

A review of accidents and failures of stepped spillways and weirs

H. Chanson, ME, ENSHM Grenoble, INSTN, PhD(Cant), DEng(Qld), EurIng, MIEAust, MAHR

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■ Recent advances in technology have permitted the construction of large reservoirs and chutes. These developments have necessitated the development of new design and construction techniques, particularly with the provision of adequate flood release facilities and safe energy dissipation. The latter may be achieved by the construction of steps on the chute. This paper discusses issues of the operation and safe design of stepped chutes. It describes the operation of several structures worldwide, focusing on accidents and failures. The causes of failures are analysed and discussed. Problems specific to stepped-chute design are discussed in the light of over 20 accidents, and common mistakes are described. The safe operation of several old stepped chutes and the experience gained from past failures must be used to gain further expertise and knowledge about the hydraulics of stepped chutes.

Keywords: dams, barrages & reservoirs; hydraulics & hydrodynamics; water supply

Notation

d_c	critical flow depth: m
H_{dam}	dam height: m
h	step height: m
l	step length: m
Q_{des}	design discharge: m^3/s
Q_w	water discharge: m^3/s
q_{des}	design discharge per unit width: m^2/s
q_w	water discharge per unit width: m^2/s
W	channel width: m
θ	slope of the pseudo-bottom formed by step edges

Introduction

Stepped channels and spillways have been used for more than 3000 years.^{1,2} The development of new construction materials such as roller-compacted concrete (RCC) and strengthened gabions has created a renewed interest in stepped chutes. The steps increase the rate of energy dissipation along the chute and reduce the size of the downstream energy dissipation basin required. Stepped cascades are used also for in-stream re-aeration and removal of volatile organic components (VOC) in water treatment plants.

2. Satisfactory performances of stepped cascades were acknowledged in classical textbooks (e.g. Humber,³ Schuyler⁴ and Wegmann⁵). Recently, the operation of modern RCC structures has attracted some attention. The Shuidong Dam, a 57 m high RCC dam in China, was equipped with a 60 m wide stepped chute ($h = 0.9$ m, $\theta = 57^\circ$) designed for $6012 \text{ m}^3/\text{s}$. In May 1994, the stepped chute discharged up to $90 \text{ m}^2/\text{s}$ (or $90 \text{ m}^3/\text{m}$ per second) successfully and the event was videotaped.⁶ The Ocoee Dam No. 2, a timber crib structure with downstream RCC buttresses, is intentionally overtopped for river rafting over 82 times per year. In 1990, the structure passed adequately a flood ($q_w = 11.9 \text{ m}^2/\text{s}$).⁷ Despite the successes, little attention has been paid to basic safety issues. Jansen⁸ discussed generally the problem of public safety. Goubet⁹ and Chanson¹ argued critically for and against the stepped chute design.

3. The present paper reviews the operation of stepped channels with an emphasis on accidents, incidents and failures. The purpose of the paper is to provide a critical review of the overall experience in this field, to alert professional engineers of potential engineering failures and to recommend safer design and operation methods.

Stepped-channel hydraulics

4. Stepped channels may be characterized by two types of flow: nappe flow and skimming flow (Fig. 1) (e.g. Rajaratnam¹⁰ and Chanson¹¹). At low flow rates and for relatively large step heights, the water flows from one step onto another as a succession of free-falling nappes (i.e. nappe flow). At larger flow rates, the flow skims over the step edges with the formation of recirculating vortices between the main stream and the step corners. The hydraulic characteristics differ significantly between the two regimes.¹

5. The transition between nappe and skimming flow is related to the flow rate, chute slope, step geometry and local flow properties at each step. The reanalysis of a large amount of model data provides an updated limiting condition for skimming flow:

$$\frac{d_c}{h} > 1.1 - 0.40 \frac{h}{l} \quad (1)$$

where d_c is the critical flow depth, h is the step



H. Chanson, Senior Lecturer, Fluid Mechanics, Hydraulics and Environmental Engineering, Department of Civil Engineering, University of Queensland, Brisbane

height and l is the step length (Fig. 2). Equation (1) is limited to flat, horizontal steps for h/l ranging from 0.15 to 1.4 and it characterizes the onset of skimming flow for uniform or quasi-uniform equilibrium flows down prismatic, rectangular channels. Its accuracy is within $\pm 30\%$ (Fig. 2). For accelerating or decelerating flows (i.e. rapidly varying flows), equation (1) is inaccurate and a more complete analysis is required (e.g. Chanson²³).

Operation, accidents, incidents and failures

6. Over the past two hundred years, at least 20 accidents or failures have involved hydraulic structures equipped with stepped spillways (Table 1). Although two failures, at the Puentes and St Francis dams, were unrelated to the spillway operation, several accidents were caused by the selection of a stepped design (e.g. New Croton Dam). Chanson¹ described in detail some of these cases. Further details of the accidents and failures listed in Table 1 are presented below in alphabetical order.

Bonshaw Weir (Australia)

