GENERAL FORMULATION OF BEST HYDRAULIC CHANNEL SECTION[a]

Discussion by Prabhata K. Swamee[2]

The paper fills a gap in the existing literature on minimum-area open-channel sections. However, the following statements of the paper require consideration:

1. "There is no solution for the case of y = depth and side slope = z as the two independent variables with b = bottom width as a constant."

For this case considering $z = x_1$; and $y = x_2$, (24) changes to

$$\frac{\partial A}{\partial z}\frac{\partial P}{\partial y} = \frac{\partial A}{\partial y}\frac{\partial P}{\partial z} \quad (49)$$

Using (37) and (38), (49) reduces to

$$yz^2 + bz - y = 0 \quad (50a)$$

Solving (50a) as a quadratic equation in z, one gets

$$z = \frac{1}{2}\left\{-\frac{b}{y} + \left[\left(\frac{b}{y}\right)^2 + 4\right]^{0.5}\right\} \quad (50b)$$

For $b = 0$ (Triangular channel section); and $z = 1$ (right-angled apex angle triangular section), which is true. As b increases, z progressively reduces; and for $b = \infty$, the section becomes rectangular.

2. "When $z = 0$, in the case of the round-bottom triangle, the solution represents the semicircular section, and in the case of the trapezoid, the solution represents rectangular section."

Though the statement is trivial for a trapezoid, it needs attention for a round-bottom triangle. Irrespective of z, the unnumbered equation for the optimality condition of a round-bottom triangle

$$T = 2r\sqrt{1 + z^2} = 2y\sqrt{1 + z^2} \text{ reduces to } r = y \quad (51, 52)$$

Eq. (52) indicates that the flow is confined to the circular part of the section. Thus, hydraulically it is a semicircular section. Since among all types of channel sections, the semicircle has the least flow area, the optimal sections for other shapes have a tendency to come closest to the semicircle. As a circular geometry is available for a round-bottom triangle, the section was converted into a semicircle. Thus, the following changed equations are obtained: $A_r = 0.5\pi y_r^2$; $P = \pi y_r$; and $R_r = 0.5 y_r$. In place of (46), (47), and (48), these changes yield

$$\frac{y_t}{y_r} = \left[\frac{0.5\pi}{2(1 + z^2)^{0.5} - z}\right]^{0.375}; \quad \frac{A_t}{A_r} = \left[\frac{2(1 + z^2)^{0.5} - z}{0.5\pi}\right]^{0.25}; \quad \frac{P_t}{P_r} = \left[\frac{2(1 + z^2)^{0.5} - z}{0.5\pi}\right]^{0.625} \quad (53, 54, 55)$$

Thus, for $z = 1$, A_r is 3.87% less than A_t, and P_r is 9.96% less than P_t. Similarly for $z = \sqrt{3}/3$, A_r is 2.47% less than A_t, and P_r is 6.3% less than P_t. And for $z = 0$, A_r is 6.23% less than A_t, and P_r is 16.3% less than P_t.

BROAD-CRESTED WEIR[a]

Discussion by Hubert Chanson[3]

The discusser congratulates the authors for their authoritative work on broad-crested weirs. For completion, he wishes to add new material on the undular weir flow. Recent experiments

[a]January/February, 1994, Vol. 120, No. 1, by Parviz Monadjemi (Paper 4181).
[2]Prof. of Civ. Engrg., Univ. of Roorkee, Roorkee 247 667, India.

[a]January, 1994, Vol. 120, No. 1, by Willi H. Hager and Markus Schwalt (Paper 4385).
[3]Lect., Dept. of Civ. Engrg., Univ. of Queensland, Brisbane QLD 4072, Australia.

TABLE 3. Summary of Experimental Flow Conditions

Reference (1)	Q (L/s) (2)	F_1 (3)	h_c/b (4)	Comments (5)
(a) Undular Jump on Broad-Crested Weir				
Woodburn (1932)	4.8 to 7.6	0.86 to 1.09	0.096 to 0.131	b = 0.305 m
Tison (1950)	39.6 to 45.9	0.91 to 1.30	0.172 to 0.190	b = 0.5 m
Hager and Schwalt (1994)	3.15 to 8.25	1.19 to 1.45	0.032 to 0.061	b = 0.499 m[a]
(b) Undular Jump in Prismatic Channel				
Binnie and Orkney (1955)	—	1.12 to 1.52	—	—
Iwasa (1955)	—	1.29 to 4.14	—	—
Montes (1979)	—	1.25 to 2	—	b = 0.20 m
Chanson (1993)	7.0	1.08 to 1.79	0.172	b = 0.25 m[b]
Chanson (1993)	10.5	1.10 to 1.77	0.224	b = 0.25 m[b]
Chanson (1993)	20.0	1.05 to 1.49	0.347	b = 0.25 m[b]

[a]Original data provided by first author.
[b]Fully developed upstream flows.

