Low-Head Hydraulic Structures in Irrigation and Drainage Engineering: Challenging Operation and Design Implications

Hubert Chanson[1]

Abstract: Irrigation and drainage engineering encompasses the human-made supply of water as well as the artificial drainage of excess water. A basic feature of many historical and modern irrigation and drainage systems has been the integrated use of hydraulic structures, most often low-head structures. These structures play a key role in water storage, conveyance, flow control and measurement, and energy dissipation. Yet, most systems are often designed assuming relatively simplistic design flow conditions. In this contribution, a number of relevant key challenges for hydraulic structures used in irrigation and drainage systems are discussed, using the operation of minimum energy loss weirs, the nonlinear behaviors of circular-crested weirs and the instabilities in fish-friendly box culverts equipped with sidewall baffles as examples. Altogether, the design approach of many hydraulic structures needs a rethink, far beyond the naive optimization for simplistic design flow conditions, with a greater focus on the safe and efficient operation across a broad range of less-than-design discharges, to be embedded in the design optimization approach. **DOI: 10.1061/JIDEDH.IRENG-10288.** *© 2024 American Society of Civil Engineers.*

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Introduction

Water is an absolute necessity for life on planet Earth. Since Antiquity, the development of civilizations has been closely linked to the availability of water resources for both drinking and food supplies (Wikander 2000). In Mesopotamia and Egypt, major irrigation systems were developed by deriving large river waters, and other societies collected rainwater, e.g., in Greece and North Africa (Smith 1971; Oleron 2000). Spectacular irrigation systems included the Moeris Reservoir system in the Fayum depression (Hathaway 1958), the pre-Inca development of the Inca estate of Tipon (Wright et al. 2006), and the Moche River irrigation systems in coastal Peru (Farrington and Park 1978).

Irrigation and drainage engineering encompasses the human-made supply of water to and the artificial drainage of excess water from land. It provides an effective means of water regime regulation of soil–water interactions (Blaskó 2014). Agriculture involves the cultivation of plants and the herding of animals and requires reliable water supply, which may be delivered by controlled irrigation. Equally important, water supply is intrinsically essential to cities. Large volumes of water are delivered to cities to meet humans needs, including excess water to flush out sewage; meanwhile, floodwaters must be diverted and drained during major rainfall events.

Hydraulic structures play a key role in the development and operation of irrigation and drainage systems (Schuyler 1909; Ministerio de Obras Públicas, Transportes y Medio Ambiente 1993) and a very large portion are low-head structures. The role of these human-made structures encompasses water storage, conveyance, flow control and measurement, and energy dissipation (Bung and Pagliara 2013) (Fig. 1). Fig. 1 presents a simplified schematic of the contribution of hydraulic structures to irrigation and drainage engineering, and interactions between the various roles. Fig. 2 illustrates specific examples of irrigation and drainage structures. To date, most hydraulic structures are designed assuming steady flow conditions, e.g., to optimize the channel conveyance, discharge performances of weir crests, stilling basin efficiencies, and culvert dimensions (Chow 1959; Henderson 1966; Montes 1998; Chanson 2004), although such a key assumption is sometimes questionable (Chanson et al. 2021).

Our planet faces major challenges for water as the population grows, with emerging economies, and as climate change alters the water cycles. How water can be managed in a sustainable and safe manner to fulfil the planet's needs? A key feature of many historical and modern irrigation and drainage systems has been the integrated use of hydraulic structures to store and control water. More recently, their environmental footprint has changed with a shift from engineering-based to nature-based design of hydraulic structures and a thrust toward the restoration of waterway connectivity (Yasuda 2011; Baudoin et al. 2014; Erpicum et al. 2021). This paper discusses a number of relevant key challenges for low-head hydraulic structures used in irrigation and drainage systems, which are rarely discussed despite their relevance to modern sustainable engineering design and which require some innovative and broad-based engineering expertise. After a review of current state-of-the-art practices, three applications are detailed encompassing the operation of minimum energy loss weirs, the nonlinear behaviors of circular-crested weirs, and the instabilities in fish-friendly box culverts equipped with sidewall baffles. The experience from these applications is then discussed.

Hydraulic Structures in Irrigation and Drainage Engineering

Hydraulic structures are commonly integrated in irrigation and drainage engineering. These structures have various purposes, covering conveyance and transportation of water, water storage, flow

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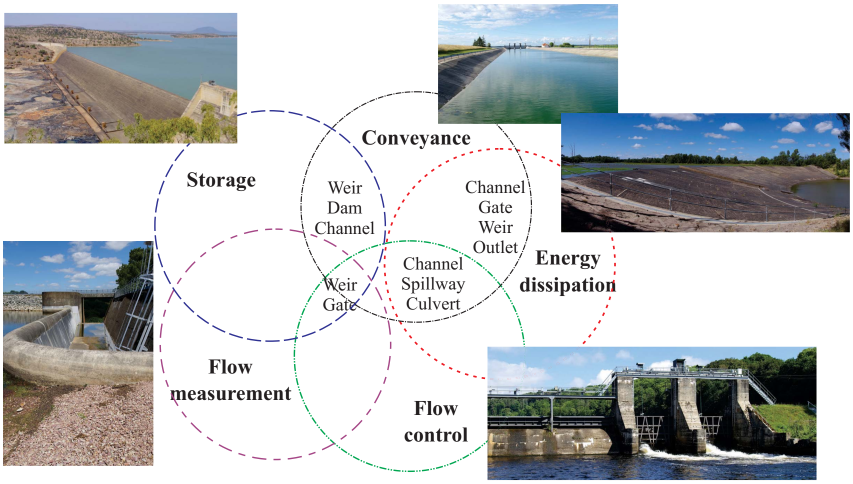


Fig. 1. How hydraulic structures contribute to irrigation and drainage. Clockwise from top left: Burdekin Falls Dam, Mount Wyatt, Australia, on November 16, 2019; drainage canal system of Seine Morge, Lusigny-sur-Barse, France, on June 7, 2022; Chinchilla minimum energy loss weir, Chinchilla, Australia, on April 6, 2018; control gates next to Nantes to Brest navigation canal at Mur de Bretagne, France, on June 11, 2022; and circular crested side spillway at Chèze Dam, Saint Thurial, France, on June 11, 2022. (Images by Hubert Chanson.)