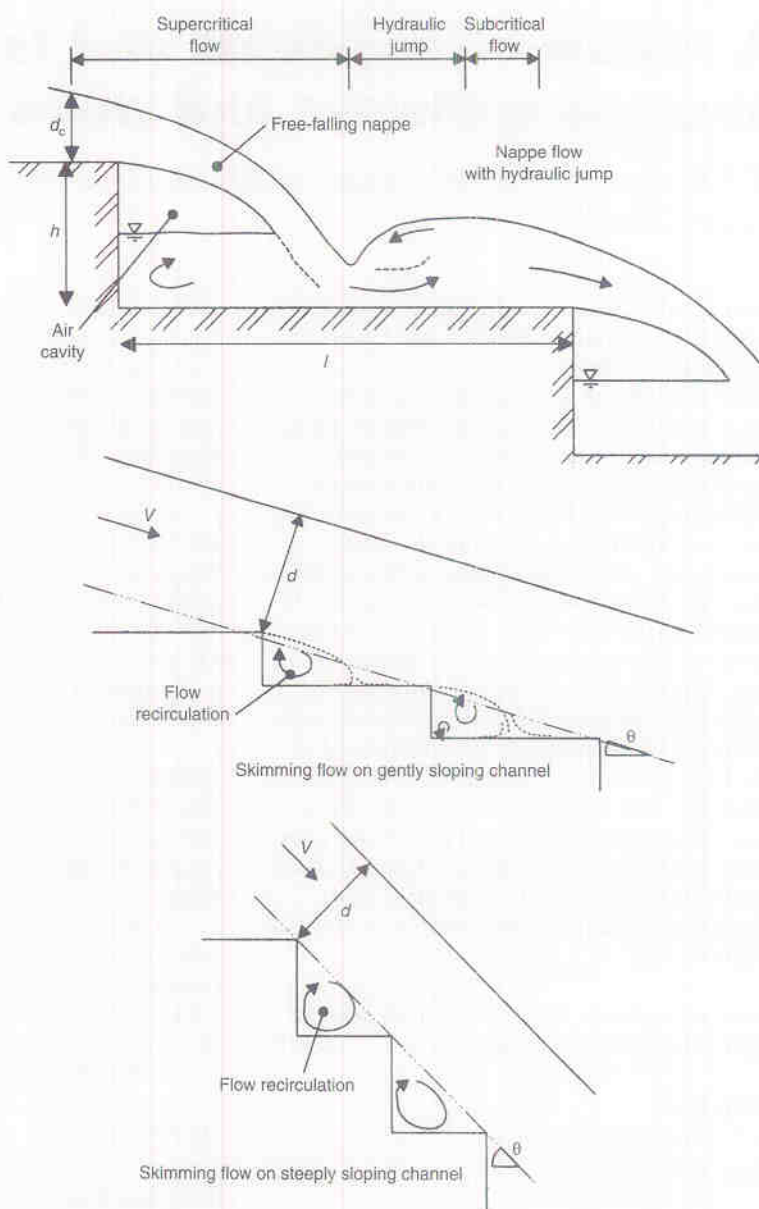
7. The (first) Bonshaw Weir was completed in May 1953 (Fig. 3). Located on the Dumaresq River, it was designed for irrigation water, the primary industry being tobacco. The 3.7 m high weir was a timber crib piled structure (144.5 m long), designed with four steps ($h = 0.9$ m).

8. A major flood occurred in January and early February 1956. Intense rainfall was recorded in the catchment (e.g. 185 mm on 21 January and 225 mm on 22 January 1956 at nearby Columba) and the river was in flood for about two months. The maximum recorded stream heights were 11.13 m in January and 8.76 m in February at Cunningham, located 60 km downstream. The nearby town of Texas was flooded and inhabitants could not cross the river for several weeks.

9. The failure of the Bonshaw Weir occurred during the overflow event. Both abutments failed. The left abutment was bypassed and it was heavily scoured. The right abutment and a section of the weir were washed away. The failure resulted from an average quality of construction and poor abutment foundations.

10. In 1958, a new weir was built, at the same site, in steel sheet piles and concrete (Fig. 3(c)). Interestingly, the new weir is only 2.4 m high and it is shorter than the first weir.

11. It is worth comparing the operation of the (first) Bonshaw Weir with that of the Cunningham Weir, another timber crib piled weir, located 60 km downstream. Completed in 1954, the Cunningham Weir suffered only minor damage during the 1956 flood. It is still in use and in good condition today.



Bucca Weir (Australia)

12. Completed in 1987, the Bucca Weir is a 12 m high RCC weir equipped with an overflow stepped spillway (Fig. 4). The design has a broad crest inclined upwards, followed by a 1.6 m drop and seventeen 0.6 m high steps ($h/l = 2$). The maximum design discharge is 7250 m³/s.

13. Shortly after completion, loud noises were heard during low overflows. At the brink of the first step (i.e. the downstream end of the crest), the water was deflected away from the chute as a free-falling nappe and the air cavity was not ventilated. Nappe instabilities developed and generated low-frequency vibrations in the structure and loud noises that were not acceptable to nearby residents. The problem was remedied by the installation of splitters on the crest edge to ventilate the nappe at low flows. The splitter devices are

Fig. 1. Sketch of nappe and skimming flows on stepped chutes

tiltable and drop at medium to large overflows.

Dartmouth Dam (Australia)

14. Completed in 1977, the Dartmouth Dam is a 180 m high earth and rockfill structure (Fig. 5). The spillway, located on the left bank, consists of an approach channel and a concrete-lined crest and chute (width 91.4 m), followed by a ten-step unlined rock cascade ($h = 15$ m, $W = 300$ –350 m). The rock is granitic gneiss and the step excavation was used for the dam rockfill. The design discharge capacity is $2755 \text{ m}^3/\text{s}$.

15. During the 1996 flood ($Q_w \approx 225 \text{ m}^3/\text{s}$), the unlined rock spillway was partially damaged. Although the discharge was much lower than the design flow rate, significant erosion was observed at several steps because of flow concentration in the right-hand side of the cascade.

16. Repairs were undertaken in 1996 and 1997 that included the provision of shotcrete protection and the construction of anchored concrete retaining walls in the three upstream steps. Walls were built to prevent concentration of low-level flows in some areas.

17. It is worth comparing the characteristics of the Dartmouth Dam spillway with those of other unlined rock cascades (Table 2). Overall, the maximum discharge capacity is lower than at the La Grande 2 cascade, but the step height is one of the largest, leading to high nappe impact velocities in excess of 17 m/s. For large impact velocities, deep pools of water are required to cushion the jet impact pressures on the rock.

