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A computational model of rockfill dam breaching caused by overtopping (RoDaB)

Un modèle numérique d'ouverture d'une brèche dans un barrage en enrochement provoquée par débordement (RoDaB)

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ABSTRACT

The outflow hydrograph from a dam failure is a boundary condition of a dam break flood model used on the downstream valleys risk management. Limited research has been made on the rockfill dams breaching process and there are no specific models to this type of structures yet. This paper describes a lumped model for the computation of the outflow hydrograph due to a Rockfill Dam Breaking named RoDaB. The methodology is based on the governing equations of reservoir routing and depletion, and breach erosion. Results obtained from experimental tests performed in a laboratory flume were considered to fulfil the phenomenological aspects to which does not exist any analytical approach so far. The erosion process from the dam breach is modelled as a function of two erosion parameters and of the breach final geometry dimensions obtained from the experiments. Finally, the model RoDaB is applied to a rockfill dam, with characteristics typical of this type of structure, and the results are compared with the ones from the BREACH model.

RÉSUMÉ

L'hydrographe du à la rupture d'un barrage est une condition de frontière pour les modèles de propagation de crues utilisés dans la gestion du risque a l'aval. Peu de recherche a été faite dans la rupture des barrages d'enrochements et on ne connaît pas aucun modèle spécifique pour ce type de barrage. Cet article présent un nouveau modèle pour la computation de l'hydrographe résultant de la rupture d'un barrage de enrochement que s'appèle RoDaB. La méthodologie c'est basée aux équations de déplétion du réservoir et d'érosion de la brèche. Les résultats obtenus avec les expérimentes en laboratoire ont été utilisés pour caractériser quelques aspects phénoménologiques pour lesquels, a ce moment, n'existe pas encore une approximation analytique. L'érosion de la barrage est modelée comme une fonction des deux paramètres d'érosion et de la géométrie finale de la brèche suggérée par les résultats des essais en laboratoire. Finalement, le modèle RoDaB est destiné aux barrages d'enrochement typiques, et on présent une comparaison avec les résultats du modèle BREACH.

Keywords: Dam break, rockfill dams, breach modelling, computational model.

1 Introduction

The presence of a dam induces an additional risk in the valley due to the huge destructive potential of the water stored in the reservoir. The outflow hydrograph resulting from a dam failure represents the upstream boundary condition of any dam break flood model to be used on risk management. During the last decades several attempts have been made to predict this type of hydrograph. These can be divided in two main groups when concerning to embankment dams: (i) approaches using historical dam failures data and regression approximations to calculate the hydrograph based on the dam and reservoir general characteristics (usually the volume of the reservoir and/or the dam height)—Wahl (2001) makes an overall view of these methods; and (ii) approaches using semi-analytical methods established from the physical laws of breach progress and of reservoir depletion—see the overview by Singh (1996). Another group includes stochastic models (Kast and Bieberstein, 1997). In CADAM (2000) there are pointed out several conclusions regarding the state-of-the-art about dam breaching modelling and it is clear that the lack of knowledge in this domain is huge. An uncertainty of about 50% in the estimate of the maximum discharge is expected on the results that the existent models can offer.

Previous studies characterizing the dam breach outflow are mainly directed to earth dams. The rockfill dam breach has not been a strong research topic so far. Nevertheless some accidents have happened with this kind of dam. Table 1 shows general information on accidents with rockfill dams. This information was obtained from the following sources: Combelles (1979), Singh and Scarlatos (1988), Serafim and Coutinho-Rodrigues (1989), Walder and O'Connor (1997), Martins (2000), and Broich (2002).

In 2001, an experimental study was carried out at the CEHIDRO Laboratory (Instituto Superior Técnico—Lisbon Technical University), in order to fulfil the lack of knowledge

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Table 1 Historical accidents with rockfill dams.