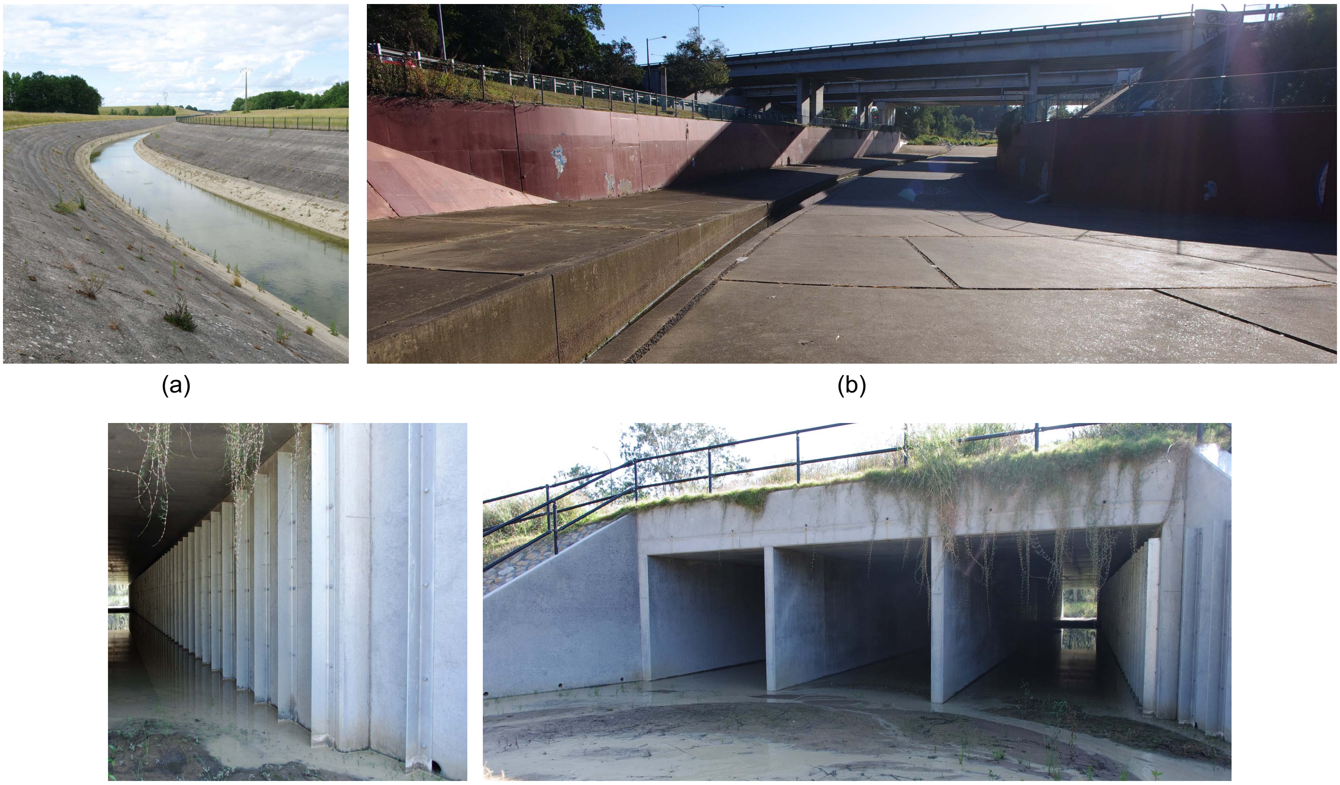


Fig. 2. Irrigation and drainage hydraulic structures: (a) human-made outflow channel below Digue de Brévonnes, Brévonnes, France, on June 7, 2022, looking downstream, design capacity $= 135\ \mathrm{m^3/s}$; (b) minimum energy loss waterway on Norman Creek, Greenslope, Australia, on June 6, 2020, looking downstream, design discharge $= 220\ \mathrm{m^3/s}$; and (c) multicell box culvert at Flagstone, Australia, on August 14, 2020, where the culvert's right outer wall is equipped with full-height baffles to assist fish passage. Inset: details of sidewall baffle installation. (Images by Hubert Chanson.)

measurement and control, and energy dissipation (Fig. 1). Several designs may be multiple purposes, e.g., a weir structure holds an upstream reservoir (storage), its crest may be used for flow control and measurement, and its spillway would be equipped with a stilling basin (energy dissipation). A vast majority of irrigation and drainage hydraulic structures are low-head structures, e.g., culverts, canals, sills, weirs, gates, and outlets. A few examples are illustrated in Fig. 2.

The design of these hydraulic structures may be based upon physical, theoretical, and numerical modeling. For relatively simple designs, design handbooks are used, with tables and charts developed and based upon robust and extensive modeling and some prototype experience (Herr and Bossy 1965; USBR 1965; Schall et al. 2012). In the case of common irrigation and drainage hydraulic structures, standardized designs are available. For road crossings and culverts, road and transportation authorities as well as manufacturers provide simple design guidelines, tables and nomographs (CPAA 1991, 2012; TAC 2004; Schall et al. 2012; QUDM 2016). A number of standard designs of hydraulic jump stilling basins and impact dissipators are available (Peterka 1965; USBR 1965) and are commonly used by industry practitioners. The design of measurement weirs and the rating curves of spillway crests has also been extensively documented (Bos 1976; Bos et al. 1991; Miller 1994) and integrated into international standards, including British Standards (BS) and ISO.

More specifically, the preliminary hydraulic design of weirs and small dams is based upon design guidelines covering the spillway crest, chute, and stilling basin (USBR 1987; USACE 1995). The design recommendations were developed using a range of physical experiments (e.g., Rehbock 1929; Blaisdell 1949; Bradley 1952, 1954; Bradley and Peterka 1957a, b, c, d, e, f; Frizell and Svoboda 2012). The laboratory experiments primarily included observations of water discharges and water surface elevations, with some limited total head and invert pressure data, always collected under established steady flow conditions. Detailed velocity and turbulent intensity data were not covered, and neither were transient flow conditions. Focusing on one type of spillway crest, the circular weir, the design was initially developed during the nineteenth century. It is a simple and efficient shape that can be easily constructed. The circular crest design was tested in laboratory, using a combination of water discharge, water elevation, and invert pressure measurements, with relatively low head above the crest. A few seminal studies include those of Bazin (1898), Rehbock (1929), Fawer (1937), and Escande and Sananes (1959). All the observations were undertaken under steady flow conditions, without consideration of any form of flow transients.