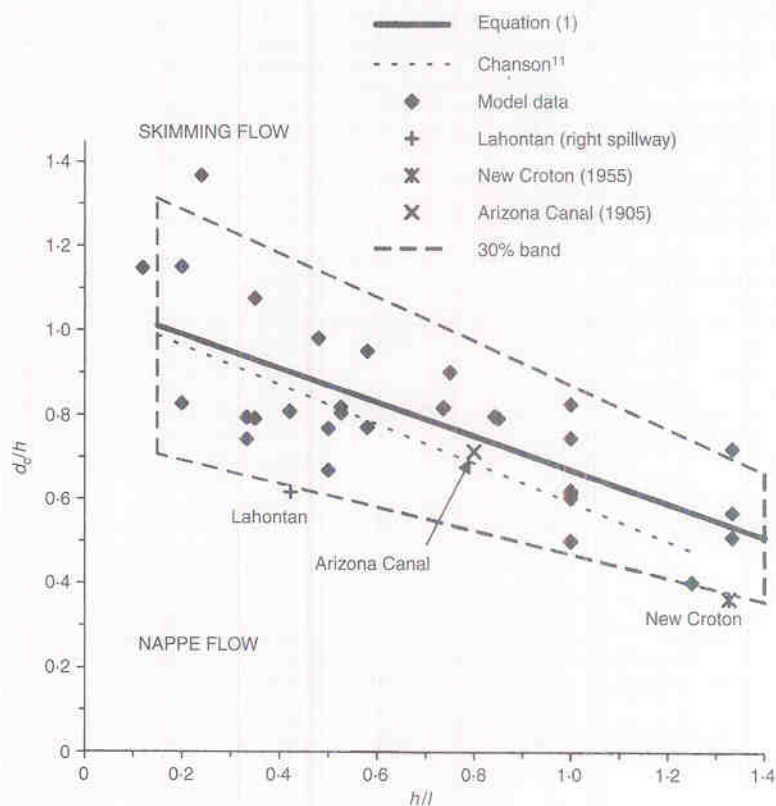
Hilliard Creek Weir (Australia)

18. The weir was completed in 1948 on Hilliard Creek, less than 1 km from the estuary. It was built for water supply and to prevent salt intrusion. The 3.4 m high weir is a timber crib piled structure. Repaired once in 1963, the weir is now in ruins because of the lack of maintenance. The timber crib skeleton and some rockfill are visible, showing the construction technique, but the weir is not holding water, i.e. it is porous.

19. Warning signs are posted around the weir indicating that it is unsafe to approach or walk on the structure. Surprisingly, the weir was still listed in a 1997 annual report of the Queensland state government.

Kobila Dam (Slovenia)

20. The Kobila Dam was a timber crib structure filled with rocks. Built in 1586 across the Idrijca River, the dam was over 10 m high and the crest length was about 20 m.²⁴ The structure was equipped with an overflow stepped spillway. The spillway crest was protected from floating logs with wooden planks.



21. The dam operated continuously up to 1948, when it was destroyed by a flood after 362 years of operation. During World War II and the subsequent years, the dam was not maintained. It is believed that the failure resulted from the lack of maintenance and care for nearly ten years.

Fig. 2. Onset of skimming flow: comparison between equation (1), Chanson's correlation¹¹ and model data¹²⁻²²

Moscovite earth dams (Russia)

22. Between 1940 and 1980, Soviet engineers developed the concept of overflow earth dams (prior to the introduction of roller-compacted concrete) under the leadership of Professor P. I. Gordienko.^{28,29} The spillway design consists of a trapezoidal channel made of precast concrete blocks laid on the downstream slope of the embankment. A protective drainage layer must be installed between the earthfill and the blocks to prevent seepage during overflows.

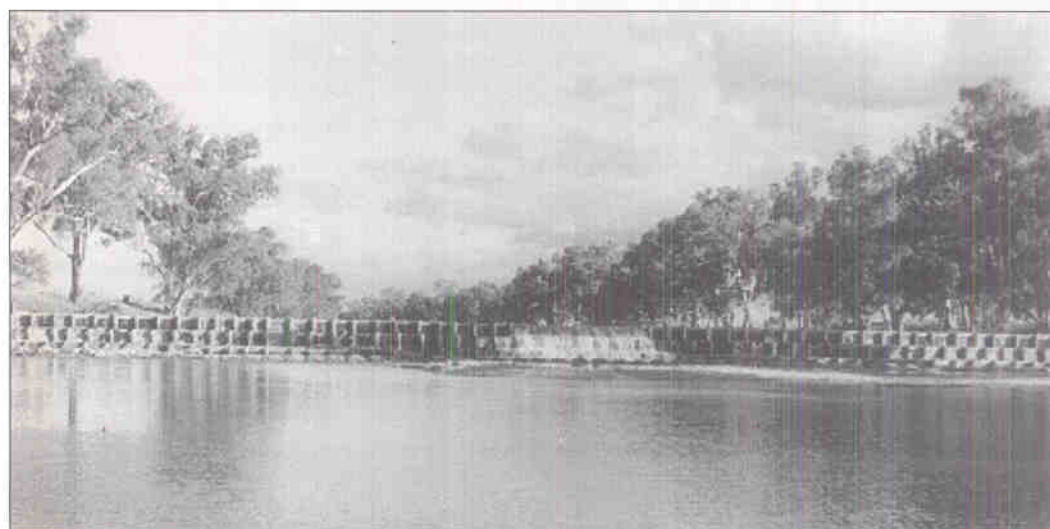
23. In 1978 five overflow embankments were built in the Moscow region.²⁷ The dams were about 7–15 m high. Each spillway system comprised an overflow with precast concrete blocks (about 60 to 120 kg each) and was designed to pass between 30 and 60 m^3/s .

