*Journal of Hydraulic Research* Vol. 43, No. 4 (2005), pp. 439–444


Discussion

# Modeling of washout of dams

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

*Discussers:*
M.J. FRANCA, PhD Student, *Laboratoire d'Hydraulique Environnementale—EPFL, 1015 Lausanne, Switzerland*
A.B. ALMEIDA, Full Professor, *Department of Civil Engineering, Technical University of Lisbon, IST, 1049-001 Lisbon, Portugal*

## 1 Introduction

Dam breach modelling still constitutes a challenge for the scientific community at it is evidenced by the relative accuracy of the simulation models (CADAM, 2000). The work from Andrey L. Rozov is a very important and valuable contribution: the topic and the numerical scheme presented can be a useful tool for practical and scientific purposes. The methodology is well structured and is sustained by experimental results. Recently the discussers (M.J. Franca and A.B. Almeida) were involved in a project on dam breaching of rockfill dams, based on laboratory experiments similar to the ones shown here (Franca and Almeida, 2002), and developed a similar computational model, applicable to rockfill dams (Franca and Almeida, 2004), where the breach simulation is based on a modified Exner erosion equation applied only in one breach cross-section. A later version of the model expanded its application scope for embankment dam ruptures caused by overtopping (Franca *et al.*, 2004). The use of the Exner equation for the breaching simulation seems to be very appropriated for use in dam breaching lumped models (Singh, 1996). Its simple formulation permits an easy implementation and small dependency on empirical parameters; the erosion constants can be related with the dam material allowing a *quasi*-rational approach of the problem. We allowed ourselves to comment A.L. Rozov's work in some points, hoping that this will contribute to improve further improvements of the author's work. We will divide our discussion in four parts, which includes: general comments on the paper, breach development, model application, and conclusions.

## 2 General comments

Several comments and questions showed up when reading A.L. Rozov's document.

1. It is referred that the initial breach has a width between one-third of the dam height and the dam height value. Franca and Almeida (2002) refers the formation of an initial breach in the rockfill dam cases with the size of about the dam height. This point should deserve a reflection: is it an input parameter or not? Is it a random output?
2. It is not clear how each essay began. It seems that, when filling the reservoir, and until the beginning of the experiment, some impervious structure was placed in the incipient breach location, avoiding the beginning of the erosion process (Figs 4 and 5). If so, it should be very interesting to compare these results with experiments where an incipient breach in the top of the dam was induced (for instance a small channel in the middle of the dam crest as made; Visser, 1998), letting the breach develop laterally and also downwards. We think that the non-consideration of the bottom breach erosion can induce errors on the maximum discharge assessment and on the reservoir depletion simulation (see Section 3 of this discussion). It is also not very clear if there was any kind of impermeable layer on the dam model or if the experiments were made accounting only on the dam material non-permeability.
3. It would be interesting to present direct hydrograph taken from one representative test and not what seems a smoothed graphic showed on Fig. 3. It would be interesting to compare the time simulation of a dam breach made with earth with the ones presented to rockfill cases by Franca and Almeida (2002). In these experiments the hydrographs obtained are characterized by the occurrence of several discharge peaks. These ones correspond to the occurrence of a cascade of rockslides or instabilities, which induce the sudden enlargement of the breach area as a consequence of the drop on the breach bottom and of the widening of the breach width.
4. Finally, it seems that Table 6 has an error on the headers of the columns, easily understandable and that does not hinder the text comprehension.

## 3 Breach development

In our opinion, the most pertinent question to be posed about the proposed model is the consideration of an incipient breach which depth is equal to the total dam height. In fact, this assumption does not agree with several historical data and cannot be applied universally to any kind of embankment dam. As shown by Franca and Almeida (2002) to rockfill dam breaching, the eroded material is deposited immediately downstream of the dam base, restraining the breach depth development to about 80% of the dam height.