The hydraulic design of standard culverts may be undertaken using design charts and nomographs, as well as software packages (Bossy 1961; Herr and Bossy 1965; Chanson 2004), which rely upon extensive series of physical experiments conducted prior to 1960 (e.g., Yarnell et al. 1926; Larson and Morris 1948; Blaisdell and Donnelly 1956), with the results embedded in tables and graphs. The experimental data included the water discharge, the free-surface profile, and sometimes total head and piezometric head data, with a focus on the design flow conditions (Metzler and Rouse 1959). The hydraulic modeling did not cover much any less-than-design flow conditions and environmental flows conducive of upstream fish passage, nor the impact of road crossing on the access of fish to feeding and breeding habitats and on the conservation of threatened species.

Hydraulic model tests of gates primarily rely upon measurements of discharge, water depths, and pressures under steady conditions, and sometimes hydrodynamic forces. The water depths are typically recorded with pointer gauges and pressures with manometer tubes (Erbisti 2014). Physical data may be complemented by theoretical solutions for simple boundary problems (e.g., Rouse 1938; Henderson 1966; Montes 1997). Physical observations under unsteady transient conditions are rare, with a few exceptions (e.g., Petrikat 1958, 1978; Sun et al. 2016). A senior engineer, Jack Lewin, further argued that "gates are designed for extreme events," while acknowledging, at the same time, that "personal experience of their performance under these conditions is limited" (Lewin 1995, p. 1).

For human-made channels, design handbooks are available (USACE 1991) and add to the extensive literature in open channel hydraulics (Chow 1959; Montes 1998; Chanson 2004).

Altogether, a vast majority of design handbooks and standardized design manuals relevant to irrigation and drainage hydraulic structures are focused on the optimum characteristics for the design flow conditions, and the calculations were developed for steady flow conditions. Unsteady flow conditions and transient flow conditions are not covered. Further, many handbooks, software packages, and user manuals tend to simplify the calculations for usage by civil engineers with limited fluid mechanics expertise (Cunge 2014). These handbooks and software are sometimes seen as black boxes with the exclusion of practical considerations: "how do you know the results are correct?" (C. J. Apelt, personal communication, 2023); "it looks good, but is it correct?" (Knight 2013, p. 14). Emeritus Professor Knight acutely commented, "running software is not simply akin to holding a driving licence" (Knight 2014, p. 138). The same should be said of the usage of design handbooks.

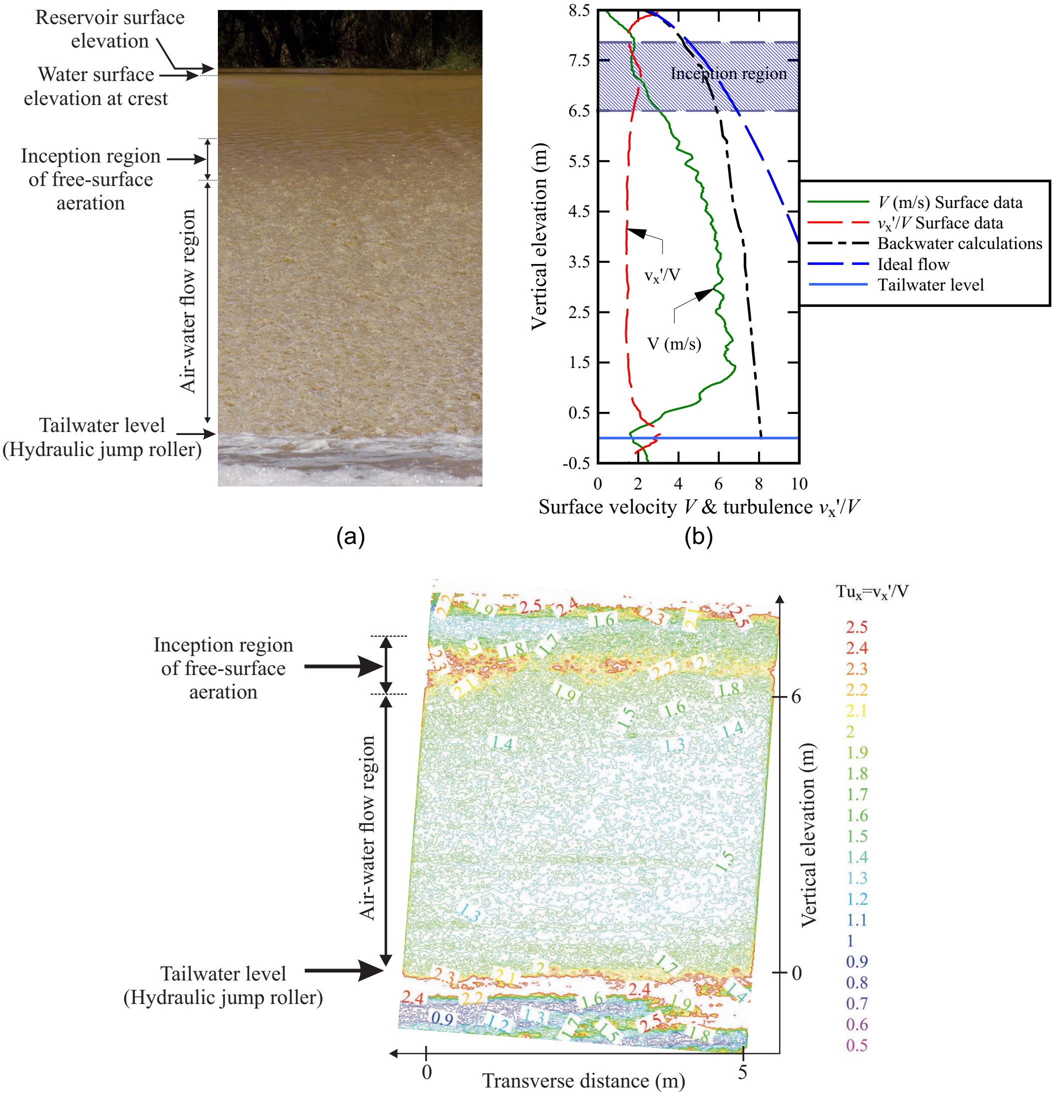
Design Concept and Operation of Minimum Energy Loss Weir

The minimum energy loss (MEL) weir design is an embankment overtopping weir structure with a smooth converging spillway chute, which was developed for river catchments affected by heavy rainfalls with very flat river bed and erodible banks (McKay 1971; Apelt and Chanson 2022). Compared with traditional weirs (e.g., USBR 1965), the MEL concept aims to minimize the total head losses across the structure with a streamlined geometry everywhere (i.e., inflow, crest, and chute) and a gradual expansion downstream (Apelt 2002). The design allows for maximum in-stream storage, some protection against river bank scour at the weir abutments and the downstream river course, and little increase in the frequency of out-of-bank flooding. The first MEL weir was the Sandy Creek weir (Clermont, Australia, 1963) (McKay 1971). The largest MEL weir (Chinchilla, Australia, 1973) is listed as a large dam (ICOLD 1984). A related design is the ungated MEL spillway inlet design, e.g., at Lake Kurwongbah (Sideling Creek Dam, Brisbane, Australia) (McKay1971; Chanson 2003). Fig. 3 illustrates the Chinchilla MEL weir in Australia.