24. Two structures failed during the first year of operation. Their protective layer did not perform adequately and the earthen embankments were eroded. In one case (the Zhilevski Dam), the smallest stone size of the drainage layer was smaller than the drainage holes in the concrete blocks. In absence of a geotextile fabric, drainage layer material and earthfill

Table 1. Summary of accidents, incidents and failures (sorted by year of accident)*

Dam/reservoir	Year of accident	Years of construction	Accident/failure construction details	Lives lost
Puentes Dam, Spain	1802	1785–1791	Dam break caused by a foundation failure Gravity dam with overflow spillway: $H_{\text{dam}} = 50$ m	608
Sreeramadavara Weir, India	1842, 1844, 1859, 1860, 1863	–	Failures Rubble masonry weir: $h = 0.348$ m, $l = 0.762$ m, $\theta = 21.8^\circ$	–
Muddoor Weir, India	1850s	–	Failure Masonry weir: $H_{\text{dam}} \sim 4.4$ m, $h \sim 0.4$ m, $l \sim 0.3$ m	–
Narora Weir, India	1898	1877	Failure of stilling basin during very large overflow ($q_w = 11.9$ m ² /s) Drop structure: $H_{\text{dam}} = 3.05$ m, $W = 1160$ m, $q_{\text{des}} = 2.93$ m ² /s	–
Minneapolis Mill Dam, USA	1899	1893–1894	Dam break during a small spill ($q_w = 0.04$ m ² /s) caused by cracks resulting from ice pressure on the dam Gravity dam with overflow spillway: $H_{\text{dam}} = 5.5$ m, $W = 51.8$ m, 9 steps, $h = 0.61$ m	–
Arizona Canal Dam	1891 and 1905	1887	Partial destruction of the dam during a flood ($q_w = 11.3$ m ² /s) caused by foundation problems and timber deterioration Timber crib dam: $H_{\text{dam}} = 10$ m, $W \approx 245$ m, 3 steps, $q_{\text{des}} \approx 33$ m ² /s	–
Warren Dam, Australia	1917	1916	Dam overtopped ($Q_w = 128$ m ³ /s) without damage Concrete gravity dam: $H_{\text{dam}} = 17.4$ m, 4 steps, $h = 0.37$ m, $Q_{\text{des}} \approx 100$ m ³ /s	None
St Francis Dam, USA	1928	1926	Dam break caused by foundation failure Arched gravity dam: $H_{\text{dam}} = 62.5$ m, $h = 0.4$ m	450
Lahontan Dam, USA	1930s	1915	Damaged spillway concrete caused by freezing and thawing, and hydraulic scour Earthfill dam with concrete spillway: $H_{\text{dam}} = 49$ m, $Q_{\text{des}} = 850$ m ³ /s. Right stepped spillway: $h = 3.35$ m, $W = 45.7$ m. Left stepped spillway: $h = 3.05$ m, $W = 76.3$ m and 45.7 m	None
Kobila Dam, Slovenia	1948	1586	Dam destruction during a flood caused by lack of maintenance Timber crib dam: $H_{\text{dam}} = 10$ m	–
New Croton Dam, USA	1955	1892–1905	Spillway damage during flood releases ($Q_w \approx 651$ m ³ /s) Masonry gravity dam: $H_{\text{dam}} = 90.5$ m, $W \approx 305$ m, $h = 2.1$ m, $Q_{\text{des}} \approx 1550$ m ³ /s	None
Bonshaw Weir, Australia	1956	1952–1953	Weir failure resulting in abutment bypass and failure during major flood overflow Timber crib piled weir: $H_{\text{dam}} = 3.7$ m, 4 steps, $h = 0.9$ m, $Q_{\text{des}} = 1560$ m ³ /s	None
Goulburn Weir, Australia	1978	1891	(1) Gate failure caused by corrosion and (2) foundation stability problem Concrete gravity dam: $H_{\text{dam}} = 15$ m, $W = 126$ m, $h = 0.5$ m, $Q_{\text{des}} = 1980$ m ³ /s	None
Moscovite earth dams, Russia	1978–1980	1978	Failure of two overflow earth dams caused by incorrect drainage layer construction Earthfill embankments with overflow stepped spillway made of precast concrete blocks: $H_{\text{dam}} = 7$ – 15 m, $Q_{\text{des}} \approx 30$ – 60 m ³ /s	–
Binda Weir, Australia	1986	1950–1953	Weir destroyed (blown) because unsafe (lack of maintenance) Timber crib piled weir: $H_{\text{dam}} = 5.2$ m, 5 steps	None
Silverleaf Weir, Australia	1995	1951–1953	Weir overtopping during refurbishment works (no damage) Timber crib piled weir: $H_{\text{dam}} = 5$ m, 4 steps	None
Dartmouth Dam, Australia	1996	1977	Unlined rock steps damaged by flow concentration during low spill ($Q_w \approx 225$ m ³ /s) Earth/rockfill embankment with unlined rock cascade spillway: $H_{\text{dam}} = 180$ m, $W = 91.4$ (concrete crest) and 300 – 350 m (cascade), $h = 15$ m, $Q_{\text{des}} = 2755$ m ³ /s	None
Bucca Weir, Australia	1987	1986–1987	Nappe deflection at first step associated with loud noise and vibrations at low overflows RCC overflow stepped weir: $H_{\text{dam}} = 11.8$ m, $W = 131$ m, $h = 0.6$ m, $Q_{\text{des}} = 7250$ m ³ /s	None
Debris dams, Taiwan	1994	1994	Ill-designed debris control system Three concrete debris dams	None
Hilliard Creek Weir, Australia	1990s	1948	Ruins (lack of maintenance) Timber crib piled weir: $H_{\text{dam}} = 3.4$ m	None

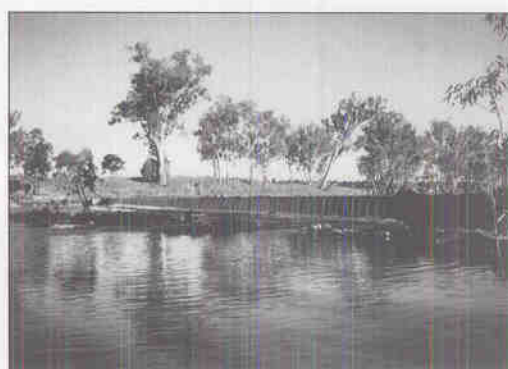
* – Unknown. References: Breznik,²⁴ Buckley,²⁵ Chanson,¹ Douma and Goodpasture,²⁶ Pravdivets²⁷ and present study.



