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DAM BREAK WAVE WITH ENERGY DISSIPATION : TWO CASE STUDIES

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Abstract: Flood waves resulting from dam break and spillway gate malfunction may have dramatic impacts. Two related cases are the overtopping of a dam by flood waves and a sudden water release down a stepped cascade. In each instance, some energy dissipation takes place and classical dam break theory are inappropriate. Both cases were investigated in large-size facilities and in a prototype. New experimental observations are presented. The results highlight a faster wave propagation than predicted by classical theories. The reduced warning time for downstream populations is a major concern and new correlations are proposed to estimate the flood wave arrival.

Keywords: Dam break wave, energy dissipation, dam overtopping, stepped cascade

1 INTRODUCTION

Flood waves resulting from dam breaks have been responsible for numerous losses. Another type of accident is the overtopping of a masonry dam by a flood wave (e.g. Vajont dam, Italy). The process is often associated with a free-falling nappe, some energy dissipation at the dam toe and the downstream propagation of the surge. The dominant feature of the advancing bore is its high initial momentum resulting from the free-falling nappe. A related case is the flood wave down a stepped chute in which energy dissipation takes place down the cascade. Such a flood wave may result from some gate malfunction or the failure of an upstream reservoir as for the Silverleaf stepped weir which was overtopped for two days when an upstream cofferdam was breached (CHANSON 1998).

In the present paper, the authors investigate flood wave propagations associated with some energy dissipation. Two case studies were investigated in large-size facilities. The results provide new information on the rate of energy dissipation and on the downstream wave celerity. They are compared with “classical” dam break wave results. Practical recommendations are derived.

Basic Theory

Considering an ideal dam break surging over a dry horizontal channel, the method of characteristics may be applied to solve the wave profile as first proposed by RITTER in 1892. The celerity of the wave front equals :

$$C_s = 2 * \sqrt{g * d_o} \tag{1}$$

where d_o is the initial reservoir water depth (e.g. HENDERSON 1966, MONTES 1998). After dam break, the flow depth and discharge at the origin x = 0 are two constants :

$$d_{(x=0)} = \frac{4}{9} * d_o \tag{2}$$

$$Q_{(x=0)} = \frac{8}{27} * d_o * \sqrt{g * d_o} * W \tag{3}$$

where the longitudinal origin (x = 0) is the dam location, the time origin (t = 0) is the instantaneous dam break and W is the channel width.

Bottom friction affects significantly the propagation of the leading tip and WHITHAM (1955) proposed an analytical solution of the wave front celerity that is best correlated by :

$$\frac{C_s}{\sqrt{g*d_o}} = 2 * \left(1.00 + 2.91 * \left(\frac{f}{8} * \sqrt{\frac{g * t^2}{d_o}}\right)^{0.43}\right)^{-1} \tag{4}$$

where f is the Darcy friction factor. WHITHAM (1955) commented that his work was applicable only for $C_s/\sqrt{g*d_o} > 2/3$. ESCANDE et al. (1961) investigated specifically the effects of bottom roughness on dam break wave in a natural valley. They showed that, with a very-rough bottom, the wave celerity could be about 20 to 30% lower than for a smooth bed.

2 EXPERIMENTAL INVESTIGATIONS

A series of flood wave experiments was conducted in a 3.4º stepped chute (24m long, h = 0.143 m) during which a known flow rate was suddenly discharged (Fig. 1, Table 1). The water release was provided by the rapid start of a pump with a variable-speed electronic controller Taian™ T-verter K1-420-M3 discharging into an initially empty pipe feeding the channel intake. (The flow rate was measured with a Dall™ tube flowmeter calibrated in-situ.)

The second series of experiments was performed in a 15m long horizontal channel (painted steel bottom) in which the surge wave was generated by the release of a known water volume through a rectangular orifice (70mm×750 mm) located 1.33-m above the 0.8-m wide channel (Fig. 1). The orifice opening occurred in less than 1/30 sec. (CHANSON et al. 2000a). (The hydrograph was deduced from water level measurements in the reservoir.) Further a sudden discharge release was videotaped at Brushes Clough dam spillway in 1994 (BAKER 1994) and the video footage was analysed by the first author.

Flood waves were studied with video-cameras. In laboratory, a VHS-C camescope National™ CCD AG-30C (30 frames/s, shutter: 1/60 & 1/1,000 s) and a digital handycam Sony™ DV-CCD DCR-TRV900 (30 fra/s (NTSC) or 25 fra/s (PAL), shutter: 1/4 to 1/10,000 s, zoom: 1 to 48) were used. One camera was installed above and along the axis of the channel while the second took sideview pictures of the wave front through glass sidewall panels.

