Journal of Hydraulic Research Vol. 43, No. 4 (2005), pp. 435–438

Discussion

Modeling a washout of dams

By A.L. ROZOV, Journal of Hydraulic Research, Volume 41, 2003, Issue 6, pp. 565–577

Discusser:
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Worldwide 83% of large dams are embankment structures (ICOLD, 1984), and the proportion is even greater for small dams. Embankment overtopping and breach is an important issue, and the Discusser congratulates the Author for his worthwhile contribution which complements the comprehensive study of Coleman *et al.* (2002). This discussion is focused on the breach development stage. It is shown that the 'hourglass' profile is in fact a constant energy state (or minimum energy loss inlet design) for which simple calculations compare favourably with prototype breach flow observations.

Breach development

The overtopping of an embankment is a relatively slow process. During breach development, the breach shape exhibits a hourglass profile as evidenced by the data of Coleman *et al.* (2002) and of the writer, and by Fig. 1. Figure 2 illustrates that a natural breach shape is very similar to inlet designs of Minimum Energy Loss (MEL) culverts and weirs (Coleman *et al.*, 2002; Chanson, 2003). Basically the flow in the breach is near-critical (i.e. $0.5 < \mathrm{Fr} < 1.8$) and the total head remains constant throughout the breach inlet up to the throat (Chanson, 2003). Head losses occurs downstream of the throat when the flow expands and separation takes place at the lateral boundaries.

This analogy between natural breach and MEL inlet design was first proposed by McKay (1970) for natural scour at bridges and for lagoon inlet breach by Gordon (1981), and demonstrated quantitatively by Chanson (2003). Further relevant field observations of inlet breach included Brodie (1988) and Gordon (1990) in Australia and Kraus *et al.* (2002) in USA, while Visser *et al.* (1990) reported a prototype experiment with a 2.2 m high dyke breached during the rising tide.

In an MEL inlet, the flow in the approach channel is contracted through a streamlined inlet into the throat where the channel width is minimum (Apelt, 1983; Chanson, 1999). The inlet must be streamlined to avoid significant form losses and flow separation, and the flow is critical from the inlet lip to the throat. At the

Figure 1 Hourglass shape of Merriespruit tailings dam breach, South Africa (Courtesy of Professor Andre Fourie). The 31-m high tailings dam was overtopped by rainfall–runoff and failed in the night of 22 February 1994.

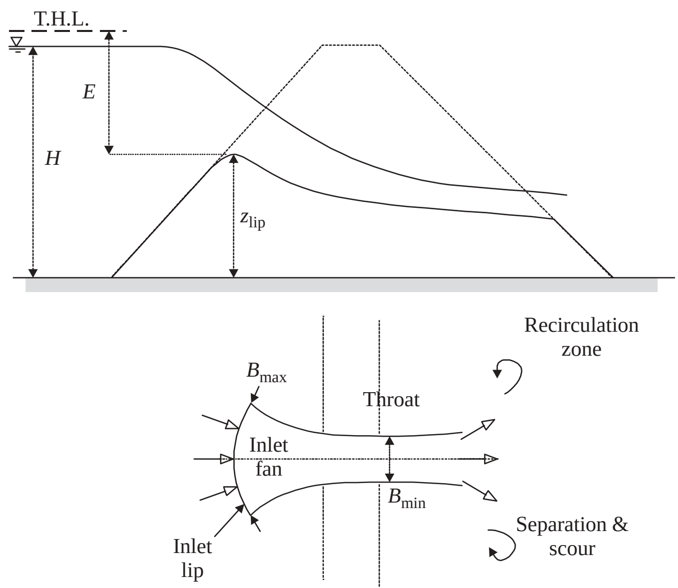

Figure 2 Definition sketch of embankment breach for non-cohesive material. Cross-section through the breach centreline and view in elevation of breach flow

inlet lip, the flow rate into the breach equals:

$$Q = C_D * \frac{2}{3} * \sqrt{\frac{2}{3} * g * E^3} * B_{max} \tag{1}$$

where E is the upstream specific energy above centreline dam breach elevation and B_{max} is the free-surface width at the upper lip of the breach (Fig. 2). The coefficient C_D accounts for the non-rectangular flow cross-sectional shape and some energy loss. For the data of Coleman *et al.* (2002), $C_D \sim 0.6\,m^{1/2}/s$. During an overtopping event, the breach size increases with time resulting in the hydrograph of the breach. The breach free-surface width and specific energy are both functions of time, embankment properties and reservoir size. The re-analysis of embankment breach data suggests that the breach dimensions satisfy:

$$\frac{z_{lip}}{H} = 1.08 * \exp\left(-0.0013 * t * \sqrt{\frac{g}{H}}\right) \quad \text{for } 100 < t * \sqrt{\frac{g}{H}} < 1750 \tag{2}$$

$$\frac{B_{max}}{H} = 2.73 \times 10^{-4} * \left(t * \sqrt{\frac{g}{H}}\right)^{1.4} \quad \text{for } 100 < t * \sqrt{\frac{g}{H}} < 1000 \tag{3}$$

$$\frac{B_{min}}{H} = 4.01 \times 10^{-7} * \left(t * \sqrt{\frac{g}{H}}\right)^{2.28} \quad \text{for } 100 < t * \sqrt{\frac{g}{H}} < 1000 \tag{4}$$

where z_{lip} is the inlet lip elevation on the breach centreline and B_{min} is the free-surface width at the throat (Fig. 2). Equations (2)–(4) are based upon a re-analysis of the data of Coleman *et al.* (2002) obtained with cohesionless materials. Note that Eqs (3) and (4) were deduced from a limited data set.

Applications

The 9 m high Glashütte dam failed on Tuesday, 12 August 2002. Bornschein and Pohl (2003) presented a forensic study of the failure. This flood retention system in the Elbe river catchment failed because of inadequate spillway capacity during a heavy storm event. Witness reports indicated that the dam wall failed completely in less than 30 min between 4:10 and 4:40 p.m. The final breach width was about 21 m at dam crest, and Bornschein and Pohl (2003) calculated a maximum breach outflow of 120 m^3/s.

Application of Eqs (1)–(4) indicate that the reservoir emptied in about 10 min with a maximum breach flow of about 200 m^3/s, while the breach width at throat B_{min} was more than 11 m. The results give some estimate of reservoir drainage time, peak outflow and breach width that are consistent with witness observations and forensic calculations. Similar comparisons were performed successfully with prototype overtopping failures listed by the writer. Overall the proposed development provides a simple, yet physically based system of equations to predict the breach development for cohesionless embankment overtopping.

Acknowledgments

The discusser thanks Dr Stephen Coleman (University of Auckland) for providing his data, and Dr Antje Bornschein (University of Dresden) for information on the Glashütte dam failure.

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Reply by the Author

The author strongly appreciates the Discusser's approach concerning not only the stage of dam breach development but also the one of initial breach formation.

Both the Discusser's approach and the author's one use the analogy between the flow in the breach and the one in the weir. Discusser's equation for the water discharge through the breach is very similar to the author's one (19)

$$Q = c_d g^{0.5} \mathrm{BH}^{1.5} \tag{1}$$

where c_d is discharge coefficient; H and B (discusser's B_{min}) are shown in Fig. 2 of the discussion.

Both approaches differ from one another in the use of a transversal breach characteristic. The author considers B, breach width in the critical section, to be the best one for it can be easily detected in experimental studies and it is used in all computer programs for the calculations of dam-break wave parameters in which the boundary condition at the dam section is applied.

Discusser's Eqs (2)–(4) present a mathematical model based on the latest experimental data of Coleman *et al.* (2002). The author was aware of the experimental studies of Coleman's research group (Coleman *et al.*, 1997) and their data were used widely in the paper. However, their latest works appeared after the author had submitted his work for publication. So, herein the author can state general considerations only. Experimental data can certainly be used for developing a mathematical model of the process at issue provided that full-scale conditions of the dam washout process are satisfied in the experimental model. One more point needs to be given proper attention to. The use of data concerning breaching of noncohesive homogeneous embankment in developing a mathematical model for simulating the process of washout of dams filled with material with apparent cohesion (typical for natural dams) needs careful substantiation.

Let us apply the methods to the investigation of the washout process offered by the Discusser for the Glashütte dam failure case.

It was a dam $H_d = 8.7\,\mathrm{m}$ high; its embankment consisted of an erodible material that can be approximately identified as one of the crushed rock type (no doubt with apparent cohesion). The dam failure process can be reconstructed as follows. The maximum reservoir depth at the dam was about $H = 9\,\mathrm{m}$. So, the maximum difference between the levels of the water overtopping of the dam and the dam crest was about $\Delta h = 0.3\,\mathrm{m}$ (Bornschein and Pohl, 2003; Fig. 4). Therefore, in this case the overtopping of the dam led to formation of the incipient breach in the range of the dam crest where the greatest velocities of the flow occurred (probably in the centre of the crest). Water flowing through the incipient breach eroded first the bottom of the breach until its bottom reached the base of the dam along the whole of its length. This breach is called in the paper the initial breach. Its width, B_0, is estimated as $[B_0 = (H_d + H_d/3)/2]$

$$B_0 = 5.8\,\mathrm{m} \tag{2}$$

Because of some shortage of necessary input data we cannot use the paper's analytical solutions [Eqs (29)–(33) or (38)–(42)]. Nevertheless, Bornschein and Pohl (2003) presented the necessary data for carrying out the theoretical analysis.