Country	Name of the dam	Year of construction/ accident	Height (m)	Reservoir volume ($\times 10^6$ m^3)	Causes of failure
Argentina	Frías	1940/1970	15	–	Spillway insufficiency; overtopping
Australia	Cascade	–/1929	19	–	–
Australia	Cethana	1968/1968	15	–	Overtopping
Brazil	Oros	1960/1960	35	650.0	During construction; overtopping
Germany	Sose	1931/1959	54	–	–
India	Chitauni	–/1968	–	–	–
Indonesia	Sempor	1967/1967	54	52.0	Overtopping
Mexico	La Calera	1963/1964	28	–	–
South Africa	Xonxa	1973/1973	48	158.0	During construction; overtopping and piping
Spain	Odiel	1970/1970	35	3.3	During construction; overtopping
URSS	Karachunovskaya	–/1934	22	–	During construction
URSS	Nizhne Tulomskaya	1938/1938	29	–	During construction
USA	Beaver Park	1914/1914	30	–	–
USA	Black Rock	1907/1909	21	–	Percolation on the foundation
USA	Bluewater	1908/1909	4,6	3.2	Overtopping
USA	Bowman North	1927/1928	51	–	–
USA	Bully Creek	1913/1925	38	–	Problems with the slope protection and the watertight element
USA	Castlewood	1890/1933	21	4.2	Foundation percolation and overtopping
USA	Cogswell	1934/1934	85	–	During construction
USA	Goose Creek	1903/1916	64	10.6	Overtopping
USA	Hell Hole	1964/1964	67	37.0	During construction; overtopping
USA	Kelly Barnes	1948/1977	13	0.5	Seepage
USA	Littlefield	1929/1929	37	–	During construction; piping
USA	Lower Otay	1897/1916	41	49.0	Spillway insufficiency; overtopping
USA	Masterson	1950/1951	18	–	–
USA	Moreana	–	–	–	–
USA	Swift	1914/1964	57	37.0	Spillway insufficiency; overtopping
USA	Wahiawa	1906/1921	41	–	–
USA	Walnut Grove	1888/1890	34	–	Overtopping
USA	Wisconsin Dells	1909/1911	18	–	Overtopping

about the breaching mechanism on rockfill dams. An experimental facility was assembled for this particular purpose (Franca and Almeida, 2002). Several tests were carried out to better understand the breaching mechanism and to characterize the breach final configuration when induced by the overtopping of the dam.

The present paper presents a semi-analytical lumped model to simulate the breaching process on rockfill dams named RoDaB. The formulation of the model cannot simulate all the details of a rockfill dam breach. In fact, the real hydrograph is characterised by several discharge peaks corresponding to the occurrence of successive rockslides that induces the sudden enlargement of the breach cross-area (Franca and Almeida, 2002). The aim of this model is to furnish a consistent continuous hydrograph appropriated enough for downstream flood simulation. The simulation of the breach evolution is based on an erosion equation and on the results obtained with the physical experiments by Franca and Almeida (2002). The erosion equation (Exner equation) reflects the erosive potential of the breach bottom material components and is characterized by two parameters, $C_{s,b}$ and $\beta_{s,b}$. The bottom sediment transport is computed as a function of the average velocity and the bottom level variation is determined using a technique similar to the approach used in the BEED model presented by Singh and Scarlatos (1988). The width evolution rate, which is a result of the lateral erosion, is described as a fraction of the bottom drop rate based on the experimental tests observation. The breach final configuration is imposed to the model from the results of the experiments. RoDaB considers an average width of the breach for the computation of the discharge, and it does not take into account any backwater effect from downstream. The model was developed for any overtopping failure of rockfill dams with a homogeneous body and an impervious layer on the upstream slope. The RoDaB model is applied to a case study and its results are compared with the ones given by the well-known BREACH model (Fread, 1984b) to earth and to rockfill dams.

This paper contains a brief presentation of the most important results achieved with the physical experiments, a description of the proposed mathematical model of the reservoir depletion and dam breaching (RoDaB), a calibration of the model RoDaB based on historical dam breach data, an example of application of the RoDaB model and a comparison with the results obtained by the BREACH model.