The discussers also did not verify simply lateral breach erosion in their experiments. It is our opinion that for earth and mixed dam cases, one should not consider only exclusive lateral erosion in the breach evolution simulation. Taking the historical data presented by Singh (1996), in about one-third of the cases where there is information, the breach does not reach the valley bottom. For the calculation of the maximum discharge through the breach and of the breach evolution time, the hypothesis made on the incipient breach may induce important errors.

It seems pertinent to introduce the integrated reservoir–breach response and the reservoir classification concept introduced by Walder and O'Connor (1997). This concept applies to embankment dams and allows the qualitative characterization of the expected outflow hydrograph. The designation of small or large reservoir does not relate with the reservoir absolute volume but with the relation between the reservoir dynamic response and the breach erosion rate. In fact the present methodology from A.L. Rozov can be considered valid for the large reservoir case. In this case the maximum discharge occurs when the breach reaches its maximum wet surface, since the reservoir water level does not change significantly along the breach evolution time (mostly one can consider it constant). However, in the small reservoir cases, the moment of the occurrence of the maximum discharge through the breach is obtained from the analysis of the evolution of the reservoir dynamic response and the breach erosion rate.

This can be shown taking the discharge equation (19) on A.L. Rozov's text, considering the hydraulic head equal to $H = N_{\mathrm{R}} - N_{\mathrm{B}}$ ($N_{\mathrm{R}}$ is the reservoir water level and $N_{\mathrm{B}}$ is the breach bottom level) and applying the equation for the discharge control section. Considering $B$, $N_{\mathrm{R}}$ and $N_{\mathrm{B}}$ time variables, knowing that the maximum occurs to $\frac{\mathrm{d}Q}{\mathrm{d}t} = 0$, we have:

$$\frac{\mathrm{d}Q}{\mathrm{d}t} = 0 \Rightarrow \frac{\mathrm{d}}{\mathrm{d}t}\left(c_{\mathrm{d}}\sqrt{g}B(N_{\mathrm{R}} - N_{\mathrm{B}})^{1.5}\right) = 0 \tag{1}$$

Considering $c_{\mathrm{d}}$ constant, and developing the last expression we obtain:

$$\frac{\mathrm{d}B}{\mathrm{d}t}\frac{1}{B} = -1.5\frac{\mathrm{d}(N_{\mathrm{R}} - N_{\mathrm{B}})}{\mathrm{d}t}\frac{1}{(N_{\mathrm{R}} - N_{\mathrm{B}})} \tag{2}$$

The maximum discharge occurs when there is equality between the relative breach width erosion rate and $-1.5$ times the relative breach hydraulic head evolution rate. The breach hydraulic head evolution rate accounts for the breach bottom erosion rate and for the dynamic response of the upstream valley confining the reservoir. This expression shows one general condition for the determination of the maximum discharge. From expression (2), the occurrence of the maximum discharge is conditioned also by the breach bottom development, which means that it can occur before the bottom erosion ends.

Franca and Almeida (2004) developed the RoDaB model with an Exner erosion equation modified and adapted to simulate the breach bottom and lateral erosion individually, and used the classical principle of the mass conservation in terms of water volume flux for the reservoir routing:

$$\frac{\mathrm{d}N_{\mathrm{B}}(t)}{\mathrm{d}t} = -C_{\mathrm{s,b}}\frac{Q_{\mathrm{B}}^{\beta_{\mathrm{s,b}}}}{A_{\mathrm{B}}^{\beta_{\mathrm{s,b}}}} \tag{3}$$

$$\frac{\mathrm{d}B(t)}{\mathrm{d}t} = C_{\mathrm{s,m}}\frac{Q_{\mathrm{B}}^{\beta_{\mathrm{s,m}}}}{A_{\mathrm{B}}^{\beta_{\mathrm{s,m}}}} \tag{4}$$

$$\lambda_{\mathrm{R}}(Q_{\mathrm{i}} - Q_{\mathrm{B}} - Q_{\mathrm{C}}) = \frac{\mathrm{d}N_{\mathrm{R}}}{\mathrm{d}t} \tag{5}$$

where $C_{\mathrm{s,b}}, C_{\mathrm{s,m}}, \beta_{\mathrm{s,b}}, \beta_{\mathrm{s,m}}$ are bottom and lateral erosion constants, $A_{\mathrm{B}}$ the wet breach surface, $Q_{\mathrm{B}}$, $Q_{\mathrm{i}}$, $Q_{\mathrm{C}}$ the breach, input and over the crest discharges and $\lambda_{\mathrm{R}}$ the dynamic response coefficient of the reservoir.