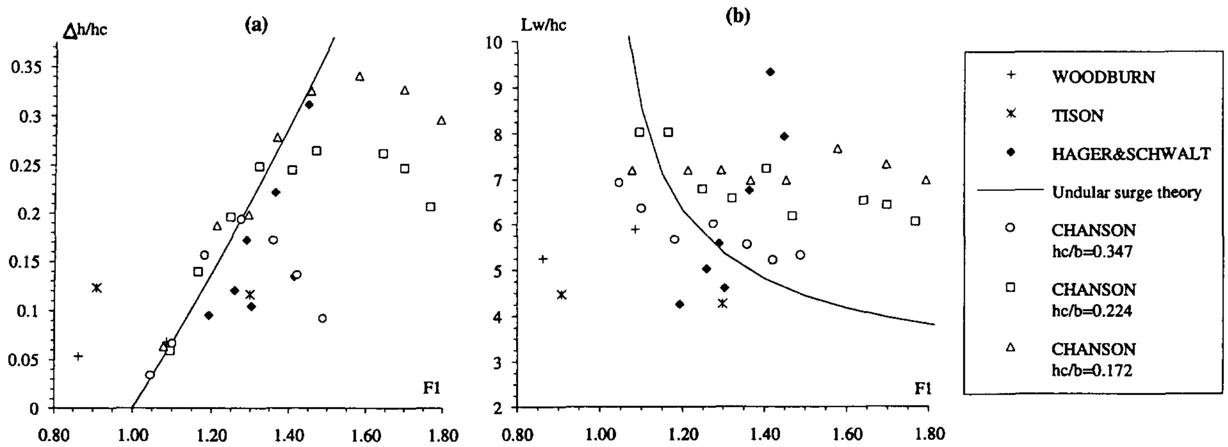

FIG. 11. Free-Surface Undulations Characteristics; Comparison between Undular Weir Flows (Woodburn 1932; Tison 1950; Hager and Schwalt 1994); Undular Hydraulic Jumps (Chanson 1993); and Undular Surge Theory (Andersen 1978): (a) Dimensionless Wave Amplitude $\Delta h/h_c$; (b) Dimensionless Wave Length L_w/h_c

of undular hydraulic jumps (Chanson 1993) provided new information on the wave length and wave-amplitude characteristics downstream of the first wave crest. The discusser will show that undular weir flows (Table 3) have similar flow properties.

The discusser has reanalyzed the characteristics of the free-surface undulations downstream of the first wave crest for undular weir flows and undular hydraulic jumps (Table 3). These data are plotted in Fig. 11, where Δh and L_w are, respectively, the wave amplitude and the wave length of the first wavelength (located between the first and second wave crests). The results are compared with the solution of the Boussinesq equation for undular surge (Andersen 1978; Montes 1979).

Fig. 11 indicates that the undular weir flows have similar free-surface undulation characteristics as undular hydraulic jumps in prismatic channels. Further, the results show that the relationship between the wave amplitude and the approach flow Froude number F_1 exhibits a distinctive shape [Fig. 11(a)].

1. For Froude numbers close to unity, the data follow closely the theoretical solution of the Boussinesq equation (e.g. Andersen 1978). The wave amplitude is nearly proportional to the Froude number (i.e. $\Delta h/h_c = 0.73 \cdot [F_1 - 1]$).

2. With increasing Froude numbers, the wave amplitude data starts diverging from the solution of the motion equation (i.e. without inclusion of the energy dissipation) and reaches a maximum value. This trend is coherent with the data of Binnie and Orkney (1955), Iwasa (1955), Montes (1979) and Chanson (1993).