2 Experimental tests

Franca and Almeida (2002) carried out several tests (the total number of tests was 22) on rockfill dam breaching, using reduced scale models in an experimental set-up built in the laboratory of CEHIDRO, in Lisbon, Portugal. The models were 0.5 m high, with slopes of 1.0 : 1.5 (vertical : horizontal) on both upstream and downstream sides, with a 0.2 m wide and 2.0 m long crest. The experimental study was made considering a Froude number similarity.

The experiments furnish, among others, the following conclusions:

(1) the geometry that best fits the breach final configuration is the parabolic one;
(2) the final top width of the breach is approximately 2.25 times the dam height;
(3) the final average width of the breach is approximately 1.70 times the dam height;
(4) the overflow induces an initial breach width of approximately 1.00 times the dam height;
(5) the final breach depth is approximately 80% of the dam height, which means that 20% of the dam height is not eroded in the breach;
(6) the average lateral erosion rate of the breach is about 80% of the average bottom erosion rate;
(7) the total failure time observed in the models was between 450 and 1200 s which corresponds to a time between 54 and 144 min in a real rockfill dam with 25 m height.

The rockfill breach development consists mainly in a multiple step process due to the rock instability behaviour as a response to the dam overtopping. Figure 1 illustrates two phases of one of the breaching tests.

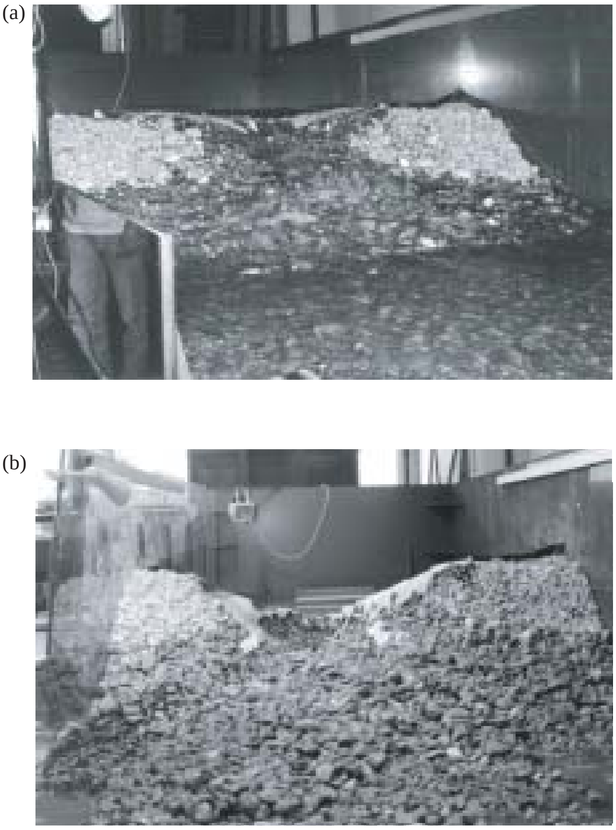

Figure 1 Example of one of the experimental tests. (a) Model overtopping, (b) final breach configuration.

The models were a representation of a specific type of rockfill dams with an impermeable membrane at upstream. In all experiments they had a special membrane that tried to simulate a thin concrete plate over the rock. A description of these experimental tests and their subsequent results can be found in Franca and Almeida (2002). These empirical results were considered in the approximate dam breach model developed by the authors.

3 Governing equations

3.1 General introduction

The RoDaB model is based on a reservoir routing scheme having the inflow hydrograph as upstream boundary condition and the discharge through the breach and over the dam crest as the downstream boundary condition. The model stands for the following hypothesis:

(1) The dam rupture is partial and gradual which allows neglecting the inertial and wavy effects in the reservoir level and in the hydraulic head on the breach due to surface wave propagation in the reservoir.
(2) The rupture is induced by the overtopping of the dam body and the local effects, caused by slope instabilities or by internal erosion due to the percolation flow, are not directly considered. Seepage effects are induced by the overflow that infiltrates through the embankment crest, these effects being globally considered in the experimental behaviour of the dam.
(3) The downstream backwater effect does not affect the discharge on the breach.