The basic design principles of a MEL weir consist of a long crest to pass the bank-full flow at critical conditions without any change in the upstream water elevation, a weir crest plan view with a suitably long circular arc shape, concave downstream to converge the streamlines horizontally toward the naturel stream channel downstream of the crest, and relatively flat upstream and downstream weir slopes to prevent rapid lateral convergence and divergence in order to reduce energy losses. Fig. 3 shows an overflow discharge smaller than the design flow.

The Chinchilla MEL weir was carefully documented during a number of flood events, and detailed field measurements were recorded during one event (Chanson and Apelt 2023). Visual observations showed the smooth approach flow leading to the weir's broad crest, with a combination of aerial photography and movies, complemented by onsite inspections between 1997 and 2024 (including during overflow events in November 1997, February and December 2020, November and December 2021, April 2022, and February 2024). The approach flow conditions emphasize the quiet inflow, even during major floods, leading to smooth critical flow conditions above the weir crest with minimum energy loss. The flow is accelerated along the downstream chute with some flow concentration. Despite the smooth inflow conditions, surface velocity measurements showed the occurrence of streets of high-velocity

Fig. 3. Operation of the Chinchilla minimum energy loss weir on December 15, 2021. Head above crest = 0.55 m and $Q = 144\ \mathrm{m^3/s}$. (Images by Hubert Chanson.)



(a) (b)

(c)

Fig. 4. Chinchilla MEL weir spillway chute observations: (a) inception of self-aeration down the converging spillway chute on December 15, 2021, with flow direction from top to bottom, head above crest = 0.55 m, and $Q = 144\ \mathrm{m^3/s}$; (b) vertical distribution of transverse-averaged streamwise surface velocity and surface turbulence v_x'/V on November 27, 2021, and comparison with ideal fluid velocity and one-dimensional computations (backwater calculations), with head above crest = 0.49 m and $Q = 121\ \mathrm{m^3/s}$; and (c) contour map of streamwise surface turbulence v_x'/V on November 27, 2021, with head above crest = 0.49 m and $Q = 121\ \mathrm{m^3/s}$. (Image by Hubert Chanson.)

and low-velocity regions at a given elevation, as well as high surface turbulence levels (Chanson and Apelt 2023) (Fig. 4).

Fig. 4 presents field observations at the Chinchilla MEL weir based upon optical measurements, including vertical distributions of transverse-averaged longitudinal surface velocity and surface turbulence [Fig. 4(b)], and a contour map of longitudinal turbulence levels [Fig. 4(c)]. Similar to prototype observations on the Hinze Dam stepped spillway, the surface velocity data imply some local concentration of kinetic energy and of turbulent kinetic energy at the chute toe, which would require some additional safety factor in the stilling basin design.

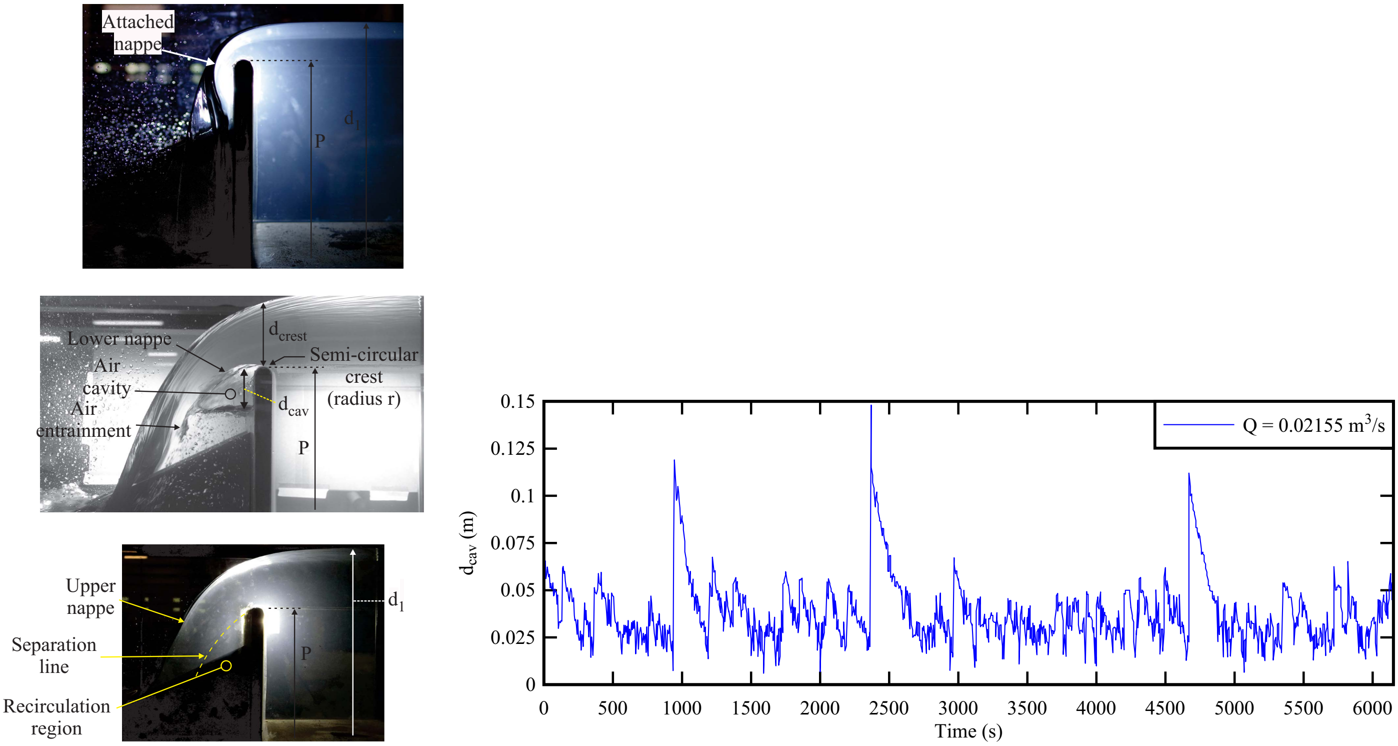
At the weir crest, the water surface is smooth and glassy. Downstream, the water is accelerated along the smooth converging chute, and the glassy water surface gradually becomes grainy and rough, until self-aeration takes place, i.e., the inception region [Fig. 4(a)]. The region of free-surface aeration inception marks a gradual transition of water surface roughness toward a very rough and choppy surface with entrained air. Further downstream, the self-aerated flow impinges into the river and induces a hydraulic jump. For large discharges and high tailwater levels, the hydraulic jump takes place before the inception region. The hydraulic jump is characterized by large amounts of entrapped air at the roller toe and uncontrolled surface aeration and detrainment across the roller surface. Visual, photographic, and cinematographic observations indicated the presence of large-scale three-dimensional vortical structures in the hydraulic jump roller, with dimensions comparable to the roller height, i.e., the difference in conjugate depths. To date, no unsatisfactory operation or performance in terms of energy dissipation has been reported for a period of over 40 years.