We shall use the paper's equation for the rate of the breach-width enlargement [Eq. (27)]

$$\frac{\mathrm{d}B}{\mathrm{d}t} = \alpha\sqrt{g}\frac{A}{\bar{H}^{1/3}}H^{0.5} \tag{3}$$

Water depth at the dam during the failure process from 16^{10} to 16^{40} can be approximated (Bornschein and Pohl, 2003; Fig. 4) by

$$H = H_0 - ct \tag{4}$$

where

$$H_0 = 9\,\mathrm{m} \text{ at } t_0 = 0; \quad H_k = 2\,\mathrm{m} \text{ at } t_k = 30\,\mathrm{min};$$
$$c = 3.9 \times 10^{-3}\,\mathrm{m/s} \tag{5}$$

Substituting (4) into (3) and performing the integration one obtains

$$B = B_0 + \frac{2\alpha\sqrt{g}A}{3c\bar{H}^{1/3}}\left(H_0^{1.5} - H^{1.5}\right) \tag{6}$$

According to the paper [Eq. (47), Table 4, crushed rock]

$$\alpha = 0.05; \quad A = 3.6 \times 10^{-2}\,\mathrm{m}^{1/3} \tag{7}$$

Analytical solution (6) with the help of (2), (5) and (7) gives for the final breach width $B_k(\bar{H} = 5.5\,\mathrm{m})$

$$B_k = 19\,\mathrm{m} \tag{8}$$

Here (8) correlates with historical data ($B_k = 21\,\mathrm{m}$ at dam crest) that are the result of the observations which were made some time after dam failure, when the material from the top of the breach might to a certain extent be crumbled, transforming the rectangular form of the breach into a trapezoidal one.

It is of value to estimate the value of Q_{max}, peak outflow discharge through the breach. Substitution of (2)–(6) into (1) after some mathematical manipulations gives [assuming $c_d = 0.5$, Eq. (22); corresponding breach width and water depth at the dam: $B = 10\,\mathrm{m}$ and $H = 7\,\mathrm{m}$]

$$Q_{max} \sim 300\,\mathrm{m}^3/\mathrm{s} \tag{9}$$

Calculated value Q_{max} by Bornschein and Pohl (2003) is

$$Q_{max} = 120\,\mathrm{m}^3/\mathrm{s} \tag{10}$$

According to their calculations breach outflow discharge after the completion of the process of washout (16^{40}) was (Fig. 4)

$$Q \sim 37\,\mathrm{m}^3/\mathrm{s} \tag{11}$$

On the other hand, according to the same Fig. 4 at time 16^{40} one has

$$H = 2\,\mathrm{m} \tag{12}$$

Time 16^{40}, one should have ($B = 19\,\mathrm{m}$)

$$Q = c_d g^{0.5} B H^{1.5} = 84\,\mathrm{m}^3/\mathrm{s} \tag{13}$$

Comparing (13) and (11) we have a right to conclude that Bornschein and Pohl (2003) somehow undervalued the breach outflow discharge.

Compare the Discusser's Fig. 2 and Fig. 6 in the paper. The diagram in Fig. 6 is used for the case of breach (lateral) development. The diagram in Fig. 2 of the discussion can be used for two cases: uniform washout of the whole length of the dam crest and formation of initial breach. The former is a very rare one. Johnson and Illes (1976) in their detailed classification of dam failures did not even mention it. This process needs particular conditions for its realization. In the case of Glashütte dam failure these conditions were absent. Therefore the diagram in Fig. 2 can be used for the case of formation of initial breach. Figure 2 displays an important feature of this process. As a consequence of the more intensive erosion of the downstream slope of the embankment (where the highest flow velocities take place) there is some low speed displacement of the topmost section of the breach in the direction of the upstream wall. This phenomenon shows the complexity and the importance of the problem at hand. The problem embraces several allied aspects. For example, according to known data values of discharge coefficient c_d depend on the ratio of reservoir's and breach's widths to some degree. The method proposed by the Discusser, which includes both experimental and theoretical techniques, is especially suitable for the study of this problem.

Acknowledgments

The author thanks Dr Vitaliy Klimovich (VNIIG B.E. Vedeneev) for providing information.

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