The initial channel conditions are described in Table 1, column 4 : i.e., initially either empty or filled with a known water depth. The time origin (t = 0) was taken as the time of nappe impact or at the flood wave arrival at the first step. The error on the time was about 1/30 s, the error on the water depth was about ±1 cm and the error on the longitudinal position was ±2 cm.

3 EXPERIMENTAL RESULTS

3.1 Flood wave down a 3.4º stepped chute

In series 1 (α = 3.4º), visual observations showed that the wave front propagated as a nappe flow. Experimental data are shown in Fig. 2. They are compared with the analysis of LAUBER and HAGER (1998) derived from the Saint-Venant equation, assuming a Darcy friction factor f = 0.017 to 0.02. Such values are lower than accepted friction coefficients on stepped chutes (e.g. CHANSON et al. 2000b) suggesting a lesser energy dissipation.

For the 24m long cascade, the average dimensionless wave celerity was found to increase with increasing relative flow rate. The propagation of the leading edge was best correlated by:

$$\frac{x_s}{\sqrt{g * d_o} * t} = 2 * \left(1.16 - \frac{0.15}{\frac{d_o}{h}} \right) \tag{5}$$

where x_s is the distance between the wave front and the intake, and d_o is the initial water level in an ideal dam break problem : i.e., $d_o = 9/4 * \sqrt[3]{Q^2/(g*W^2)}$ (Eq. (3)).

3.2 Flood wave down a 18.4º stepped chute

At Brushes Clough, video footage highlighted that the waters flow down the chute in a nappe flow fashion. Further the flood wave was highly aerated.

The wave front propagation was best correlated by

$$\frac{x_s}{\sqrt{g * d_o} * t} = 6.1 \tag{6}$$

for one flow rate ($d_o/h \approx 2.2$), where x_s is the distance between the wave front and the start of the stepped chute.

The results show a rapid wave propagation, which is faster than analytical predictions for sloping smooth chute (LAUBER and HAGER). The writers speculate that the strong aeration of the wave front might induce some drag reduction mechanism as observed in self-aerated chute flows (CHANSON 1994).

3.3 Flood wave runup downstream of nappe impact

For an initial dry channel, experimental observations showed strong splashing and mixing at the nappe impact. Further the data showed consistently a greater wave front celerity than for the classical dam break analysis for $x/d_o < 25$, where d_o is a function of the initial discharge (Eq. (3)). Close to the nappe impact, the wave celerity was found to be nearly twice that of the dam break wave (WHITHAM's solution) (Fig. 3), the data highlighting a large initial momentum of the wave despite energy loss at nappe impact. The shape of the wave front was close to the analytical profile of WHITHAM (1955).

Practically the experimental data for an initially dry channel were best correlated by :

$$\frac{C_s}{V_i} = \frac{0.598}{1 - 0.0336 * \frac{x}{d_o} + 0.00237 * \left(\frac{x}{d_o}\right)^2} \qquad (7)$$

where V_i is the initial jet impact velocity. For $x/d_o > 30$, the agreement between the data and WHITHAM's solution was fair.

For a channel initially filled with water, the propagation of the flood wave is a different mechanism because the dam break wave is lead by a positive surge. The data showed a slightly greater wave front celerity than for HENDERSON's (1966) theoretical analysis for $x/d_o < 20$ to 25. As a first estimate, the data were best fitted by :

$$\frac{C_s}{V_i} = \frac{0.598}{1 + 0.0263 * \frac{x}{d_o} + 0.00117 * \left(\frac{x}{d_o}\right)^2} \qquad (8)$$

Equation (8) was obtained for $0.1 < d_1/d_o < 0.8$ and $10 < x/d_o < 25$, where d_1 is the initial water depth in the channel. Note that the result is little affected by the initial water depth within the range $0.1 < d_1/d_o < 0.8$.

4 DISCUSSION

Downstream of an overfall, the experimental data highlight large wave celerities during the initial stage (i.e. $x/d_o < 10$) and a wave velocity greater than predicted by WHITHAM's theory for $x/d_o < 30$. Further downstream, some deceleration is caused by bottom friction and turbulent energy dissipation.