RoDaB calculates the breach evolution in the flow control section

3.2 Reservoir routing

The principle of the reservoir routing used in the RoDaB model follows the well-known equation based on the mass conservation principle in terms of water volume flux (flow):

$$Q_i - Q_\mathrm{B} - Q_\mathrm{C} = \frac{\mathrm{d}V_\mathrm{R}}{dt} \tag{1}$$

where Q_i is the upstream inflow [$\mathrm{L}^3\mathrm{T}^{-1}$], Q_B the discharge through the breach [$\mathrm{L}^3\mathrm{T}^{-1}$], Q_C the discharge over the dam crest [$\mathrm{L}^3\mathrm{T}^{-1}$] and V_R the water volume in the reservoir [L^3]. Lateral inflow and other outflow discharges through turbines, spillways, and other outlet structures, were not considered but they can easily be introduced in the model; however, they are assumed to be negligible when compared with the values taken by the outflow volume through the dam crest and the breach. Equation (1) can be written in the following form that represents the depletion rate of the reservoir:

$$\lambda_\mathrm{R}(Q_i - Q_\mathrm{B} - Q_\mathrm{C}) = \frac{\mathrm{d}N_\mathrm{R}}{\mathrm{d}t} \tag{2}$$

where dN_R is the variation of the level in the reservoir [L] and λ_R is the time reservoir coefficient [L^{-2}] that gives information from the hypsometric function (this latter one reflects the upstream valley geometry). A rectangular valley upstream of the dam was here considered as a simplification; therefore the time reservoir coefficient corresponds to the inverse of the reservoir reference (or surface) area (A_R). Typically, rockfill dams are built in large valleys, where the hypothesis of approximate rectangular valleys is fairly valid. This assumption is made between two time steps in a reservoir routing computation and it is very easy to introduce an additional procedure in the model that adjusts the value of the time coefficient λ_R for each time step as a function of the reservoir level in the preceding time step.

The main task is to define routines to compute the variables related to the outflow over the dam crest (Q_C) and through the dam breach (Q_B).

3.3 Discharge over the dam crest

In an overtopping dam failure the discharge over the crest can be very intense during the first instants of the accident. The rockfill is considerably more stable and less erosive than the compacted soil (Stephenson, 1979). Consequently, rockfill dams tend to be more stable and resistant to the overtopping flow than earth dams. In a rockfill structure, the overtopping discharge can attain relatively high values without inducing the beginning of the failure (Franca and Almeida, 2002). For the computation of the flow over the crest, the following methodology based on the results presented by Martins (1984) for dams with an impervious zone upstream and an unprotected crest was implemented. This method was considered the most suitable and it is based on the broad crested weir discharge equation. The discharge coefficient is calculated using a regression equation obtained from experimental results (Martins, 1984):

$$Q_C = C_C L'_C \sqrt{2g}(N_R - N_C)^{1.5} \tag{3}$$

$$C_C = 0.333 + 0.132\frac{N_R - N_C}{B} \tag{4}$$

where C_C is the crest discharge coefficient [−], L'_C is the overtopped crest length [L], g is the gravity acceleration [LT^{-2}], N_C is the crest level [L] and B is the crest width [L]. This formulation is consistent with the hypothesis of neglecting the inertial head effects caused by wave propagation in the reservoir.

It is assumed that during the first instants of overflowing, the erosive capability of the water flow is enough to remove the crest protection. Nevertheless, this is not a closed issue and further research should be carried to clarify the discharge law over a rockfill dam crest.

3.4 Breach discharge

According to Singh and Scarlatos (1988), experimental and field observations showed that the discharge through the control section of the breach could be calculated with a broad crested weir equation. Several authors uses this approximation: Visser (1998), Loukola and Huokona (1998), Mohamed *et al.* (1999) and the well-known models BEED (Singh, 1996), BREACH (Fread, 1984b) and DAMBRK (Fread, 1984a). The broad crested weir equation used on the RoDaB model is the following:

$$Q_B = C_B W_B (N_R - N_B)^{1.5} \tag{5}$$

where C_B is the breach discharge coefficient [−], W_B is the average breach width [L], and N_B is the breach bottom level [L]. Coleman *et al.* (1997) presented results from experiments with cohesionless material in the dam body and suggested a constant value of 1.3 for the breach discharge coefficient. This value was implemented on the RoDaB model. Since the discharge coefficient is considered constant, the flow through the breach will be only influenced by breach evolution and by the hydraulic head.

