e.g. $0.15Q_{\text{des}}$).

When the jump height level is higher than the tailwater level, the jump may not take place on the apron but downstream of the apron. The flow above the apron becomes a jet flow and insufficient energy dissipation takes place.

In such a case (i.e. JHRL > TWRL), the apron must be lowered or the tailwater level must be artificially raised. The apron level may be lowered (below the natural bed level) using an inclined upward apron design or a sill at the end of the apron (Fig. 19.22). The tailwater level may be raised also by providing a downstream weir at the end of the basin (and checking that the weir is never drowned).

When the jump height level is lower than the natural tailwater level, the jump may be drowned and a backwater effect takes place. The jump is 'pushed' upstream onto the chute and it may become similar to a plunging jet flow. In this case (JHRL < TWRL), a downward slope or a higher apron level followed by a drop may be incorporated in the apron design (Fig. 19.22).



(a) (b)

Fig. 19.21 Effect of tailwater levels on the Silverleaf weir QLD, Australia (courtesy of Mr J. Mitchell). Timber crib stepped weir completed in 1953 (5.1 m high structure): (a) operation at very low flow of the stepped weir and (b) operation at large overflow. Note the high tailwater level.

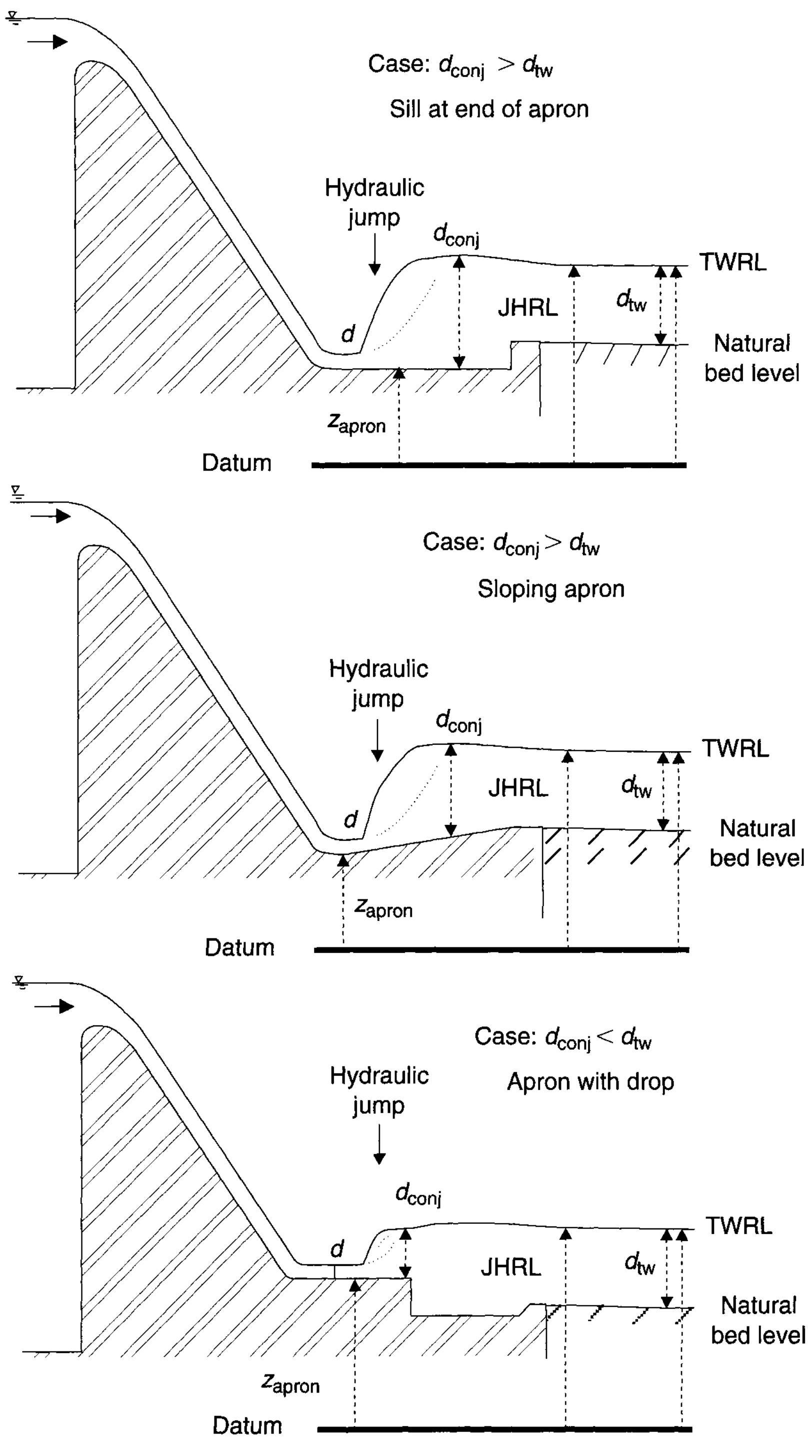


Fig. 19.22 Apron disposition to match the JHRL and TWRL.

19.5.3 Practical considerations

For hydraulic jump energy dissipators, it is extremely important to consider the following points:

- The energy dissipator is designed for the reference flow conditions (i.e. design flow conditions).
- For overflow discharges larger than the design discharge, it may be acceptable to tolerate some erosion and damage (i.e. scouring and cavitation). *However, it is essential that the safety of the dam is ensured.*
- For discharges smaller than the design discharge, perfect performances are expected: i.e. (a) the energy dissipation must be controlled completely and it must occur in the designed dissipator and (b) there must be no maintenance problem.