For a dam operator, an accurate estimate of the warning time before flood wave arrival is essential to alert populations leaving in the downstream valley. A flood wave may result from a gate malfunction, a rubber dam accident (deflation) or an impulse wave caused by a landslide in the reservoir (e.g. VISCHER and HAGER 1998). A related case is a flood wave propagation over a fully-silted dam for which the reservoir siltation prevents flood attenuation (CHANSON and JAMES 1999, CHANSON et al. 2000a).

Practically the experimental results imply shorter warning times, compared to classical dam break analysis. Detailed calculations were conducted for two prototypes : Palagnedra reservoir and Moore Creek dam. The former is a 72m high concrete dam which was overtopped by a flood wave on 7 August 1978 (BRUSCHIN et al. 1982). The latter is a fully-silted reservoir with zero flood attenuation (CHANSON and JAMES 1999). For a dry channel, the results show that the classical dam break theory overestimates the time of flood wave arrival by up to 50% (CHANSON et al. 2000a). For a downstream channel initially filled with 1m water depth, classical calculations again underestimate the arrival time of the bore. (Calculations were conducted for flood wave discharges ranging from 500 to 3,000 m^3/s and 100 to 500 m^3/s at Palagnedra and Moore Creek respectively.)

The above results (Series 2) may also apply to the prediction of tsunami runup on the shoreline. When a tsunami wave break on the coastline, the bore propagation is somewhat similar to a dam break wave (VISCHER and HAGER 1998, CHANSON et al. 2000a). On a flat shoreline, the abnormal rise of sea level associated with the tsunami wave may runup across dry lands and wetlands, sweeping away buildings and carrying ships inland. (For example, the USS Wateree was carried about 1 km inland during a tsunami on 8 Aug. 1868 in Peru.)

5 SUMMARY AND CONCLUSION

New experiments were conducted in two large-size facilities and in a prototype to predict flood wave propagation when some energy dissipation takes place. The results show that flood wave propagation down a stepped cascade occurs with little energy dissipation, compared to steady stepped spillway flows. For the case of dam overtopping flood wave, the wave celerity is greater than classical calculations for $x/d_o < 30$, where d_o is the equivalent initial water depth. The results are summarised in a series of correlations : i.e., Equations (5) to (8).

Practically the study highlights a reduced warning time for the downstream populations. In two prototype applications, detailed calculations suggest that the warning time could be reduced by up to 50%.

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Table 1 Experimental flow conditions

Experiment	Run	Q(t=0+) m^3/s	Initial channel condition	Remarks
(1)	(2)	(3)	(4)	(5)
Series 1	CT1	0.019	Wet	3.4º chute, h = 0.143 m, W = 0.5 m,
	CT2	0.030	Wet	10 horizontal steps
	CT3	0.040	Wet	
	CT4	0.075	Dry	
	CT5	0.019	Wet	
	CT6	0.030	Wet	
	CT7	0.040	Wet	
	CT8	0.075	Wet	
	BC1	0.5	Empty	Brushes Clough dam spillway : 18.4º chute, inclined downward steps, trapezoidal channel (2 m bottom width)
Series 2	1	0.117	Wet	Horizontal channel, W = 0.8 m, 15 m long. H(t=0) = 1.98 m.
	2	0.124	Wet	H(t=0) = 2.07 m.
	3	0.124	0.03 m	H(t=0) = 2.07 m.
	4a	0.129	0.20 m	H(t=0) = 2.09 m.
	4b	0.111	0.20 m	H(t=0) = 1.89 m.
	4c	0.087	0.20 m	H(t=0) = 1.65 m.

Note : H(t=0) : initial head above channel invert; Q(t=0+) : initial flow rate.