Further investigation should be made on the characterization of the flow through the breach on this kind of dams taking into account the variation of the discharge coefficient with the hydraulic head, the influence of lateral effects due to the breach walls (mainly on the exponent of the Eq. 5) and the submergence of the breach. Fread (1984b) presents a method to take into account the submergence of the breach in its outflow. This research needs to be based on controlled experiments both in reduced and in full-scale models.

3.5 Breach erosion

For the evaluation of the breach configuration, a rational method combining an analytical formulation based on the Exner equation and the empirical results from the experiments is presented. In the RoDaB model the Exner equation (Exner, 1925) is applied on the discharge control section of the breach for the determination of the breach bottom level evolution. Graf and Altinakar (1998) proposed a rearrange of the Exner equation rewritten in the form of the solid phase continuity equation. Considering a unit space step ($\Delta x = 1$) and considering that at the upstream boundary (corresponding to the reservoir) the solid discharge is null, the later equation takes the following form, where the solid discharge is computed in the downstream section of the unit space step:

$$\frac{dN_B}{dt} + \frac{1}{1-p} q_{s,b} = 0 \tag{6}$$

where p is the porosity [−] and $q_{s,b}$ is the specific solid discharge from the bottom material [L^2T^{-1}] (b means bottom). Graf and Altinakar (1998) suggested that the solid discharge calculation could simply be computed as a function of the average flow velocity and of two erosion constants.

$$q_{s,b} = \alpha_{s,b} U_B^{\beta_{s,b}} \tag{7}$$

where $\alpha_{s,b}$ is the erosion coefficient [$L^{2-\beta}T^{\beta-1}$], $\beta_{s,b}$ is the erosion exponent that depends essentially on the dimensions of the bottom material [−] and U_B is the average flow velocity on the breach [LT^{-1}]. In the RoDaB model the computation of the breach bottom evolution is based on Eqs (6) and (7), using the next equation for the determination of the bottom erosion rate:

$$\frac{dN_B}{dt} = -C_{s,b}\frac{Q_B^{\beta_{s,b}}}{A_B^{\beta_{s,b}}} \tag{8}$$

where A_B is the breach flow cross-section [L^2] and $C_{s,b}$ is an empirical coefficient that includes the constants $\alpha_{s,b}$ and $1/(1-p)$ [$L^{1-\beta}L^{\beta-1}$]. The breach cross-section is computed using the average width multiplied by the hydraulic head on the breach bottom. For the computation of the breach width erosion rate the following result from the experiments was taken into consideration: the average lateral erosion rate of the breach is about 80% of the average bottom erosion rate.

According to the experimental results, the RoDaB model assumes, in the beginning of the computation, an initial breach width of 1.00 times the dam height that works as a trigger to the following breaching process. The breach final configuration also obtained from the experimental results from Franca and Almeida (2002) is used as a final limit for its evolution. These imposed conditions reflect the specific characteristics of a rockfill dam breach according to the experiments.

4 Model calibration

Considering the empirical final breach configuration conditions as a solved issue, the RoDaB model depends only on two empirical values suitable of calibration: the two erosion parameters $C_{s,b}$ and $\beta_{s,b}$. Using data from historical dam accidents, or predictions believed to correspond to a good approximation of reality, the RoDaB model was indirectly calibrated taking into consideration the maximum discharge through the breach and its time of occurrence by a simple trial and error method. Without the monitoring data of real full-scale dam failures this approximate calibration was the one chosen for the model validation.

Singh (1996) showed, with a similar lumped mathematical model for the computation of the breach outflow, that analytical solutions for the rectangular breach evolution are only valid when the exponent $\beta_{s,b}$ is less than or equal to 2. However, several authors present values of $\beta_{s,b}$ greater than 2 in the sediment transport formula, namely CUR/RWS (2000) that proposes the value of 5 applied to rockfill under flowing water. In the present work the calibration of the model considers exponent values equal to 1, 2 or 5. The trial and error procedure is applied to adjust the coefficient $C_{s,b}$ to each of these $\beta_{s,b}$ values.