These objectives are achieved by (1) a correct design of energy dissipation basin dimensions, (2) a correct design of sidewalls (i.e. training walls) to confine the flow to the dissipator and (3) provision of an end sill and cut-off wall at the downstream end of the apron (Fig. 19.17). The training wall must be made high enough to allow for bulking caused by air entrainment in the chute flow and for surging in the hydraulic jump basin.

Note

Practically, it is uppermost important to remember that a hydraulic jump is associated with large bottom pressure fluctuation and high risks of scour and damage. As a result, the jump must be contained within the stilling basin for all flow conditions.

Discussion: air entrainment in stilling basin

The effects of air entrainment on hydraulic jump flow were discussed by Wood (1991). Chanson (1997) presented a comprehensive review of experimental data. Air is entrained at the jump toe and advected downstream in a developing shear layer. Experimental results suggested that the air bubble diffusion process and the momentum transfer in the shear flow are little affected by gravity in first approximation (Chanson and Brattberg, 2000).

Practically, the entrained air increases the bulk of the flow, which is a design parameter that determines the height of sidewalls. Further air entrainment contributes to the air–water gas transfer of atmospheric gases such as oxygen and nitrogen. This process must be taken into account for the prediction of the downstream water quality (e.g. dissolved oxygen content).

19.6 EXERCISES

A broad-crested weir is installed in a horizontal, smooth channel of rectangular cross-section. The channel width is 55 m. The bottom of the weir is 5.20 m above the channel bed. At design flow conditions, the upstream water depth is 6.226 m. Perform hydraulic calculations for design flow conditions.

Compute the flow rate (in the absence of downstream control) assuming that critical flow conditions take place at the weir crest, the depth of flow downstream of the weir and the horizontal sliding force acting on the weir.

Solution: $Q = 98\,\mathrm{m^3/s}$ and $F = 9.4\,\mathrm{MN}$ acting in the downstream flow direction.