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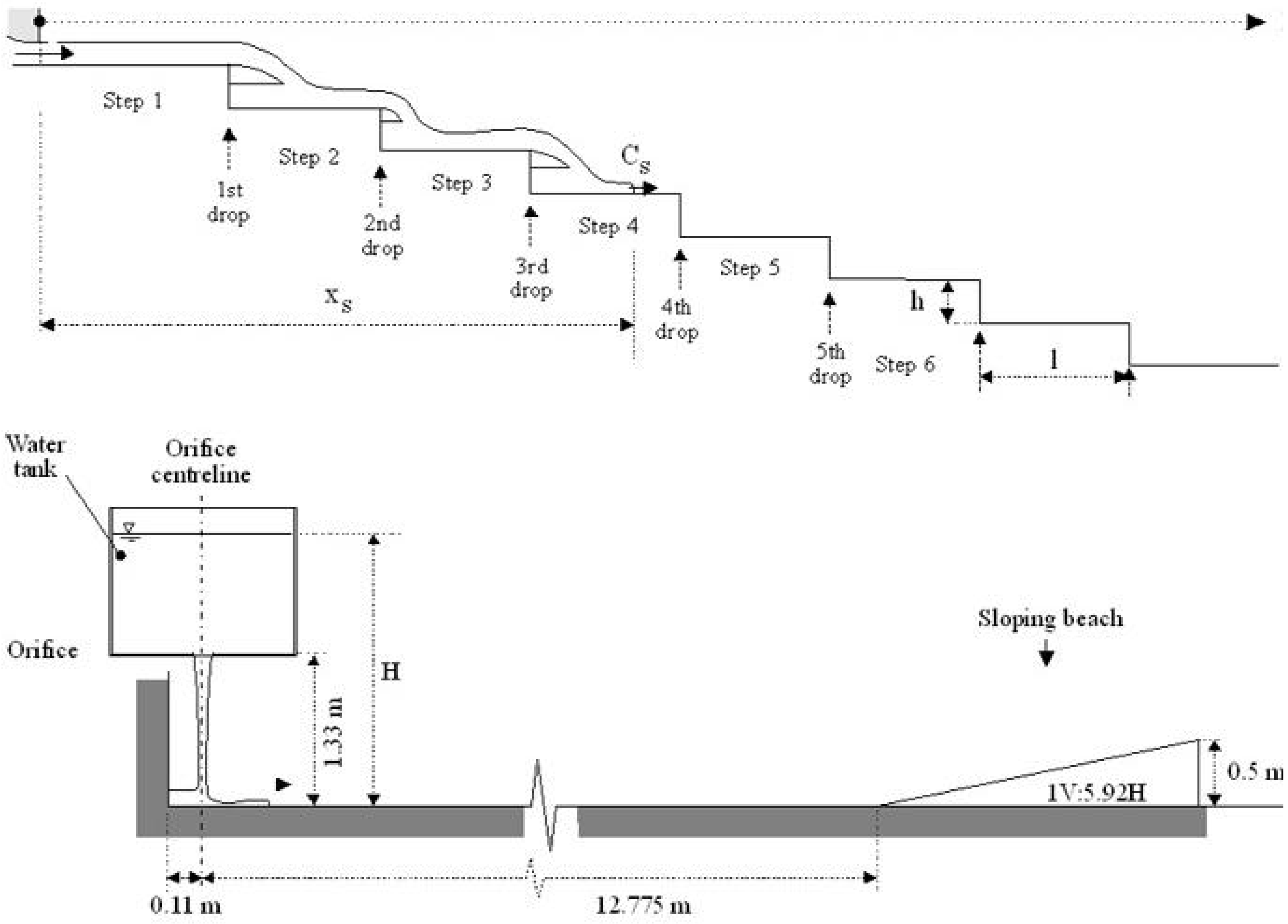