## 4 Model application

We analyse now the results for the Salles Oliveira dam accident (Brazil) using the RoDaB model. In RoDaB, calibration is made only on the erosion constants.

The Salles Oliveira earthen dam was overflowed for seven hours (Ponce, 1982 cited by Singh, 1996) until a discharge peak due to the upstream Euclides da Cunha dam breaking induced the beginning of the breaching process. The breaching process took about 2 h until it was finished. According to Singh (1996) a discharge peak of $1.01 \times 10^3\ \mathrm{m}^3/\mathrm{s}$ was released from the collapse of the upstream dam. This value was used in the definition of the inflow hydrograph upstream from the Salles Oliveira dam. The general dam characteristics of the dam and the breach parameters are given in Table D1:

Table D1 Application example—Salles Oliveira dam and breach characteristics (Singh, 1996)

| Dam height (m) | Crest length (m) | Reservoir volume ($\mathrm{m}^3$) | Breach width (m) | Breach depth (m) | Peak discharge ($\mathrm{m}^3/\mathrm{s}$) |
|---|---|---|---|---|---|
| 35 | 167 | $2.6 \times 10^{7}$ [a] | 131 [b] | 35 | 7200 |

[a] This value is different from the one considered in A.L. Rozov's applications; here we consider not the released volume but the reservoir volume.
[b] In A.L. Rozov's paper the final breach width referred is equal to the valley width; anyway, as we will see, the breach final width does not affect the maximum discharge value.

The RoDaB model was calibrated taking into account the time of breach formation and the peak discharge released when the dam collapse occurred. We considered $t_{\mathrm{B}}$ (breach time evolution) the moment when the discharge is maxima. The trigger moment for the beginning of the breaching process was the existence of an overflow depth of 0.10 m on the dam crest. The breach discharge coefficient considered in formula (19) from Rozov's document is $c_{\mathrm{d}}\sqrt{g} = 1.30$, which is in accordance with Coleman *et al.* (1997). The calibrated values for the erosion constants are $C_{\mathrm{s,b}} = C_{\mathrm{s,m}} = 3.2 \times 10^{-4}$ and $\beta_{\mathrm{s,b}} = \beta_{\mathrm{s,m}} = 2$. The following table shows the comparison between our results, the ones from A.L. Rozov and the historical data.