Considering a 10 m wide, horizontal, rectangular stilling basin with a baffle row (Fig. E.19.1), the inflow conditions are $d = 0.1$ m and $V = 10$ m/s: (1) For one flow situation, the observed tailwater depth is 1.2 m. Calculate the magnitude of total force exerted on the baffle row. In which direction does the force applied by the flow onto the blocks act? (2) Variations of the tailwater level are expected. Calculate the force exerted by the flow onto the baffle row for tailwater depths ranging from 0.85 to 2.2 m. *The inflow conditions remain unchanged.* Plot your results on a graph. *The force exerted by the flow on the baffle row is positive in downstream flow direction.*

Solution: (1) $F = 22$ kN acting in the downstream flow direction. (2) The solution is shown in Fig. E.19.2.

An energy dissipator (hydraulic jump type) is to be designed downstream of an undershoot sluice gate. The invert elevation of the transition channel between the sluice gate and the dissipator is set at 70 m RL. The channel bed will be horizontal and concrete lined. The gate and energy dissipator will be located in a 20 m wide rectangular channel. The design inflow conditions of the prototype dissipator are discharge: 220 m^3/s and inflow depth: 1.1 m (i.e. flow depth downstream of gate). Four dissipator designs will be investigated:

(i) Dissipation by hydraulic jump in the horizontal rectangular channel (70 m RL elevation) (no blocks).

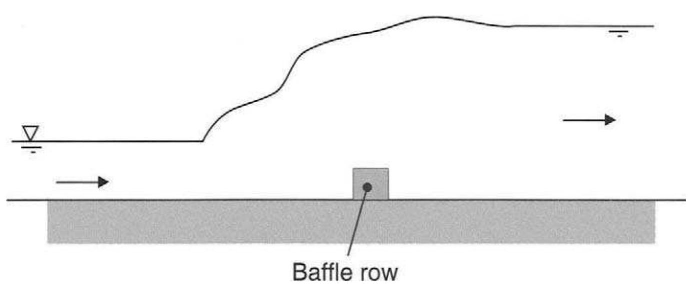


Fig. E.19.1 Sketch of a stilling basin with a baffle row.

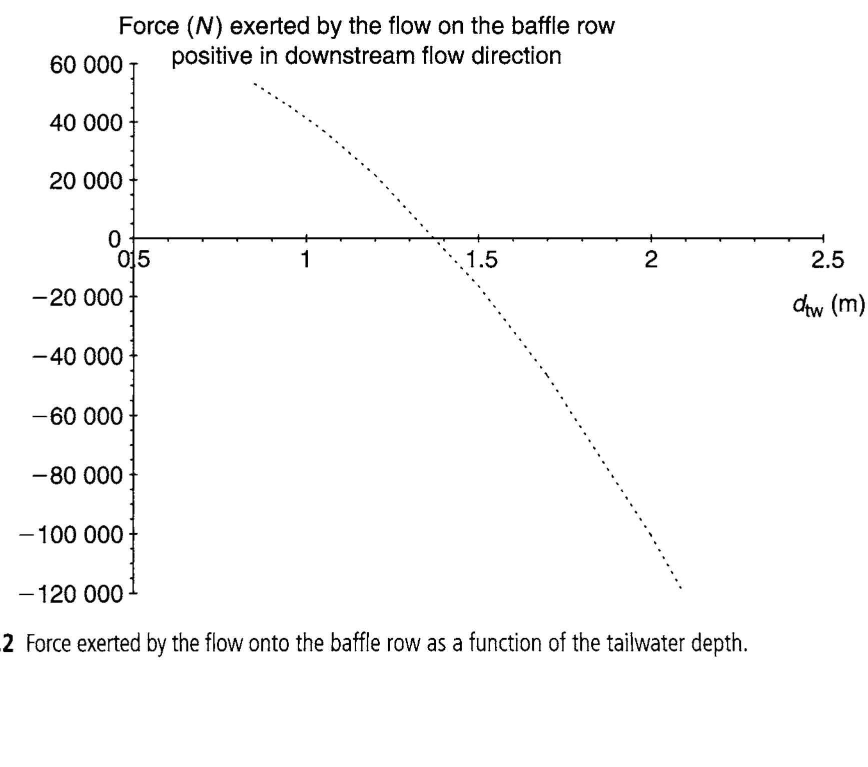


Fig. E.19.2 Force exerted by the flow onto the baffle row as a function of the tailwater depth.