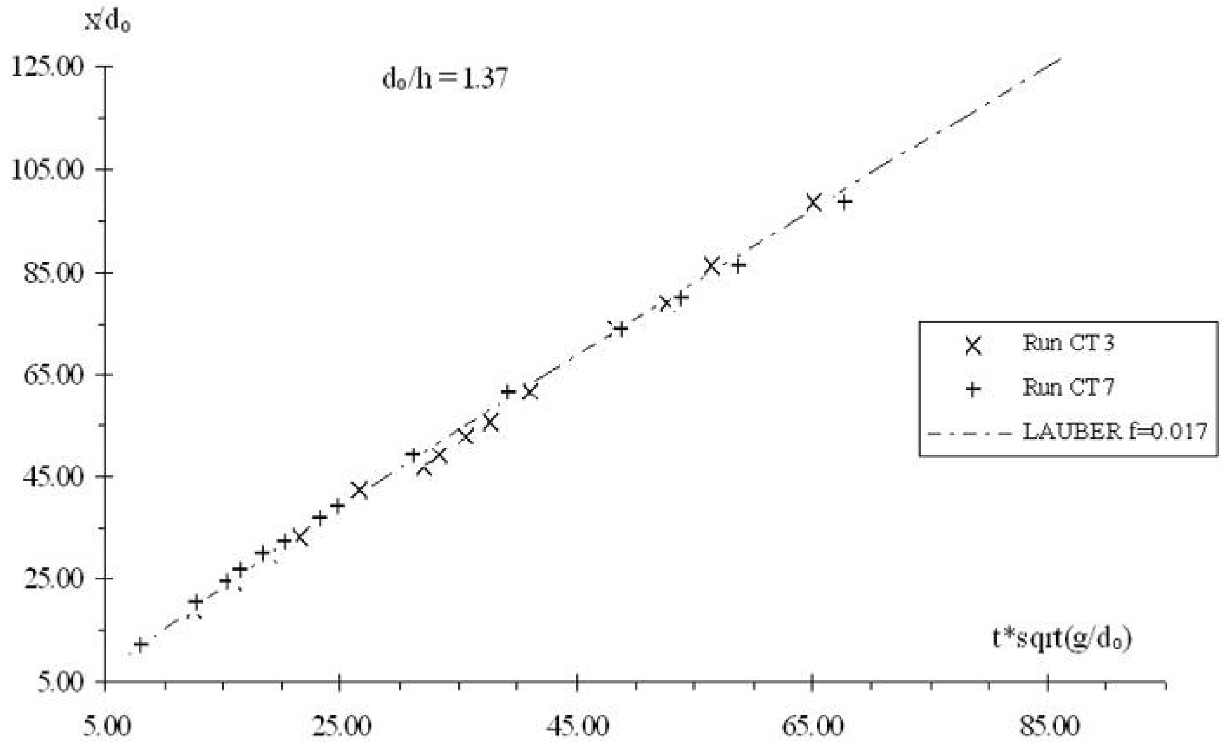
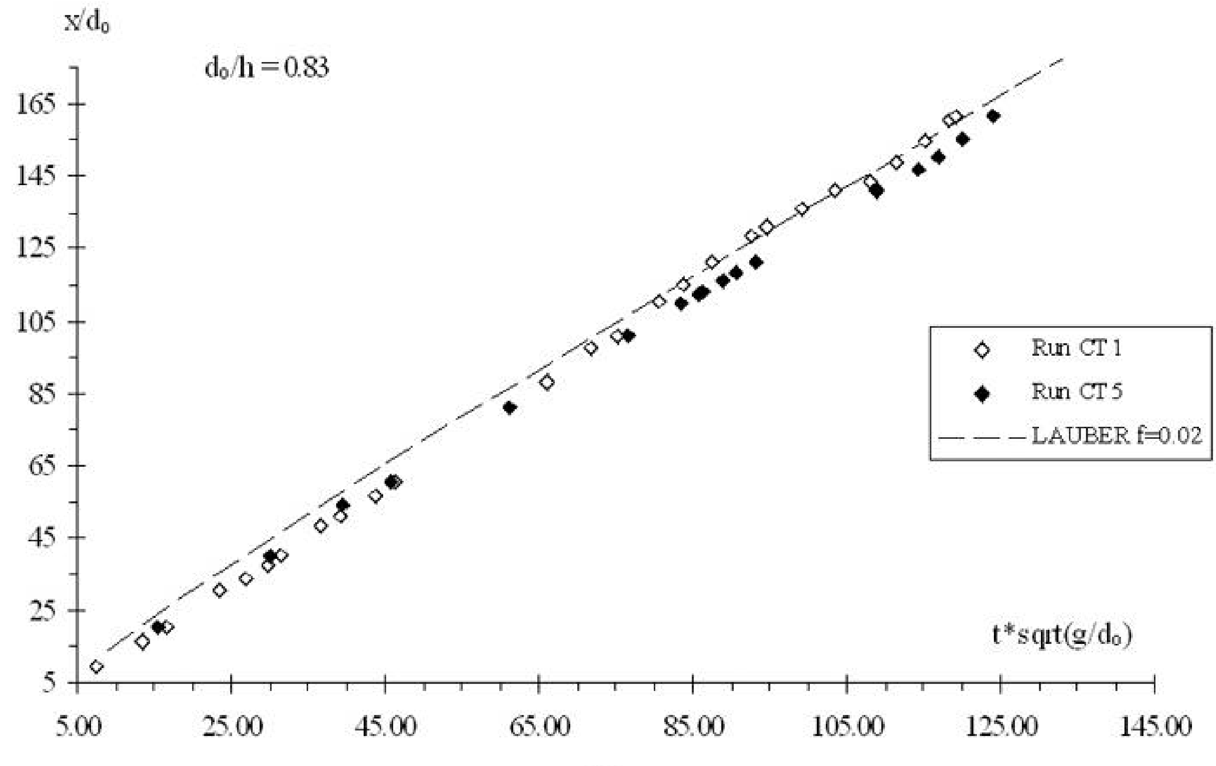