Table 2 presents the historical data used for the model calibration ($Q_{P,B}$—maximum breach discharge; t_F—time of failure; h_B—breach final depth; W_B—final breach average width) and Table 3 shows the correspondent results obtained with the RoDaB model for the several tested pair of values $C_{s,b}/\beta_{s,b}$ ($Q_{P,B,sim}$—maximum breach discharge simulated; $t_{F,sim}$—time of failure simulated; $h_{B,sim}$—breach final depth simulated; $W_{B,sim}$—final breach average width simulated). The historical dam failures data presented were collected from several sources indicated below the table. Data corresponding to the failures of Bluewater, Lower Otay and Sempor dams were calculated using the empirical formulae proposed by Froehlich (1987).

Table 2 Historical data from rockfill dam failures (due to overtopping only).

Dam	$Q_{P,B}$ (m^3/s)	t_F (h)	h_B (m)	W_B (m)
Bluewater[a]	253	2.20	–	39.0
Cethana[b]	170	5.17	9.1	–
Goose Creek[c]	565	0.5	4.1	26.4
Hell Hole[d]	7350	0.75	35.10	–
Lower Otay[a]	8566	1.30	–	141.6
Oros[c]	11500	–	35.5	200[e]
Sempor[a]	12230	1.05	–	152.1

[a] Based on Froehlich (1987).
[b] Broich (2002).
[c] Singh and Scarlatos (1988).
[d] Walder and O’Connor (1997).
[e] Top width.

In the computations made with the RoDaB model, the inflow discharge was considered null, which seems to be an acceptable approximation (Singh, 1996). Only the cases where the dam failure was caused by overtopping of the crest were considered for this analysis.

When the erosion exponent was equal to 5 the model RoDaB was not able to provide good results. Thus in the following only will be considered the simulations where the exponent was $\beta_{s,b} = 1$ or $\beta_{s,b} = 2$.

The simulations of the Bluewater and Goose Creek cases are the only ones where it was not possible to reach a good estimate of the maximum breach discharge. These two cases correspond to the so-called ‘large’ reservoirs situations referred by Walder and O’Connor (1997) and Wahl (2001) since the water level does not change significantly during the simulation. This means that the maximum discharge of the breach outflow hydrograph is mostly a function of the final configuration of the breach. In these cases, the maximum discharge occurs when the breach reaches its lowest level.

The predicted breach depth gives reasonable results with a maximum error of about 53%, but the predicted breach width is always smaller that the expected value. In the case of the ‘large’ reservoirs these two breach parameters are crucial for the determination of the maximum discharge thus more experimental research should be made on the determination of the final breach geometry for this situation.

In the other cases, $\beta_{s,b} = 1$ offers better results than with the exponent equal to 2. In fact, good results are obtained with the unitary value, in terms of the pair maximum discharge and failure time. From our tests the erosion coefficient $C_{s,b}$ is about 2.0×10^{-3} for $\beta_{s,b} = 1$ and $5.0 \times 10^{-4}\ m^{-1}\ s$ for $\beta_{s,b} = 2$. Figure 2 shows the distribution of the calculated erosion coefficients for the chosen dam failure cases.

Comparing the erosion coefficient here estimated with the one calibrated for the BEED model to earth dams (Singh and Scarlatos, 1988), the average erosion coefficient presented for the RoDaB model is about one half of it. Jandora (2001) proposed for the erosion coefficient the value 6.0×10^{-3} that is about the triple form RoDaB’s proposed coefficient. Thus, one can state that the erosion potential is higher in the case of earth dams than in rockfill dams. This means that the consequences of a rockfill dam failure in the downstream valley are expected to be of

Table 3 Calibration of erosion coefficients from the RoDaB model (erosion coefficients).