3. For large Froude numbers, the wave amplitude decreases with increasing Froude numbers and shows a trend similar to Fig. 8(a).

Serre (1953) predicted the apparition of wave breaking for $F_1 = 1.46$, which is close to the authors' result. But with undular hydraulic jumps, the discusser showed that the disappearance of free-surface undulations is a function of the aspect ratio h_c/b and the approach flow conditions (Chanson 1993).

ACKNOWLEDGMENTS

The discusser wishes to thank Dr. W. H. Hager (ETH, Zurich, Switzerland), in providing the original data.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

h_c = critical flow depth (m);
L_w = first wave length (m) defined as the distance between the first and second wave crests; and
Δh = wave amplitude (m) of the first wave length located between the first and second wave crests.

Closure by Willi H. Hager,[4] and Markus Schwalt[5]

The writers would like to thank Chanson for the interesting comments. There is evidently a similarity between the undular flow over a broad-crested weir and the undular hydraulic jump. However, as was observed by the writers, the undular weir flow is much smoother than the undular hydraulic jump, which must be attributed to turbulence effects. Also, the structure of

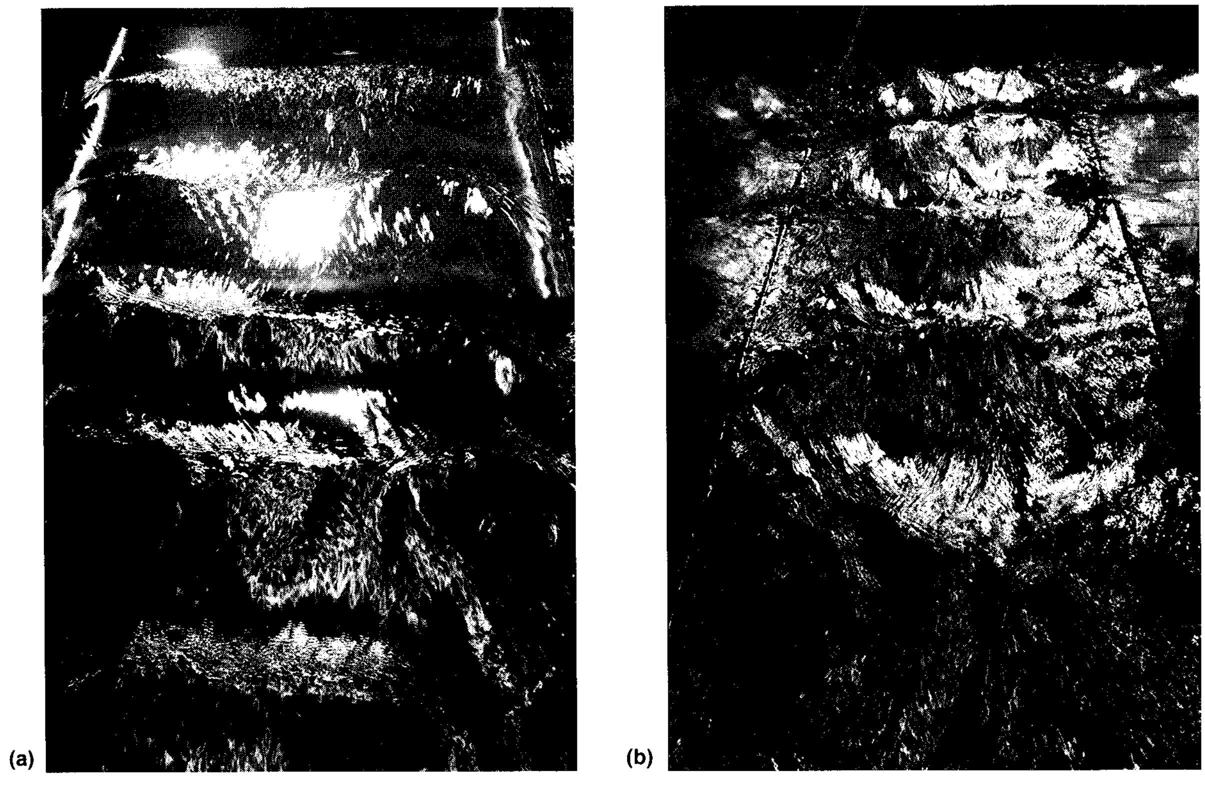
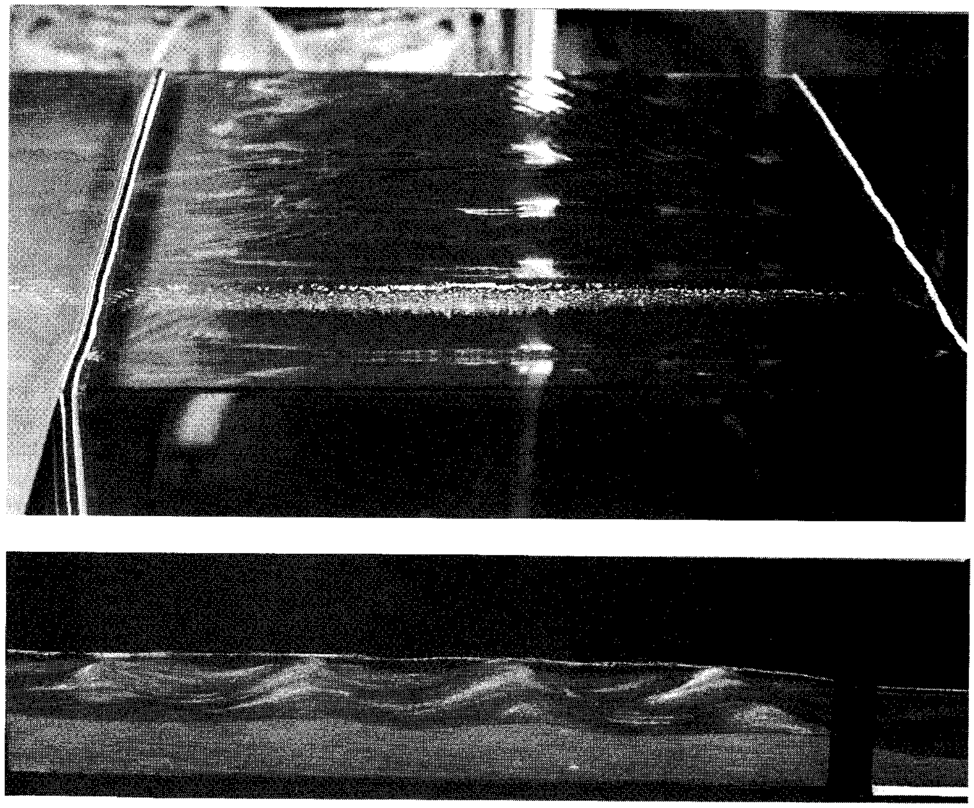