(a)



(b)



(c)

Fig. 3. Bonshaw Weir: (a) first Bonshaw Weir around 1953, view from downstream (courtesy of the Department of Natural Resources); (b) small overflow on 19 April 1955 (courtesy of the Department of Natural Resources); (c) view of the present weir from the left bank: note the ruins of the first weir downstream of the sheet pile cascade (photograph taken in February 1998)

were sucked into the drainage holes and washed away during a storm, leading to the failure.

Lahontan Dam (USA)

25. Built in 1915, the Lahontan Dam is a 49 m high earth structure across the Carson River in Nevada. The dam is owned by the US Bureau of Reclamation. It is equipped with two stepped spillways, on the left and right abutments. The maximum discharge capacity is 424.8 m³/s per spillway. The right spillway consists of two drops followed by a long, curved, flat chute with converging walls, leading to a series of seven steps ($h = 3.35$ m, $\theta = 22.9^\circ$, $W = 45.72$ m). The left spillway consists of a series of six steps ($h = 3.05$ m, $\theta = 26.6^\circ$, $W = 76.3$ m), a converging flat chute section and a curved stepped channel (three steps, $h = 3.05$ m, $l = 6.096$ m, $W = 45.72$ m) with a curvature radius ranging from 39 to 50 m. The right and left spillways meet in a circular stilling basin at the dam toe.

26. In the late 1930s, the concrete of the spillways was in poor condition, particularly in the right spillway, although only two floods

had been recorded since construction.²⁶ It was suggested that the concrete construction was damaged by freezing and thawing.⁸ Hydraulic tests showed that a hydraulic jump took place at the end of the flat chutes.²⁶ A reanalysis showed further that the right spillway could experience a transition of the flow from nappe to skimming at the maximum flow rate ($d_c/h = 0.62$). It is thought that the spillway damage was initiated by scour, associated with the presence of hydraulic jumps and further enhanced by flow instabilities at the transition between nappe flow and skimming flow in the right spillway.

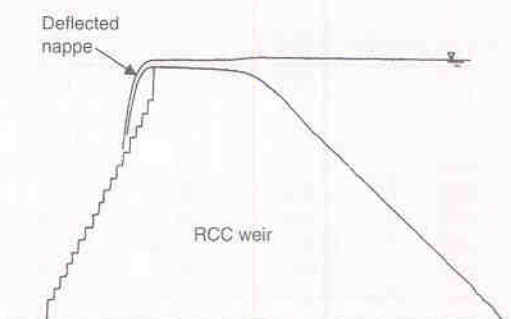
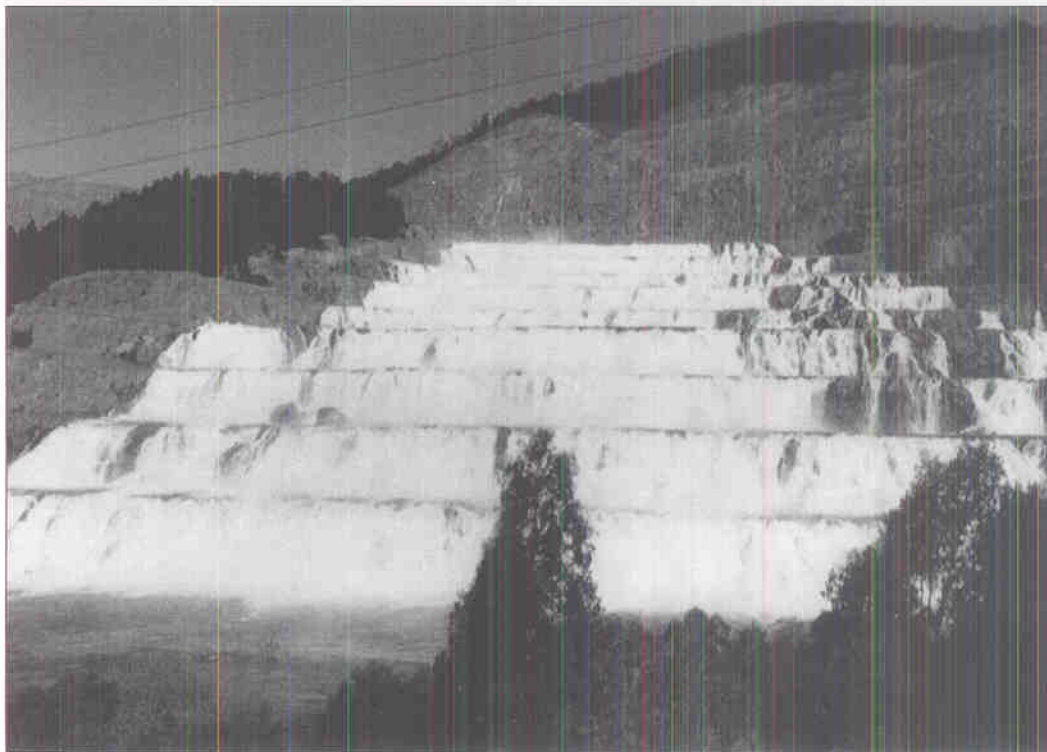
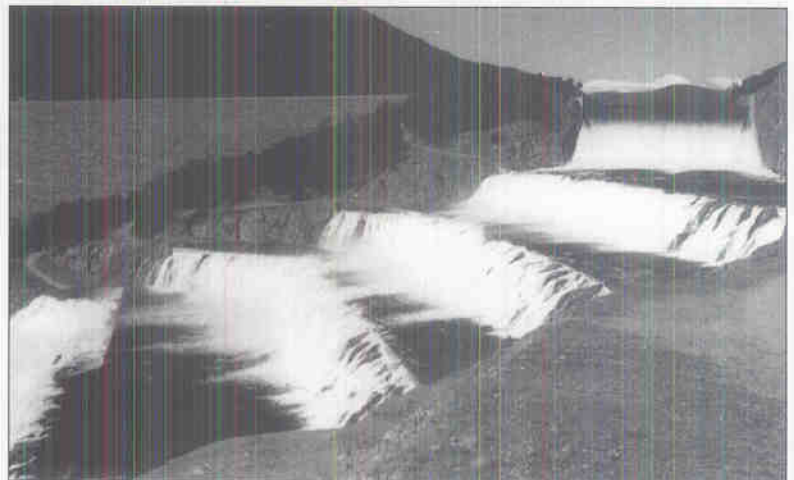