Finally, at most in-river weirs, including the Chinchilla weir, the visual observations often highlighted the light-brown color of the water surface, indicating a three-phase mixture of water, air, and sediments. A complete solution of the three-phase flow field requires three sets of fluid motion equations, one for each phase, together with coupling of equations at the various phase interfaces. Their correct implementation is not trivial and rarely considered during the design stages.

Nonlinear Rating Curves of Circular Weir

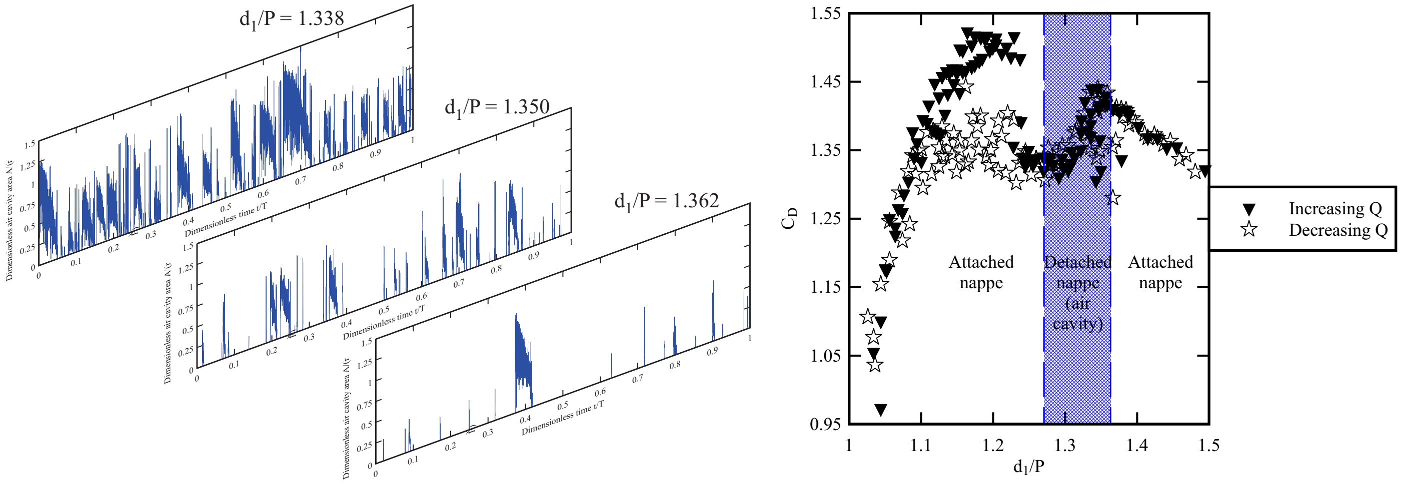
Waters passing over rounded crest weirs experience a rapidly-accelerated flow region near the crest. The circular-crested weir design was developed during the nineteenth century to improve the discharge capacity for a given head above crest, compared with thick-crested weirs (Bazin 1898). The simplicity in shape, design and construction is well suited to lateral spillways (Fig. 1, bottom left). Recent experiments with unventilated semicircular weirs showed nonlinear rating curves linked to flow hysteresis and associated with very-low-frequency nappe instabilities (Tullis et al. 2019; Chanson 2020), although it is acknowledged that the associated instabilities are affected by scale effects (Petrikat 1978; Lodomez et al. 2019).

With unventilated semicircular weirs, the overflow may be attached to the downstream wall at low discharges, detached for a range of intermediate heads with an air cavity forming beneath the lower nappe, or reattached at large upstream heads with the disappearance of the air cavity [Fig. 5(a)]. The transition from attached to detached nappe, and from detached to attached nappe, is seen to be a function of the ratio d_1/P, where d_1 is the upstream water depth and P is the weir height (Tullis et al. 2019; Chanson 2020; Chanson and Memory 2022). Noteworthy, the changes between attached and detached nappe regimes are characterized by some instabilities, including changes in upstream and downstream flow properties, and sometimes loud noise. Fig. 5(b) illustrates some long-duration observations in laboratory of the air cavity height for flow conditions close to these transitory regimes. During the 102-min–long record, three major nappe detachments are seen with a sudden expansion of the air cavity size [Fig. 5(b)].



(a) (b)

Fig. 5. Overflow above an unventilated semicircular weir: (a) side views, with increasing flow rate from top to bottom and flow direction from right to left; and (b) long duration observations of fluctuating cavity height d_{cav} with a partially open air cavity for $Q = 0.02155$ m^3/s, $d_1/P = 1.267$, $r = 0.010$ m, $P = 0.250$ m, and $B = 0.40$ m, with partially open air cavity. Data recorded every 5 s for 6,130 s. (Images by Hubert Chanson.)



(a)

(b)

Fig. 6. Nonlinear flow characteristics above an unventilated semicircular weir with $r = 0.010$ m, $P = 0.250$ m, and $B = 0.40$ m: (a) dimensionless time variations of air cavity volume $A/(r \times P)$ for $Q = 0.0245$, 0.0257, and 0.0270 m^3/s and data sampling of 25 frames per second (fps) for $T = 1{,}800$ s; and (b) rating curves of circular-crested weir, with dimensionless discharge coefficient C_D as a function of the dimensionless upstream depth d_1/P. Data were recorded with increasing and decreasing discharges as shown. Dashed region indicates detached nappe with air cavity ($r = 0.010$ m, $P = 0.250$ m, and $B = 0.40$ m).