Fig.1 Sketch of the experiments



(B)



(A)

Fig. 2 Wave front propagation experiments series 1 (α = 3.4º, h = 0.143 m)

(A) d_o/h = 0.83, comparison with analysis of LAUBER and HAGER (1998) assuming f = 0.02

(B) $d_o/h = 1.37$, comparison with analysis of
LAUBER and HAGER (1998) assuming f = 0.017

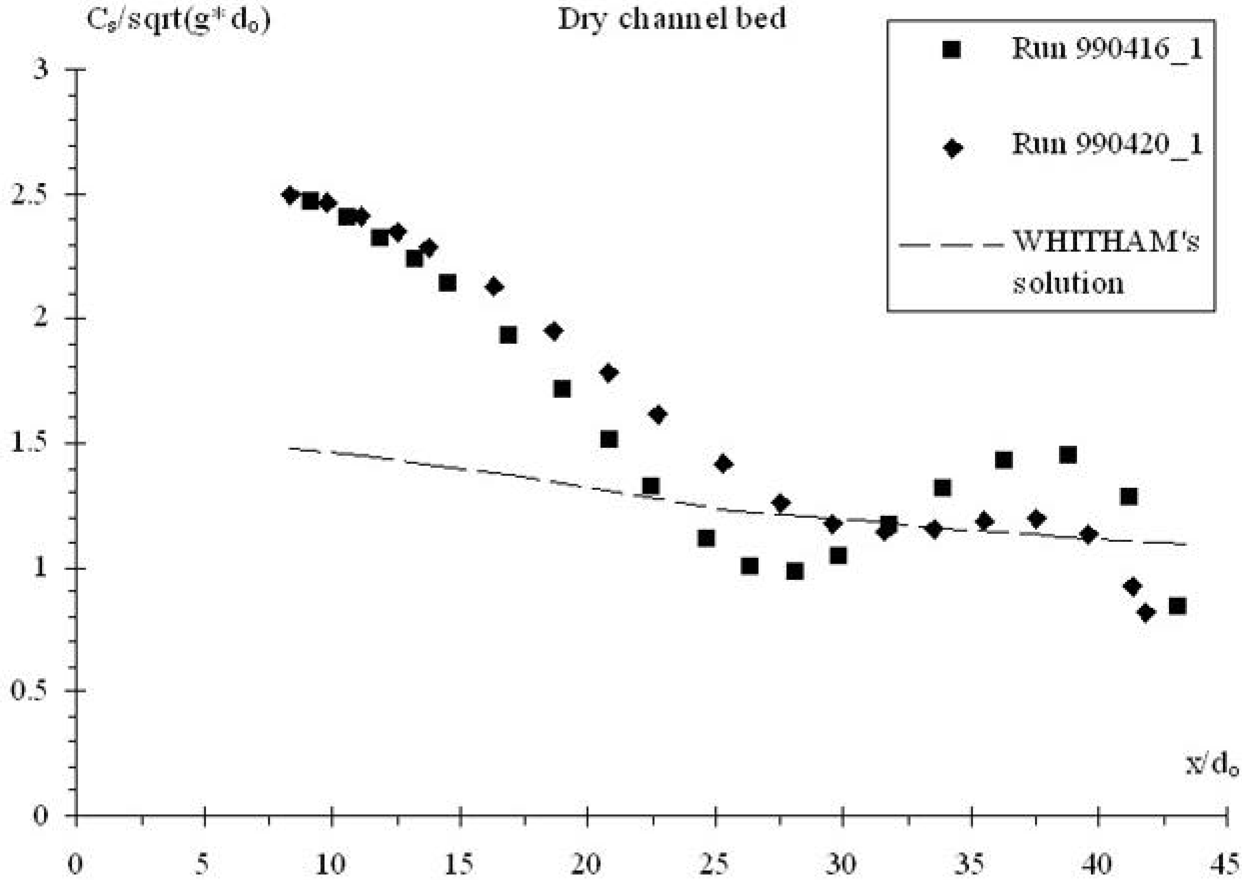


Fig. 3 Experiments Series 2 : Propagation of a flood wave on a dry bed downstream of an overfall - Comparison with WHITHAM's (1955) theory