(ii) Dissipation by hydraulic jump in the horizontal rectangular channel (70 m RL elevation) assisted with a single row of nine baffle blocks.
(iii) Dissipation in a standard stilling basin (e.g. USBR and SAF) set at 70 m RL elevation.
(iv) Dissipation in a specially designed stilling basin. The apron elevation will be set to match the tailwater conditions.

For the design (i): (a) compute the JHRL at design flow conditions.

For the design (ii): (b) compute the force on each baffle block when the tailwater level is set at 73.1 m RL at design inflow conditions. (c) In what direction is the hydrodynamic force acting on the blocks: i.e. upstream or downstream?

For the design (iii): (d) what standard stilling basin design would you select: USBR Type II, USBR Type III, USBR Type IV and SAF? (If you have the choice between two (or more) designs, discuss the economical advantages of each option.)

(e) Define in words and explain with sketch(es) the terms TWRL and JHRL. (Illustrate your answer with appropriate sketch(es) if necessary.)

Design a hydraulic jump stilling basin (design (iv)) to match the jump height and tailwater levels. The downstream bed level is at elevation 70 m RL. At design discharge, the tailwater depth equals 3.1 m.

(f) Calculate the apron level in the stilling basin to match the jump height to tailwater at design flow conditions.

(g) Alternatively, if the apron level remains at the natural bed level (70 m RL), determine the height of a broad-crested weir necessary at the downstream end of the basin to artificially raise the tailwater level to match the JHRL for the design flow rate.

An overflow spillway is to be designed with an uncontrolled ogee crest followed by a stepped chute and a hydraulic jump dissipator. The width of the crest, chute and dissipation basin will be 127 m. The crest level will be at 336.3 m RL and the design head above crest level will be 3.1m. The chute slope will be set at 51° and the step height will be 0.5 m. The elevation of the chute toe will be set at 318.3 m RL. The stepped chute will be followed (without transition section) by a horizontal channel which ends with a broad-crested weir, designed to record flow rates as well as to raise the tailwater level:

(a) Calculate the maximum discharge capacity of the spillway.
(b) Calculate the flow velocity at the toe of the chute.
(c) Calculate the residual power at the end of the chute (give the SI units). Comment.
(d) Compute the JHRL at design flow conditions (for a hydraulic jump dissipator).
(e) Determine the height of the broad-crested weir necessary at the downstream end of the dissipation basin to artificially raise the tailwater level to match the JHRL for the design flow rate.
(f) Compute the horizontal force acting on the broad-crested weir at design inflow conditions. In what direction will the hydrodynamic force be acting on the weir: i.e. upstream or downstream?
(g) If a standard stilling basin (e.g. USBR and SAF) is to be designed, what standard stilling basin design would you select: USBR Type II, USBR Type III, USBR Type IV or SAF?

Notes: In calculating the crest discharge capacity, assume that the discharge capacity of the ogee crest is 28% larger than that of a broad crest (for the same upstream head above crest). In computing the velocity at the spillway toe, allow for energy losses by using results presented in the book. The residual power equals $\rho g Q H_{\text{res}}$ where Q is the total discharge and H_{res} is the residual total head at chute toe taking the chute toe elevation as datum.

An overflow spillway is to be designed with an un-gated broad crest followed by a smooth chute and a hydraulic jump dissipator. The width of the crest, chute and dissipation basin will be 55 m. The crest level will be at 96.3 m RL and the design head above crest level will be 2.4 m. The chute slope will be set at 45° and the elevation of the chute toe will be set at 78.3 m RL. The stepped chute will be followed (without transition section) by a horizontal channel which ends with a broad-crested weir, designed to record flow rates as well as to raise the tailwater level.

(a) Calculate the maximum discharge capacity of the spillway. (b) Calculate the flow velocity at the toe of the chute. (c) Calculate the Froude number of the flow at the end of the chute. (Comment.) (d) Compute the conjugate flow depth at design flow conditions (for a hydraulic jump dissipator).