Physically, the changes between flow patterns take place across a breadth of discharges, as illustrated in Fig. 6. Fig. 6(a) shows the variations with time of the dimensionless air cavity volume during 30 min for three different discharges. For these data, a zero cavity volume means an attached nappe. During such long observation data sets, the nonlinear behavior of unventilated semicircular weir is observed in the form of cyclic pattern between nappe reattachment and nappe detachment. A key physical observation is the marked change in streamline curvature at the crest. An attached nappe is associated with a strong streamline curvature and a smaller upstream water depth with a larger discharge coefficient, consistent with ideal flow theory (Streeter 1948; Chanson 2014). Both visual and quantitative data showed a relatively slow air cavity filling process, spanning over several minutes in laboratory, whereas the cavity opening is very abrupt and violent, sometimes linked to some a loud bang. These features are seen in Figs. 5(b) and 6(a). In both figures, the data show fast fluctuations superimposed to long-period major instabilities. For completeness, nappe instabilities and oscillations may also occur without enclosed air pocket behind the free-falling nappe (Petrikat 1978).

The nappe instabilities typically interact with aeroelastic oscillations in the air cavity and may lead to resonances (Rockwell and Naudascher 1978). Such interactions are undesirable and hazardous. Typical features and behaviors encompass the transverse banding on the water jet, vibrations to the weir structure, and low-frequency acoustic energy release, as well as downstream surge waves associated with changes in discharges. A related effect is the large scatter of the weir rating curve, illustrated in Fig. 6(b). Fig. 6(b) presents some dimensionless discharge coefficient C_D data, defined as follows:

$$C_D = \frac{q}{\sqrt{g} \times \left(\frac{2}{3} \times (H_1 - P)\right)^{3/2}} \tag{1}$$

where q = unit discharge; g = gravity acceleration; and H_1 = upstream total head. For this data set, the mean changes in flow patterns are shown with vertical lines and dashing. In Fig. 6(b), the rating curve data show a marked hysteresis, with different curves for increasing and decreasing discharges. More, the dimensionless discharge coefficient data present a broad scatter, in line with earlier data (Tullis et al. 2019). It is believed that the scatter derives from a combination of nonlinear instabilities observed during long-duration experiments, differences between short and long duration records, and laboratory-specific conditions (Tullis et al. 2019; Chanson 2020).

The implications are broad in terms of design and modeling. For example, a recent study recommended long-duration investigations with a 30-min flow establishment followed by physical laboratory observations "for a minimum of 30 min and [. . . .] repeated over several hours" (Chanson and Memory 2022). Although the advice would apply to both physical and numerical studies, in practice, the nappe instabilities are typically not reproduced numerically with commercial software and CFD packages using desktop workstations. Despite their importance, e.g., because of violent fluid–structure interactions, these nonlinearities and transient flow conditions are rarely well-understood by asset owners and engineers, including numerical and experimental modelers.

Fish-Friendly Box Culvert Design: Discharge Capacity and Free-Surface Instabilities

Fish migration is commonly seen in rivers and streams as a natural phenomenon. But, human-made road crossings (e.g., bridges and culverts) act as channel constriction, adversely impacting on the fauna, flora, and fish species diversity (Warren and Pardew 1998). The manner in which road crossings block fish movement is closely linked to the targeted fish species (Sagnes and Statzner 2009; Gigleux and de Billy 2013; Januchowski-Hartley et al. 2014). Most small-bodied fish species and juveniles of larger fish have limited swimming capabilities. With such weak swimmers, the high velocities in the culvert barrel are often a major hinderance for upstream fish migration (Larinier and Chorda 1995; Hurst et al. 2007). Fish response to turbulence is complicated despite being very relevant to fish-friendly culvert design. Recent findings indicate that the small fish swim preferentially in regions of high turbulence, intense secondary motion, and low velocity (Goettel et al. 2015; Chanson 2019). That is, the fish shelter from high velocities to traverse the culvert

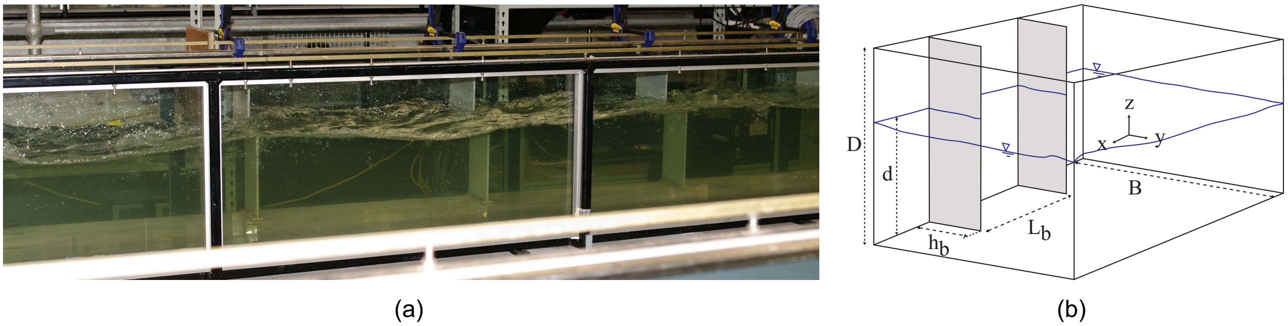


Fig. 7. Standard box culvert barrel equipped with sidewall baffles: (a) free-surface instabilities with $Q = 0.092\ \mathrm{m^3/s}$, $B = 0.5$ m, $h_b = 0.167$ m, and $L_b = 0.667$ m, with flow direction from right to left; and (b) definition sketch. (Image by Hubert Chanson.)