FIG. 12. View from Tailwater to: (a) Undular Weir Flow; (b) Undular Hydraulic Jump

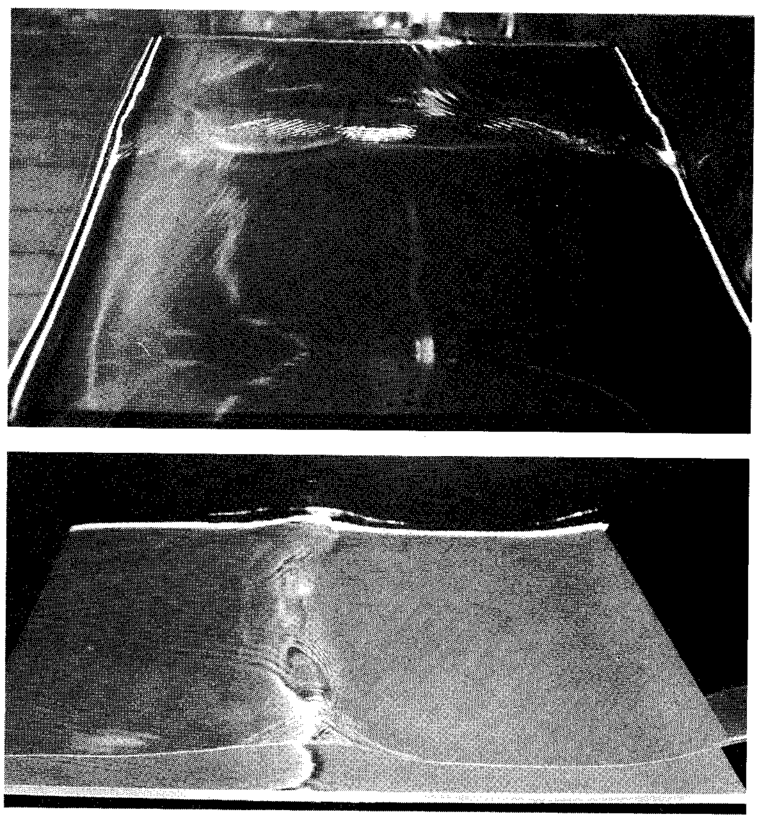
[4]Sr. Res. Engr., VAW, ETH-Zentrum, CH-8092 Zurich, Switzerland.
[5]Project Engrs., c/o Ingenieurbüro S. Sechauser, Eisackstr. 3, I-39100 Bolzano, Italy.