Fig. 4. Bucca Weir spillway: low overflow (cross-section)



(a)

Fig. 5. Dartmouth Dam spillway during the 1996 overflow ($Q_w \sim 225 \text{ m}^3/\text{s}$) (courtesy of Mr Jeffery, Goulburn-Murray Water): (a) spillway viewed from downstream: note the flow concentration in the middle of the photograph and the absence of flow on the right edge of the photograph; (b) details of the cascading flow from the left bank, showing the concrete-lined chute and the first steps



(b)

Narora Weir (India)

27. Completed in 1877, the Narora Weir was part of the Lower Ganges Canal. It was a 3.05 m high, 1160 m long drop structure built in brickwork.²⁵ It was designed to pass $3400 \text{ m}^3/\text{s}$.

28. The weir failed in March 1898. Concrete and stonework of the stilling basin were damaged over a 107 m wide section. The weir was discharging $13\,800 \text{ m}^3/\text{s}$ at the time of failure, i.e. four times the design flow rate! The weir was strengthened in 1898 and 1899.

New Croton Dam (USA)

29. Completed in 1905, the New Croton Dam was the highest dam in the world at that time. It was designed to supply water to the city of New York and it still does. The spillway was designed with a stepped geometry specifically to dissipate the energy of the flow ($Q_{\text{des}} = 1550 \text{ m}^3/\text{s}$).³⁰ The dam was built of concrete and the stepped chute was protected

with granite blocks (step geometry: $h = 2.13 \text{ m}$, $l = 1.6 \text{ m}$).

30. In October 1955, the spillway was heavily damaged during a $650 \text{ m}^3/\text{s}$ overflow.

Table 2. Characteristics of unlined rock stepped cascades*

Name	Year	Q_{des} : m^3/s	W : m	h : m	Comments
Ternay, France	1868	—	—	0.3–0.8	Still in use. Dam height 41 m
La Tâche, France	1891	—	2.2	2.4–5.4	Still in use. Dam height 51 m
Junction Reefs, Australia	1897	37.9	20	2–4	Still in use. Dam height 19 m. Reservoir fully silted
Bellfield, Australia	1966	20.9	23	12–2	Dam height 55 m
Dartmouth, Australia	1977	2755	300–350	15	Dam height 180 m
La Grande 2, Canada	1982	16 140	122	9.1–17.8	Dam height 134 m

* – Data not available.



(a)

Fig. 6. Silverleaf Weir: (a) during construction in April 1952 (courtesy of Queensland Department of Natural Resources); (b) view from downstream right bank during a low overflow, shortly after completion (courtesy of Mr J. Mitchell); (c) refurbished weir viewed from left bank: the upstream concrete wall is clearly visible (photograph taken in November 1997)



(b)



(c)

Extensive repairs were carried out between 1955–1957,³¹ and the stepped chute is still used. The spillway damage was caused by an improper spillway intake, and, further, overflow conditions at the transition between nappe and skimming flow aggravated the damage.¹

Silverleaf Weir (Australia)

31. The Silverleaf Weir was built between 1951 and 1953 for stock water. The 5.1 m high weir was designed as a timber crib piled weir with concrete protection of the four steps (Fig. 6). The upstream wall was sealed with a

clay embankment held by boulders set in mortar. The weir was refurbished in 1995 with the installation of a new outlet, construction of an upstream vertical concrete wall to replace the clay fill and to heighten the weir crest by an additional concrete step, and construction of a counterweir downstream.

32. During the refurbishment, an earthfill cofferdam was built 20 m upstream to dewater the weir. A flood event took place while the weir was dry and the clayfill was removed. The cofferdam was overtopped on 7 February 1995 and the resulting flood wave submerged the weir. The weir was overtopped by over 1 m of water and overtopping lasted for 24 h, with seepage through the timber cribs for a further 12 h before repairs could start. After the event, inspections showed that the weir sustained the flood wave without major damage. The overtopping was filmed and the document highlighted the good stability of the structure, the significant energy dissipation during large overflow and the seepage through the rockfill material during the receding flood surge.

33. The Silverleaf Weir refurbishment was completed as planned and the weir is still in use (Fig. 6(c)). Overall, the experience suggests that the timber weir design was sound and the structure is very stable despite its age.

Sreeramadavara and Muddoor weirs (Mysore, India)

34. Sankey³² described the accidents affecting stepped weirs in Mysore, India. The Sreeramadavara Weir was originally a mass of rubble and large stones. The downstream face consisted of stones 'well shaped and laid in regular courses, each course projecting about 2½ feet beyond the upper one'. The step height and length were 0.348 m and 0.762 m, respectively (i.e. $\theta = 21.8^\circ$). The weir was breached five times and was repaired in 1842, 1844, 1859, 1860 and 1863–64. Interestingly, the stepped design was abandoned and the latest weir had a steep, smooth downstream face. Another structure, the Muddoor Weir, failed and was recon-

structed with a masonry stepped overflow. The newer design had a 2.74 m long, broad crest followed by eleven steps ($H_{\text{dam}} \sim 4.4$ m, $h \sim 0.4$, $l \sim 0.3$ m).