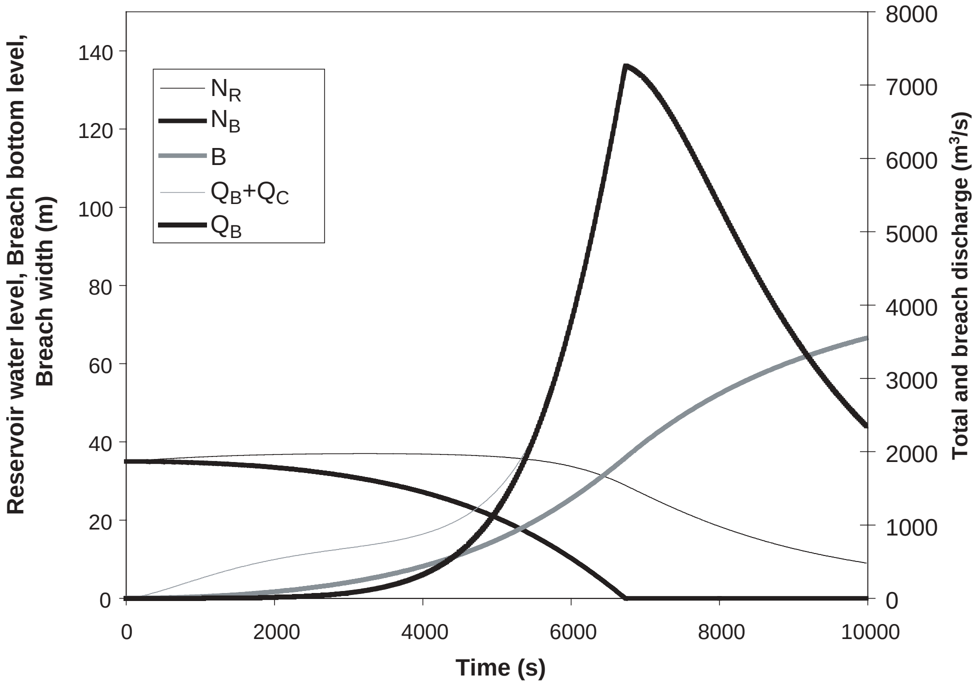


Figure D1 Results from the RoDaB model to the Salles de Oliveira dam accident.

Table D2 Salles Oliveira dam accident: historical data (Singh, 1996); A.L. Rozov's results; RoDaB results

| Historical data | | | A.L. Rozov's results | | | RoDaB results | | |
|---|---|---|---|---|---|---|---|---|
| $B$ (m) | $Q_{max}$ ($m^3$/s) | $t_B$ (h) | $B$ (m) | $Q_{max}$ ($m^3$/s) | $t_B$ (h) | $B$ (m) | $Q_{max}$ ($m^3$/s) | $t_B$ (h)[b] |
| 131 | 7200 | 2 | 120–147 | 14400 | 2.84 | 90[a] | 7260 | 1.9 |

[a]At the moment of the maximum discharge the breach width was 37 m.

Figure D1 shows the time evolution of several of the variables simulated by the RoDaB.

In this case the breach lateral development is not conditioning for the maximum discharge value. According to our results the maximum discharge occurs when the breach reaches the dam bottom, i.e. when the hydraulic head in the breach bottom is maximum. From Figure D1, and according to RoDaB results, the breach development can continue after the maximum discharge until its final configuration. The non-consideration of the input discharge by A.L. Rozov in his simulations may also justify the differences on the example here presented. It would be interesting to see his results considering now the inflow discharge into the reservoir referred by Singh (1996) of $1.01 \times 10^3$ $m^3$/s.

## 5 Conclusions

As shown, the maximum discharge calculation, can be a function of: the final hydraulic head on the breach bottom or the final breach wet surface, both for large reservoirs; or the balance between the reservoir dynamic response and the breach lateral and bottom erosion rates for small reservoir. In our opinion, the consideration of the breach bottom evolution is of easy implementation on Rozov's model and the additional computational effort is not relevant.

Further improvements based mainly on experimental studies must be made in this kind of models, A.L. Rozov's and RoDaB, in order to clarify some points: time variation of the discharge coefficient accounting to the variation of the hydraulic head, the influence of lateral and of tail water effects; better calibration of the erosion parameters; better final breach geometry estimations.

Dam breaching depends on multiple factors and each case must be considered as unique. The compound effect of the factors has a random component. Monte Carlo and stochastic models can be very useful in the context of a probabilistic analysis. In our opinion, it will always be very difficult to predict each breach event with full accuracy. This fact imposes the acceptance of error margin and risk evaluation criteria.

## Acknowledgments

The authors wish to express their gratitude to Frank Braunschweig for his help on the RoDaB simulations.

## References

1. CADAM—Concerted Action on Dambreak Modelling (2000). Final Report, January.
2. Coleman, S.E., Jack, R.C. and Melville, B.W. (1997). "Overtopping Breaching of Noncohesive Embankment Dams". *Proceedings of the 27th Congress on the IAHR—Energy and Water: Sustainable Development*, San Francisco, August.