(a)

(b)

FIG. 13. Flow with Four Surface Waves: (a) Undular Weir Flow; (b) Undular Hydraulic Jump



(a)

(b)

FIG. 14. Solitary Wave on Broad-Crested Weir: (a) Upstream View; (b) Side View

undular weir flow is almost two-dimensional, whereas the undular hydraulic jump has a distinct three-dimensional surface pattern, mainly in the form of oblique fronts due to the supercritical approach flow. Fig. 12 shows photographs of both flows from where an axial concentration of waves in the undular jump is clearly visible. Fig. 13 compares the flows with four waves and the distinct difference both in the smoothness and the spatial effect is again illustrated. Both flows have comparable characteristic lengths.

Another flow situation not addressed by the discusser are undular surges. They appeared to the writers always smoother than the standing undular jumps but such secondary effects have not yet been analyzed.

The undular flow on a broad-crested weir may thus be considered as the ideal structure for observation because the flow is almost plane, and steady. The outstanding example is certainly the solitary wave (Fig. 14), which has so far always been associated with a translatory wave. These particular features of undular weir flow may be used both to allow for steady experimental observation, and educational purposes.

Design for Tunnel-Type Sediment Excluder[a]

Discussion by Edmund Atkinson[4]

Predictions of excluder efficiency are vital, especially in view of some poor prototype efficiencies which have been observed (Sahay et al. 1980; Sharma et al. 1972). The paper is therefore to be welcomed.

A more directly applicable measure of efficiency, as used by Sahay et al. (1980), is suggested

$$E = \frac{C_{us} - C_c}{C_{us}} \quad (2)$$

where C_{us} = mean sediment concentration in the river upstream from the intake; and C_c = mean concentration entering the canal. The values for predicted efficiency given in Tables 1, 3 and 4 translate to E = 86%, 96%, and 82%, respectively. Sharma et al. (1972) give measurements of the performance of the prototype excluder at Narora, India, which is predicted in the paper as E = 66%. Data in table 2 of Sharma et al. (1972) indicate that efficiency, E, was only about 4%. (The presence of some silt in the suspended load samples, and the single vertical in the river measurements, both imply that observed efficiency was marginally higher than 4%, but the large discrepancy would remain). The reasonable prototype efficiencies at Narora given in Table 5 appear to have been taken from the analysis reported by Sharma et al. (1972). However, their analysis included only bed load, which the observations showed was an insignificant proportion of the total load.

Sahay et al. (1980) report the efficiency, E, of the prototype excluder at the Eastern Kosi canal as only 15%. The predicted value given in Table 4 translates to E = 82%. The two values are not directly comparable because predicted efficiency was calculated for the derived design, not the actual design. However, predicted efficiency for the actual design may well be even higher, due to a larger excluder discharge (325 m^3/s, rather than 168 m^3/s given in Table 4).

It is likely that the overestimates of efficiency, which appear to have been obtained by the authors, are due to inaccuracies in their methods for predicting concentration distribution, reference concentration, or bed load.

There is, however, much potential in analytical methods of the kind proposed, especially in view of the poor predictions of efficiency that can be obtained from physical models for this particular application.

A three-dimensional numerical model that predicts flow and sediment movement at intakes has been applied to the Narora intake (Atkinson 1989). Efficiency was predicted as 7%, which agrees with the low observed value (4% or marginally higher). The model has since been verified with data collected at other intakes.

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[4]Sr. Sci., Overseas Development Unit, HR Wallingford, Wallingford, Oxfordshire, OX10 8BA, U.K.