The natural tailwater level (TWRL) at design flow conditions is 81.52 m RL: (e) Determine the apron elevation to match the JHRL and the TWRL at design flow conditions.

(f) If a standard stilling basin (e.g. USBR and SAF) is to be designed, what standard stilling basin design would you select: USBR Type II, USBR Type III, USBR Type IV or SAF?

(*Note*: In computing the velocity at the prototype spillway toe, allow for energy losses.)

A weir is to have an overflow spillway with ogee-type crest and a hydraulic jump energy dissipator. Considerations of storage requirements and risk analysis applied to the 'design flood event' have set the elevation of the spillway crest 671 m RL and the width of the spillway crest at $B = 76$ m. The maximum flow over the spillway when the design flood is routed through the storage for these conditions is 1220 m^3/s. (*Note*: the peak *inflow* into the reservoir is 3300 m^3/s.) The spillway crest shape has been chosen so that the discharge coefficient at the maximum flow is 2.15 (SI units). For the purpose of the assignment, the discharge coefficient may be assumed to decrease linearly with discharge down to a value of 1.82 at very small discharges. The chute slope is 1V:0.8H (i.e. about 51.3°). The average bed level downstream of the spillway is 629.9 m RL and the TWRC downstream of the dam is defined as follows:

Discharge (m^3/s)	TWRL (m)	Discharge (m^3/s)	TWRL (m)
0	629.9	400	634.45
25	631.5	500	634.9
50	631.95	750	635.75
100	632.6	950	636.35
150	633.0	1220	637.1
250	633.65	1700	638.2

Design three options for the hydraulic jump stilling basin to dissipate the energy at the foot of the spillway as follows: (A) apron level lowered to match the JHRC to the TWRC; (B) apron level set at 632.6 m RL and tailwater level raised artificially with a broad-crested weir at the downstream end of the basin; (C) apron level set at 629.9 m RL (i.e. average bed level) *but* spillway width B is changed to match the JHRC to the TWRC.

In each case use a sloping apron section if this will improve the efficiency and/or economics of the basin. In computing the velocity at the foot of the spillway, allow for energy losses. The design calculations are to be completed and submitted in two stages as specified below: Stage 1 (calculations are to be done for 'maximum' flow *only*) and Stage 2 (off-design calculations).

Stage 1: (a) Calculate apron level to match the JHRC to natural TWRC at 1220 m^3/s. (b) Calculate height of broad-crested weir to raise local TWRC to JHRC at 1220 m^3/s for an apron level at 632.6 m RL. (c) Calculate B to match JHRC to natural TWRC at 1220 m^3/s for an apron level at 629.9 m RL.

Stage 2: For both cases (A) and (B): (a) Calculate the JHRC for all flows up to and including 1220 m^3/s in sufficient detail to plot the curve. (b) Use the results from Stage 1 to check the correctness of your calculation for 1220 m^3/s. (c) Plot the JHRC from (a) and the TWRC (natural or local, as required) on the same graph and determine the height of sloping apron required to match JHRC and TWRC for all flows. (This is obtained from the greatest vertical separation of the two curves as plotted. If the maximum separation occurs at a flow less than 200 m^3/s, set the height of the sloping apron to the separation of the curves at 200 m^3/s.) (d) Draw to scale a dimensioned sketch of the dissipator.

Summary sheet (Stage 1)

(a)	Apron level		m RL
(b)	Height of broad-crested weir above apron		m
	Minimum crest length		m
(c)	Spillway crest width		m
(d)	If the crest width is changed from 76 m (design (C)), and for a peak inflow into the reservoir of 3300 m^3/s, will the 'maximum' spillway overflow change from 1220 m^3/s?	Yes/No……	
	Give reason for answer		

Summary sheet (Stage 2)

Design A	Apron level		m RL
	Height of sloping apron		m
	Length of horizontal apron		m
Design B	Height of broad-crested weir above apron		m
	Height of sloping apron		m
	Length of horizontal apron		m

For each case (A) and (B), supply plotted curves of JHRC and TWRC (natural or raised as required). Supply dimensioned drawing of dissipators to scale.