3. FRANCA, M.J. and ALMEIDA, A.B. (2002). "Experimental Tests on Rockfill Dam Breaching Process". In: *Proceedings of the International Symposium on Hydraulics and Hydrological Aspects of Reliability and Safety Assessment of Hydraulic Structures*. St. Petersburg, Russian Federation.
4. FRANCA, M.J. and ALMEIDA, A.B. (2004). "A Computational Model of Rockfill Dam Breaching Caused by Overtopping (RoDaB)". *J. Hydraul. Res. IAHR* 42(2), 888–999.
5. FRANCA, M.J., BRAUNSCHWEIG, F. and ALMEIDA, A.B. (2004). "RoDaB: a User-friendly Computational Model of Dam Breaching". Poster pres. *International Conference on Hydraulic Dams and River Structures*, Teheran, Iran.
6. PONCE, V.M. (1982). "Documented cases of earth dam breaches". SDUS–Civil Eng. Series No. 82149, San Diego State University, San Diego, CA.
7. SINGH, V.P. (1996). *Dam Breaching Modeling Technology.* Kluwer Academic Publishers, Dordrecht, pp. 151–219,
8. VISSER, P.J. (1998). *Breach Growth in Sand-Dikes, Communications on Hydraulic and Geotechnical Engineering—Report No. 98.1.* Technische Universiteit Delft, Delft.
9. WALDER, J.S. and O'CONNOR, J.E. (1997). "Methods for Predicting Peak Discharges of Floods Caused by Failure of Natural and Constructed Dams". *Water Resour. Res.* 33(10), 2337–2348.

**Reply by the Authors**

The author would like to thank M.J. Franca and A.B. Almeida for the very useful discussion of the paper. The discussers suggested to examine extremely important subjects concern features of the dam-breach development and the calculation of the maximum discharge through the breach.

First, note that the discussers consider the presented methodology valid for the large reservoir cases when "the reservoir water level does not change significantly along the breach evolution time". Undoubtedly, it is a certain misunderstanding. According to the model one has, for example

$$B \propto \frac{B_k}{2}, \quad H \propto 0.75 H_0 \qquad \text{at } t = t_{\max} \tag{1}$$

where $t_{\max}$ is the time of maximum water discharge through the breach, $B$ and $H$ are breach width at the critical section and reservoir depth at the dam at the time $t = t_{\max}$, respectively, and $H_0$ is initial reservoir depth at the dam.

The presented model assumes the formation of a comparatively small ($B \sim H_d/3 - H_d$, where $H_d$ is dam height) initial breach with its bottom reaches the base of the dam and further breach lateral development keeping a rectangular form. This model is the result of the author's experimental investigations. Practically the same results were derived from experimental studies for earth dam cases by Tinney and Hsu (1961), Bechteler and Broich (1991), Coleman *et al.* (1997) and for mixed and rockfill dam cases by Tinney and Hsu (1961). The same pattern of dam-breach erosion is presented by Johnson and Illes (1976) (based on approximately 100 historical dam failure cases) and MacDonald and Langridge-Monopolis (1984) (based on 42 historical dam failure cases). This physical pattern of the process seems contradict some other historical data. Note should be taken that historical data are often the result of the observations which have been made some time after dam collapse, when the material of the embankment might, to a certain extent, be crumbled. Besides, flow velocities in the breach by the end of the reservoir depletion decrease to the value of sedimentation velocity (especially for gravel, pebbles and crushed rock) that causes sedimentation in the breach. So, in the experiments by Bechteler and Broich (1991) the depth of the sediment layer in the breach was about 0.03 m; approximately the same sedimentation in the breach was in the author's experiments. But this stage of the reservoir depletion does not have an influence on dam-break wave parameters. At the same time, historical data obtained during an active stage of the breach development are the evidence of the breach with its bottom reaching the base of the dam (Water, 1960) (mixed and crushed rock). So are the data of the large-scale field dam-breach washout experiment (Tinney and Hsu, 1961) (mixed and crushed rock dam).

As the determination of the dam-break wave parameters demands use of the breach which is being formed during its development, then one should operate with the breach which bottom reaches the base of the dam.