Debris dams in Taiwan

35. A different experience is provided by the story of three check dams in eastern Taiwan. Three drop structures were completed in 1994 to protect a catchment on the eastern coast. Each dam was a straight concrete wall about 12 m high. All the dams failed to control debris flows and to protect the downstream catchment during a typhoon event in 1994. Less than six months after completion, the three dams were covered by debris material (mud, gravel and rocks) after torrential floods and debris avalanches in the catchment. Photographs taken after the flood showed only the tips of the crests of the dams emerging from the newly formed channel bed.

36. Discussions with experts suggest that the debris control system was ill-designed. A completely new system is to be built.

Analysis of past accidents and failures

37. The safe design of a stepped spillway must provide adequate discharge facilities as well as safe operation of the chute. Twenty documented accidents and failures are listed in Table 1. Fourteen cases occurred during overflow. Two dominant modes of failures were observed: basic design errors and failure processes that are specific to the stepped chute design (Table 3).

38. Basic design errors cover improper hydrological assessment of the catchment. Both the Warren Dam spillway and the Narora Weir were overtopped by flood flows significantly larger than the design discharge (Table 1), and this led to the failure of the Narora Weir. Foundation failure is another basic design error. The complete failures of the Puentes Dam, St Francis Dam and Bonshaw Weir could have been avoided with sound geological and geotechnical studies prior to construction. Jansen⁸ commented on the St Francis Dam site: 'in view of the many deficiencies of the site, the

Table 3. Analysis of accident and failure cases

Mode of failure	Description	Spillway/weir
Design errors	Improper hydrological assessment Foundation failure and stability problem	Narora, Warren, debris dams in Taiwan Puentes, St Francis, Bonshaw, Goulburn
Problems specific to stepped chutes	Flow instabilities associated with transitory regime between nappe and skimming flow Maintenance (lack of) Quality of construction	Arizona Canal, Lahontan, New Croton Kobila, Arizona Canal, Hilliard Creek, Binda New Croton, St Francis (cracks); Lahontan (concrete); Bonshaw (timber); Moscovite earth dams

survival of the structure for 2 years is remarkable indeed'.

39. Table 3 highlights three series of failures that are directly related to the stepped chute design, in the author's opinion. Firstly, the flow conditions near the transition between the nappe and skimming flow regimes are characterized by a periodic disappearance of the cavity beneath the free-falling nappes and the appearance of a quasi-homogeneous stream flow. The phenomenon presents some similarity to the cavity filling of ventilated cavities. It is characterized by transitory fluctuations from one regime to another, which might include improper or dangerous flow behaviours. The hydrodynamic instabilities could cause large, fluctuating hydrodynamic loads on the steps and unnecessary vibrations of the structure. In two well-documented cases (Arizona Canal and New Croton), failures occurred during overflow conditions very close to the onset of skimming flow (Fig. 2). In another case (Lahontan), the design flow of the right-hand spillway corresponded to a transition flow. Extensive damage to that chute was experienced to the point that it was considered to pass 'flood flows over the left side only' to reduce 'the cost of repairs'.²⁶ Basically, designers must avoid such transition conditions and consider additional hydraulic and structural tests if they cannot avoid a flow regime transition. If the design flow regime is nappe flow, all the flow conditions up to the design flow conditions will be nappe flow. If the design flow is a skimming flow, the transition flow regime is unavoidable for some flow rate below design discharge. For gated spillways, suitable gate operation must be selected to avoid transitory flow conditions (Fig. 2). For ungated spillways, it is recommended to select a spillway geometry (step height and width) such that the transition occurs at very low flow rates.

40. Secondly, the quality of construction is critical. Larger hydrodynamic loads are experienced on the steps compared with those on a smooth invert. In nappe flow, the impact pressure of the free-falling nappe is at least ten times larger than the corresponding hydrostatic pressure (e.g. Chanson,¹ pp. 204–205). In skimming flow, the averaged bed shear stress is about 30 times greater than in a smooth chute flow, for the same flow rate and bed slope.¹ Several failures may be attributed to poor (or below-standard) quality of work. At the New Croton and St Francis dams, large cracks were observed along the spillway (New Croton³¹) and within the dam masonry below the spillway (St Francis^{33,34}). In the Moscow region, two overflow earth dams failed because of poor-quality drainage filter layers. The failures of Bonshaw Weir and the Lahontan Dam spillway similarly derived from lower construction standards.

41. Thirdly, maintenance is another important issue for safe operation. During overflow, damage might occur and repair works must be regularly conducted. Lack of maintenance may lead to partial or complete failure, for example the Arizona Canal, the Kobil Dam and the Binda Weir. Experience has shown that several stepped structures have behaved satisfactorily for over a century with adequate maintenance; for example, the Kobil Dam operated for 362 years before failure, the Pabellon Dam was built in the 1730s and was still in use in the 1950s, and Gold Creek Dam has achieved 110 years of operation of its concrete stepped spillway. In the early 20th century, Schoklitsch³⁵ (p. 913) commented: 'cascades were formerly used a great deal ... [but] cascades have seldom lived up to their expectations, and have seldom justified their high cost'.

Discussion

Operation of old stepped chutes

42. Several old stepped chutes have been in use for over a century. These include masonry chutes,* unlined rock cascades,† the world's first non-reinforced concrete chute‡ and a composite crib-masonry structure,|| all of which have behaved satisfactorily thanks to adequate maintenance. The experience gained from these successful structures highlights one critical issue, that is, refurbishment. New hydrological data and risk assessment methods have led to an increase in the discharge capacity of existing stepped spillways. Chanson and Whitmore³⁵ argued that the discharge capacity is often increased by a new spillway intake (e.g. the Pas-du-Riot, Gold Creek and La Tâche dams). In such a case the designer must check the new flow conditions on the spillway, the type of flow regime and the magnitude of the hydrodynamic forces to ensure that the original stepped chute can pass the new maximum discharge without damage. The larger residual flow energy associated with the new maximum discharge might induce some damage downstream of the spillway.