A weir is to have an overflow spillway with ogee-type crest and a hydraulic jump energy dissipator. Considerations of storage requirements and risk analysis applied to the 'design flood event' have set the elevation of the spillway crest 128 m RL and the width of the spillway crest at $B = 81$ m. The maximum flow over the spillway when the design flood is routed through the storage for these conditions is 1300 m^3/s. (*Note*: the peak *inflow* into the reservoir is 3500 m^3/s.) The spillway crest shape has been chosen so that the discharge coefficient at the maximum flow is 2.15 (SI units). For the purpose of the assignment, the discharge coefficient may be assumed to decrease linearly with discharge down to a value of 1.82 at very small discharges. The chute slope is 1V:0.8H (i.e. about 51.3°). (*Note*: both stepped and smooth chute profiles will be considered.)

The average bed level downstream of the spillway is 86.9 m RL and the TWRC downstream of the dam is defined as follows:

Discharge (m^3/s)	TWRL (m)	Discharge (m^3/s)	TWRL (m)
0	86.9	400	91.45
25	88.5	520	91.9
50	88.95	800	92.75
100	89.6	1000	93.35
150	90.0	1300	94.1
250	90.65	1800	95.2

Design three options for the hydraulic jump stilling basin to dissipate the energy at the foot of the spillway as follows: (A) smooth concrete chute with apron level lowered to match the JHRC to the TWRC; (B) stepped concrete chute (step height $h = 0.15$ m) with apron level lowered (or raised) to match the JHRC to the TWRC; (C) smooth concrete chute with apron level set at 86.9 m RL (i.e. average bed level) *but* spillway width B is changed to match the JHRC to the TWRC.

In each case use a sloping apron section if this will improve the efficiency and/or economics of the basin. In computing the velocity at the foot of the spillway, allow for energy losses. The design calculations are to be completed and submitted in two stages: Stage 1 for design flow conditions (1300 m^3/s) and Stage 2 for non-design flow conditions.

Stage 1: (a) Calculate apron level to match the JHRC to natural TWRC at 1300 m^3/s. (b) Calculate apron level to match the JHRC to natural TWRC at 1300 m^3/s (with a stepped chute). (c) Calculate B to match JHRC to natural TWRC at 1300 m^3/s for an apron level at 86.9 m RL.

Stage 2: For both cases (A) and (B): (a) Calculate the JHRC for all flows up to and including 1300 m^3/s in sufficient detail to plot the curve. (b) Use the results from Stage 1 to check the correctness of your calculation for 1300 m^3/s. (c) Plot the JHRC from (a) and the TWRC (natural or local, as required) on the same graph and determine the height of sloping apron required to match JHRC and TWRC for all flows (up to design flow conditions). (This is obtained from the greatest vertical separation of the two curves as plotted. If the maximum separation occurs at a flow less than 200 m^3/s, set the height of the sloping apron to the separation of the curves at 200 m^3/s.) (d) Draw to scale a dimensioned sketch of the dissipator. (e) Discuss the advantages and inconvenience of each design case, and indicate your recommended design (with proper justifications).

Summary sheet (Stage 1)

(a)	Apron level		m RL
(b)	Apron level		m RL
(c)	Spillway crest width		m
(d)	If the crest width is changed from 81 m, will the 'maximum' flow rate change from 1300 m^3/s	Yes/No......	
	Give reason for answer		

Summary sheet (Stage 2)

Design A	Apron level		m RL
	Height of sloping apron		m
	Length of horizontal apron		m
Design B	Apron level		m RL
	Height of sloping apron		m
	Length of horizontal apron		m

For each case (A) and (B), supply curves of JHRC and TWRC (natural or raised as required). Supply dimensioned drawings of dissipators to scale.