Analyse the results of the discussers' experiments. In these experiments maximum flow velocities in the breach can be estimated as

$$v \propto \sqrt{\frac{2g}{3} \times H} \propto \sqrt{\frac{2g}{3} \times 0.3} \propto 1.4\,\mathrm{m/s} \tag{2}$$

where the hydraulic head $H = N_R - N_B$; $N_R$, $N_B$ are reservoir water and breach bottom levels, respectively; $N_R \propto 0.4$ m; $N_B \propto 0.1$ m (Franca and Almeida, 2002, 2004).

The material of the dam was $d_{50} = 0.0189$ m. The author estimates the material characteristics as

$$v_n \propto 1 - 1.2\,\mathrm{m/s}; \quad v_m \propto 2.8 - 3.4\,\mathrm{m/s} \tag{3}$$

where $v_n$ is non-eroding velocity and $v_m$ is critical flow velocity (under this velocity the bottom layer with its thickness no less than $d_{50}$ comes to the motion).

In this experiments flow velocities in the breach were less than $v_m$ and did not satisfy the full-scale condition of the dam washout process (Eq. 1; Rozov, 2004). Under such conditions the material of the embankment crumbled during the process and accumulated within some limits on the bottom of the breach. In the result the depth of the breach was less than dam height. Certainly, these experimental data are of a large value for dam-breach washout investigations but they cannot be used for direct recommendations for natural conditions.

Examine the discussers' relation (2). The discussers consider it proves the possibility of the occurrence of the maximum discharge through the breach before its bottom reaches the base of the dam. One has

$$B,\ \frac{\mathrm{d}B}{\mathrm{d}t},\ N_{\mathrm{R}},\ N_{\mathrm{B}},\ (N_{\mathrm{R}}-N_{\mathrm{B}}),\ -\frac{\mathrm{d}N_{\mathrm{R}}}{\mathrm{d}t},\ -\frac{\mathrm{d}N_{\mathrm{B}}}{\mathrm{d}t} > 0 \tag{4}$$

The left part of the discussers' Eq. (2) is positive. Hence, its right part has to be positive too, that is the following relations have to be satisfied:

$$\frac{\mathrm{d}N_{\mathrm{R}}}{\mathrm{d}t} < \frac{\mathrm{d}N_{\mathrm{B}}}{\mathrm{d}t},\quad \left|\frac{\mathrm{d}N_{\mathrm{R}}}{\mathrm{d}t}\right| > \left|\frac{\mathrm{d}N_{\mathrm{B}}}{\mathrm{d}t}\right| \tag{5}$$

But if (5) is realized from the very beginning of the overtopping of the dam then the process will be soon finished and the dam will be safe. Hence, dam failure needs realization the converse condition from the outset of the process

$$\frac{\mathrm{d}N_{\mathrm{R}}}{\mathrm{d}t} > \frac{\mathrm{d}N_{\mathrm{B}}}{\mathrm{d}t},\quad \left|\frac{\mathrm{d}N_{\mathrm{R}}}{\mathrm{d}t}\right| < \left|\frac{\mathrm{d}N_{\mathrm{B}}}{\mathrm{d}t}\right| \tag{6}$$

Consequently, the transformation from (6) into (5) during dam washout is the necessary condition for the occurrence of the maximum discharge through the breach. The discharge through the breach is negligible and hence the value of $|\mathrm{d}N_{\mathrm{R}}/\mathrm{d}t|$ is very little till the bottom of the breach reaches the base of the dam as it simultaneously means that the breach width is rather small [it is validated by experimental and historical dam failure data (see above)]. Therefore, one would expect that the transformation from (6) to (5) is possible after the bottom of the breach reaches the base of the dam.

Another type of dam failure cases, characterizing by uniform washout of the whole length of the dam crest in consequence of powerful overtopping, is very rare (Rozov, 2004). Certainly, this type of dam washout deserves thorough examination because of substantial discharge over the dam crest during its erosion.