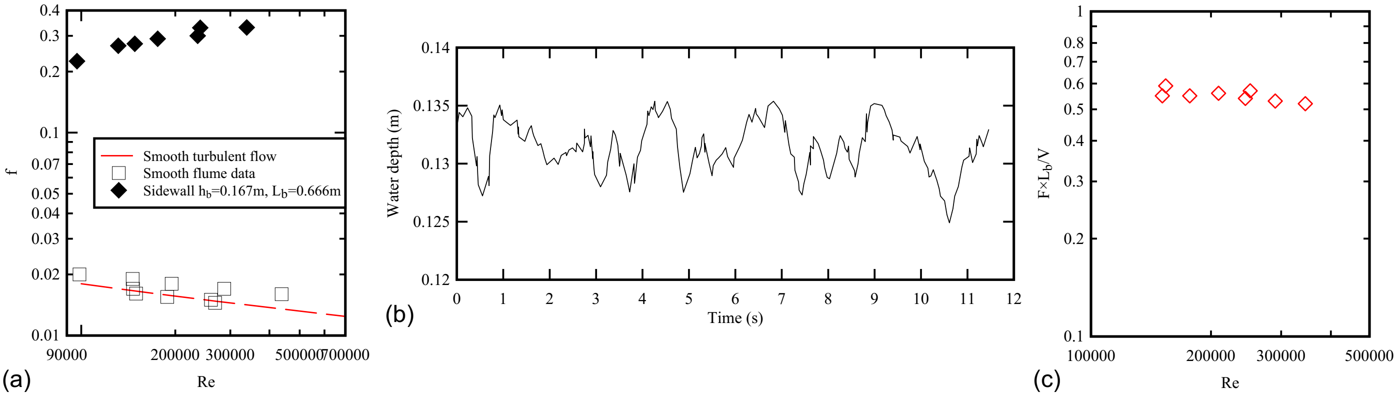


Fig. 8. Flow properties of standard 12-m-long by 0.5-m-wide box culvert barrel equipped with asymmetrical sidewall baffles: (a) flow resistance and comparison between smooth box culvert barrel and culvert barrel equipped with asymmetrical sidewall baffles (data from Cabonce et al. 2019; Leng and Chanson 2020; Hu et al. 2022); (b) time variation of water elevation ($h_b = 0.167$ m and $L_b = 0.667$ m) for $q = 0.0578\ \mathrm{m^2/s}$; and (c) Strouhal number of dominant free-surface oscillation ($h_b = 0.167$ m and $L_b = 0.667$ m). (Data from Hu et al. 2022.)

barrel actively using regions of strong turbulent vortices and secondary motion (Cabonce et al. 2018). Boundary roughening and baffles are among the common solutions to create low-velocity zones conducive to upstream fish passage, albeit with a cost in terms of design, construction, discharge capacity, and maintenance. One design is the full-height sidewall baffles [Fig. 2(c)], although the very few field trials were limited to very low flows, which are not representative of real flood events (Marsden 2015; Leng and Chanson 2020).

A detailed physical modeling was conducted in a near-full-scale facility across a broad range of flow rates corresponding to less-than-design discharges (Leng and Chanson 2020; Hu et al. 2022) (Fig. 7). The results showed a number of hydrodynamic features that are keys to a successful fish-friendly culvert design. A large low-velocity zone (LVZ) develops in the wake of each baffle. This three-dimensional volume provides some resting area for fish, with a strong recirculation, high turbulence levels, and secondary currents across a wide range of discharges. Thinking like a fish, the main challenge is to negotiate the baffles, typically next to the channel bed, to shelter in an intense secondary current region.

In terms of hydrodynamics, the installation of sidewall baffles increases the Darcy-Weisbach friction factor by one order of magnitude [Fig. 8(a)], which is considerable. This is illustrated in Fig. 8, showing the friction factor in a 12-m-long, 0.5-m-wide culvert barrel channel, with a comparison between smooth channel and asymmetrical sidewall baffle configurations. The flow resistance is increased due to form drag and flow separation downstream of each baffle, as well as secondary currents and flow asymmetry. The significantly larger flow resistance reduces the hydraulic conveyance of the fish-friendly culvert design. For relatively high water levels, e.g., during flood events, the hydrodynamic instabilities significantly increase with large-amplitude undulations. At such discharges, the strong turbulent shear in the wake of each baffle might prevent the upstream fish progression because small-bodied fish might not be capable to navigate past a baffle. In terms of the Manning resistance coefficient, this increase in flow resistance would represent an increase by a factor five, but the readers are reminded that the use of empirical coefficients in human-made channels is incorrect: "The (Chézy and Manning) equations express our continuing ignorance of turbulent processes" (Liggett 1975, p. 45); "Manning's n has certain limitations" (ASCE 1963, p. 97); "Flow resistance calculations in open channels must be performed in term of the Darcy friction factor" (Chanson 2004, pp. 81–82).

Qualitative and quantitative observations show the occurrence of large free-surface waviness along the baffled channel for a range of discharges [Figs. 7(a) and 8(b)]. Fig. 8(b) presents some instantaneous water elevation record in the sidewall baffle channel, about midway into the barrel. For comparison, the smooth channel flow presented free-surface roughness of ±0.5 mm in the same flume for the same water discharge. With baffles, the free-surface fluctuation range may reach in excess of 20% of water depth and be associated with some wave breaking and air bubble entrapment at the free surface. The predominant frequency F is nearly constant across cavity area and mainstream region for a broad range of discharge and about $F \times L_b/V \sim 0.55$ [Fig. 8(c)].

A culvert is a confined channel designed to operate as an open channel, and a minimum clearance must be provided between the free surface and culvert barrel obvert. Design guidelines for smooth channels and culverts typically recommend a 20% freeboard at design discharge (Chanson 2004; QUDM 2016, sections "Basin Freeboard" and "Open Channel Hydraulics"). With large wave heights, e.g., caused by the baffles, an extra clearance is required because of the hydrodynamic instabilities. The combination of large flow resistance and freeboard requirements yields a drastically lower discharge capacity of the box culvert equipped with sidewall baffles. To achieve the same design discharge capacity, a larger culvert structure would need to be considered into the budget planning of fish-friendly culvert proposals and constructions.

All in all, the near-full-scale testing suggests that the installation of full-height sidewall baffles to assist fish passage might not be straightforward. Practical design and operational considerations must be accounted for. These may include the installation of solid baffle anchors in culverts walls to resist the hydrodynamic loads on baffles, the safety of swift water rescuers adversely impacted by metallic baffles, and the increased debris trapping requiring a regular maintenance program. All these factors must be included in the total costs of the structure.

Discussion

Traditionally, hydraulic structures are designed for a range of operational conditions and optimised for the design flow conditions, typically a design discharge Q_{des} and a maximum acceptable afflux (USBR 1987; Novak et al. 2007). During their lifetime, the vast majority of structures operate across a considerable range of less-than-design discharges, i.e., $Q \ll$ to $<Q_{\mathrm{des}}$, for which the operational conditions are not optimized, and undesirable situations might occur, although perfect performances would be expected, e.g., in terms of flow conveyance, energy dissipation, and maintenance.

Considering an overflow weir spillway, the spillway chute rarely operates at design discharge ($Q = Q_{\mathrm{des}}$). The vast majority of spills correspond to less-than-design discharges for which the spillway chute is not optimized and three-dimensional flow motion is observed, as documented in situ at the Chinchilla weir (discussed previously) and Hinze Dam spillway (Chanson 2022b). In turn, the flow concentrations linked to high-velocity streets create regions of high kinetic energy that must be dissipated safely, despite many design procedures based upon quasi-one-dimensional (1D) flow approximations and upon physical modeling in small-size facilities in which three-dimensional (3D) flow motion is not measurable.