Modern designs and errors

43. A spillway structure is designed to last at least 50 years and might be subjected

*For example, the Cantref Reservoir (UK, 1892), Goulburn Weir (Victoria, 1891) and New Croton Dam (USA, 1905).

†For example, the Ternay and La Tâche dams (France, 1867 and 1891).

‡Gold Creek Dam spillway (Australia, 1890).³⁶

||The Malsbury Dam (1870), the first large stepped spillway in Australia.²

to exceptional events. How would stepped spillways behave under such extreme conditions? Past experience and accidents may be used to improve current designs. The proper operation of a hydraulic structure results from a correct design associated with satisfactory and safe operation. The author's experience suggests, however, that some engineers do not have a sound understanding of stepped-channel hydraulics. The New Victoria Dam (Australia), completed in 1991, is a 72 m high RCC dam equipped with a stepped spillway ($Q_{des} = 702 \text{ m}^3/\text{s}$, $W = 130 \text{ m}$, $h = 0.6 \text{ m}$). The spillway design includes an unnecessary break in slope, which has increased the risks of jet deflection at the transition between crest and chute, as at Bucca Weir. In fact, the design was duplicated from the Upper Stillwater Dam spillway, which was supposed to have a 4.57 m wide crest followed by a straight 59° chute (Fig. 7). The final Upper Stillwater design has a 9.14 m wide crest followed by a 72° chute and then a 59° chute: 'The top width was then increased to 30 feet [9.14 m] to facilitate construction and the slope of the upper portion of the downstream face was increased to 0.32:1 [72°] so that it intersected the 0.60:1 slope [59°] at elevation 8100.0'.³⁷ The crest was widened to allow truck access and the change in design had no hydrodynamic justification. Errors were also made during recent calculations for the Gold Creek Dam spillway (Australia, 1890). During a hydrological reassessment of the reservoir, a consultant calculated a maximum discharge capacity of about $400 \text{ m}^3/\text{s}$. A recent investigation showed, however, that overtopping and associated embankment erosion would occur with spills exceeding $280 \text{ m}^3/\text{s}$.³⁶

44. Design engineers must, further, be reminded that a stepped chute will operate in a nappe flow regime at low to medium flows although it is designed for a skimming flow regime at maximum flow capacity. 'The main hydraulic features of stepped channel flows are the two different flow regimes'; in each regime, 'the step faces are subjected to mean pressures and pressure fluctuations quite different' (Chanson,¹ p. 206 and p. 197, respectively). The spillway and stilling basin must operate safely and efficiently for a wide range of flow rates. Physical modelling may be used during design stages to optimize the structure and to ensure a safe operation. Note that, at Lahontan, laboratory tests were performed only afterwards to assess the causes of failure.

45. The above mistakes and past accidents suggest that the design and operation of stepped chutes could be better addressed by some professionals.

Conclusion

46. The study has reviewed over 20 acci-

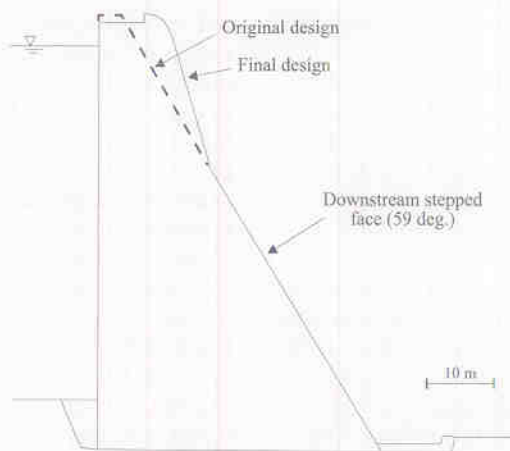


Fig. 7. Cross-section of the Upper Stillwater Dam spillway

dents and failures involving stepped spillway structures. Some failures derived from basic design errors but others are related to problems specific to stepped chutes. Although the paper does not provide explicit guidelines for stepped-spillway design, the study highlights three types of failures that are specific to stepped chutes. These are: flow instabilities associated with the transition between nappe and skimming flow, the lack of maintenance and low quality of construction.

47. The transition between nappe and skimming flow must be avoided. It is characterized by hydrodynamic instabilities causing additional fluctuating loads on the structures, and the resulting fatigue might increase the rate of failure. Fig. 2 illustrates that the transition regime is related to the flow rate (or critical depth), step height and chute slope. If further provides some indication of the range of flow conditions for which transitory flows might occur.

48. In comparison with smooth chutes, stepped spillways require higher standards of construction. The steps are subjected to greater hydrodynamic loads than a smooth invert. Regular maintenance of the chute is also essential because damage to the steps might occur more often than on conventional smooth chutes.

49. Lessons may be learned from both successful designs and failures. The author hopes that further experience of stepped-spillway operation could be shared. He includes a questionnaire (Appendix 1) and he proposes to gather together any additional relevant case studies.

Appendix 1. Questionnaire

50. Please address any relevant information to the author: Dr H. Chanson, Department of Civil Engineering, The University of Queensland, Brisbane QLD 4072, Australia.
Fax: (61 7) 33 65 45 99.
Email: h.chanson@mailbox.uq.edu.au.

Project name	
Reservoir	
Name	
Catchment area: km ²	
Capacity: m ³	
Purpose	
Dam	
Name	
Type	
Construction material	
Dam height/length: m/m	
Further details	
Spillway	
Name (if different)	
Probable maximum flood: m ³ /s	
Design discharge: m ³ /s	
Type	
Intake details Width, type	
Chute details Width, slope, step height	
Energy dissipator Type, width, length	
Failure/accident	
Date	
Flow rate: m ³ /s Maximum duration	
Rainfall/hydrological event	
Accident/failures	
Other information	

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Please email, fax or post your discussion contributions to the secretary:

email: wilson_l@ice.org.uk; fax: +44 (0)20 7799 1325; or post to Lesley Wilson, Journals Department, Institution of Civil Engineers, 1–7 Great George Street, London SW1P 3AA.