Discussers suggested to analyze Salles Oliveira dam failure. They presented related to this failure data unknown to the author. Summarizing all known data concern this disaster one can come to the following dam failure scenario. Due to the upstream Eucldes da Cunha dam (with its reservoir volume and depth substantially larger than Salles Oliveira ones) breaking, overtopping of the Salles Oliveira began. Dam crest erosion began some hours later since, to all appearances, the dam had a protective covering. According to MacDonald and Langridge-Monopolis (1984) the difference between the reservoir water level and the dam crest level was more than 3 m. Under such conditions after the protective covering was destroyed uniform erosion of the whole length of the dam crest took place. In the result the whole length of the dam with the exception of non-erosion parts (spillway, electric power station) was washed out. In consequence in this case breach depth equaled dam height according to the all known historical data. Thus, Salles Oliveira dam washout case falls outside the limits of the applicability of both the author's model and the discussers' one because there was not lateral erosion of the breach in this case; on estimating the maximum water discharge through Salles Oliveira dam one has to take the length of the embankment as the breach width.

The author can answer the discussers' questions in the following way:

1. The width of the initial breach was estimated on the basis of known historical data. Especially on the data by Johnson and Illes (1976) in which triangular form of initial breach was replaced by a rectangular one keeping the water discharge through the breach the same.
2. The hydrograph presented in the paper, in fact, was not smoothed. During experiments discharge peaks were absent.
3. Until the beginning of the experiments measures were taken to prevent both the reservoir water outflow through the breach and water filtration into the embankment. There was not any kind of impermeable layer on (or into) the dam because of low filtration velocities, $10^{-4}$–$5 \times 10^{-4}$ m/s, and rather little duration of dam-breach development, 250–300 s.

The author fully agrees with the discussers that his model needs for simulation of the process of formation of the initial breach in the dam and, in particular, uniform washout of the whole length of the dam crest in the case of powerful overtopping. In a first approximation the simulation is presented in the work (Rozov, 2004).

The author completely agrees with the discussers' list of necessary experimental investigations to clarify some aspects of the problem in question and considers the discussers' experimental facility built at the CEHIDRO Laboratory (Franca and Almeida, 2002, 2004) has all necessary conveniences to execute them properly provided due account of restrictions on embankment's material and carry out dam washout measurements.

The author agrees with the discussers that Monte Carlo and stochastic models can be useful in the context of a probabilistic analysis of some aspects of the problem in question concern, for example, the probability of realization of conditions that lead to dam failures. But calculations of the process of dam washout should have a deterministic nature and call for use theoretical models that fit the process adequately.

**Acknowledgments**

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

**References**

1. BECHTELER, W. and BROICH, K. (1991). "Effects in Dam-Break Modelling". *Proceedings of the 24th Congress of the IAHR* Madrid, Spain, Vol. A.
2. COLEMAN, S.E., JACK, R.C. and MELVILLE, B.W. (1997). "Overtopping Breaching of Noncohesive Embankment Dams". *Proceedings of the 27th Congress of the IAHR* San Francisco, CA, USA, theme D.
3. JOHNSON, F.A. and ILLES, P. (1976). "A Classification of Dam Failures". *Water Power Dam Constr.* December, 43–45.

4. MacDonald, T.C. and Langridge-Monopolis, J. (1984). “Breaching Characteristics of Dam Failures”. *J. Hydraul. Engng. ASCE* 110(5), 576–586.
5. Rozov, A.L. (2004). “Formation of Initial Breach in Erodible Dams”. p. 2721, in review.
6. Singh, V.P. and Hsu, Y.H. (1988). “Analysis of Gradual Earth-Dam Failure”. *J. Hydraul. Div. ASCE* 114(1), 1–29.
7. Tinney, R.E. and Hsu, Y.H. (1961). “Mechanics of Washout of an Erodible Fuse Plug”. *J. Hydraul. Div. ASCE* 87(3), 1–29.
8. Water (1960). “The Breaching of the Oros Earth Dam in the State of Ceara, North-East Brazil”. *Water Engng.* August, 351–355.