Altogether, the design flow conditions for a spillway are rarely experienced, if not never, in situ. Further, with many projects, the design discharge is often reevaluated during the operational lifetime of the structure, sometimes with massive increases in capacity (Gill et al. 2005; Lemperière et al. 2012; Tullis 2013; Chanson et al. 2021). In plain terms, one must query the relevance of a spillway optimization for an unique design discharge, i.e., Q_{des}. The hydraulic design optimization must be more robust, e.g., by considering a range of flow conditions covering design, less-than-design, and larger-than-design flow rates.

With road crossings, many two-lanes culverts are designed for 1-in-5 to 1-in-20 annual exceedance probability (AEP) flood events. Thus, most culverts operate primarily at less than design flows, e.g., with a number of box culverts operating with $Q/Q_{\mathrm{des}} < 5\%$ for more than 91% of flow records in eastern Australia (Leng et al. 2019, 2020). During such less-than-design discharges, the addition of appurtenances, e.g., baffles, may induce flow instabilities linked to an increased flow turbulence and flow resistance, as well as some drastic reduction in discharge capacity (Olsen and Tullis 2013; Leng and Chanson 2020; Hu et al. 2022). Despite the usefulness of such baffles for fish passage, a detailed knowledge of the predominant instability frequency (as shown previously) is relevant in industrial applications to infer whether a structural frequency responds to some particular frequencies of the flow across a wide range of discharges. The energy associated with the hydrodynamic instability frequency or frequencies may further give a measure of the magnitude of the associated turbulent dissipation in the culvert.

Noteworthy, both culvert and weir operations may experience transient unstable conditions (discussed previously). The associated hydrodynamic instabilities can induce sudden changes in discharge capacity, with the formation of upstream and downstream surge waves, as well as some increased hydrodynamic cyclic loads, e.g., on culvert baffles and vertical weir walls. These might require the implementation of changes in operational procedures, including the avoidance of a range of discharges, the ventilation of weir nappe, or a different design of fish-friendly culvert barrel.

In summary, the hydraulic design of a low-head structure is traditionally based upon simple calculations, optimized for the design flow conditions. With small- to medium-size projects, the design methodology may often be limited to desktop calculation, using one-dimensional calculations and design charts. Previous examples (as discussed in the preceding sections) illustrated the need for a different, more holistic hydraulic design approach, accounting for transient and unstable conditions for less-than-design discharges, the associated hydrodynamic loads, and the concentrations of kinetic energy.

Traditionally, physical and/or numerical computational fluid dynamics (CFD) modeling may be conducted for large or high-head structure projects, although often for a limited amount of flow conditions. The hydraulic modeling is undertaken, physically and numerically, to assist with the design, and the model aims to reproduce the hydrodynamic conditions most important in the prototype. The modeling approach must be developed based upon the fundamental principles of similitude and dimensional analysis (Bertrand 1878; Rouse 1938). In a hydraulic model, the flow conditions are said to be similar to those in the prototype structure when the model displays similarity of form of motion and of forces (Chanson 2004). A perfect similarity requires for all the dimensionless dependant parameters to be identical with those in the hydraulic model and prototype; this is physically impossible unless working at full scale, i.e., field observations. In practice, many hydraulic models are smaller than the prototype and developed based upon a combined Froude and Morton similitude. Hence, gravity effects are considered predominant, implying that the Froude number is the same in the hydraulic model and at full scale, and the model uses the same fluids (i.e., air and water) as the full-scale structure.

With any form of hydraulic modeling, the upscaling can be challenging. The extrapolation of the model data to full-scale prototype might require some "correction factor [. . .] to be applied" (Elder 1984, p. 0.1–1). When the model, physical or numerical, is too complicated and the testing, including the modifications, is laborious, the lapse highlights some basic technical flow, and the hydraulic modeller could spend unnecessary time solving incorrectly identified problems. Altogether, a detailed hydraulic modeling is not trivial, e.g., when encompassing transient and unstable conditions, and advanced modeling likely requires some composite methodology embedding detailed physical modeling and computational fluid dynamics calculations (Bombardelli 2012; Chanson 2022a), while acknowledging the intrinsic limitations of numerical models validated with incomplete or small-size experiments.

All in all, hydraulic engineering professionals and academics must work together to foster engineering education and research and development (R&D) addressing these broad challenges, using a combination of theoretical, physical, and numerical modeling validated by broad field observations. When using design handbooks, software manuals, and simplified design guidelines, professional engineers should not forget what they learned in fluid mechanics and hydrodynamics, nor blindly follow instructions of handbooks and software manuals, only too often ignoring the fact that these results are nothing else than approximate solutions of fluid dynamics equations. More, multidisciplinary expertise is required for these teams interacting across a range of disciplines.

Conclusion

Hydraulic structures play a key role in irrigation and drainage engineering, and are often low-head structures. The design of these hydraulic structures may be based upon theoretical, physical, and numerical modeling, or a combination of these. For simple structures, the design methodology is focused on some optimization for the design discharge, with calculations developed for steady flow conditions. Design handbooks and simplified design guidelines are often used, in combination with tables, charts, and nomographs. In practice, a vast majority of irrigation and drainage structures operate across a wide range of nondesign discharges, during which undesirable situations might happen, leading to unsatisfactory performances in terms of conveyance, energy dissipation, and even maintenance. The operation of three types of hydraulic structures has been illustrated and shows a number of practical considerations based upon real-world physical experiences. The hydraulic modeling of complicated flow patterns is not trivial, especially when covering transient and unstable conditions, and this might require some composite approach.

Beyond the design and construction, the operations and maintenance of the hydraulic structures must encompass the implementation of proactive short- and long-term strategies (e.g., France et al. 2018). Hydraulic structures operate in a complex water infrastructure network, and each one has a number of functions within the water network in which it stands. Further, the end of a structure's lifetime must be planned carefully ahead, including through replacement, restoration, or renovation. Future developments will encompass a combination of environmentally friendly nature-based designs and technical advancements, together with the retrofitting, replacement, or removal of an aging infrastructure. With a growing population on the planet, reliable and sustainable water management is a necessity to fulfill the planet's needs. An integrated usage of hydraulic structures to store, convey, and control water is essential. This review of a number of hydraulic structures used in irrigation and drainage systems highlighted several key challenges for current and future hydraulic structures. As a direct consequence, the design approach of such structures may need a rethink, beyond a naive optimization for the design flow conditions. Instead, a greater focus on the safe and efficient operation for a broad range of less-than-design discharges must be embedded in the design optimization.

Data Availability Statement

Some or all data, or models that support the findings of this study are available from the corresponding author upon reasonable request.

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