Dam	$\beta_{s,b}$ (–)	$C_{s,b}$ $((m/s)^{1-\beta})$	$Q_{P,B,sim}$ (m^3/s)	$t_{F,sim}$ (h)	$h_{B,sim}$ (m)	$W_{B,sim}$ (m)	$Q_{P,B,sim}/Q_{P,B}$ (–)	$t_{F,sim}/t_F$ (–)	$h_{B,sim}/h_B$ (–)	$W_{B,sim}/W_B$ (–)
Bluewater	1	3.40E-04	66	2.20	3.7	7.8	0.26	1.00	–	0.20
	2	3.33E-04	69	2.20	3.7	7.8	0.27	1.00	–	0.20
	5	5.26E-05	0	2.20	3.7	7.8	0.00	1.00	–	0.20
Cethana	[a]	[a]	–	–	12.2	25.8	–	–	1.34	–
	[a]	[a]	–	–	12.2	25.8	–	–	1.34	–
	[a]	[a]	–	–	12.2	25.8	–	–	1.34	–
Goose Creek	1	1.70E-03	138	0.50	4.8	10.2	0.24	1.01	1.17	0.39
	2	1.50E-03	139	0.49	4.8	10.2	0.25	0.98	1.17	0.39
	5	7.00E-03	139	0.50	4.8	10.2	0.25	1.00	1.17	0.39
Hell Hole	1	2.60E-03	7347	1.45	53.7	114.1	1.00	1.93	1.53	–
	2	5.51E-04	7342	2.14	53.7	114.1	1.00	2.85	1.53	–
	[b]	[b]	–	–	53.7	114.1	–	–	1.53	–
Lower Otay	1	1.70E-03	8574	1.42	32.9	69.9	1.00	1.09	–	0.49
	2	3.38E-04	8569	2.50	32.9	69.9	1.00	1.92	–	0.49
	[b]	[b]	–	–	32.9	69.9	–	–	–	0.49
Oros	1	2.00E-03	11496	1.04	28.4	60.4	1.00	–	0.80	0.30
	2	4.50E-04	11561	1.63	28.4	60.4	1.01	–	0.80	0.30
	[b]	[b]	–	–	28.4	60.4	–	–	0.80	0.30
Sempor	1	2.35E-03	12205	1.23	43.2	91.8	1.00	1.17	–	0.60
	2	4.30E-04	12015	2.12	43.2	91.8	0.98	2.01	–	0.60
	[b]	[b]	–	–	43.2	91.8	–	–	–	0.60

[a] Not enough available data.
[b] Solutions given by the model were not physically possible (since failure time tends to infinite).

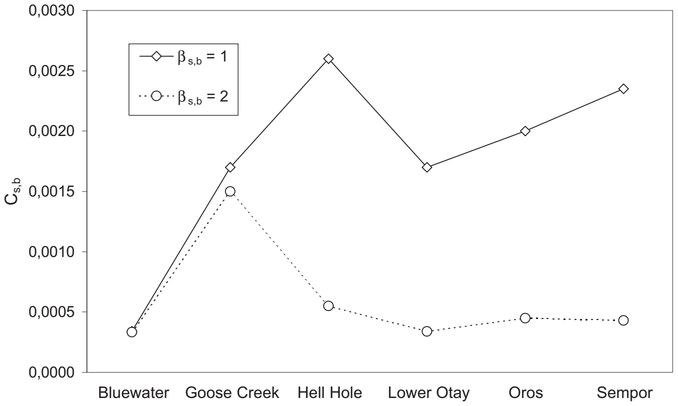

Figure 2 Calibration of the model—variation of the erosion coefficients.

minor degree since the rupture mechanism tend to be more gradual therefore the destructive potential will be lower. Any time delay during the dam breaching process will be a positive factor in what concern the valley warning and evacuation procedures (Almeida and Viseu, 1997).

5 Applications

5.1 Data

The RoDaB model was applied to a typical rockfill dam considering $\beta_{s,b} = 1$ and taking the average value of the erosion coefficient ($C_{s,b} = 2.0 \times 10^{-3}$). The characteristics of this example dam are shown in Table 4. The dam dimensions correspond to a scaling factor of $\lambda_l = 50$ relatively to the dam models considered on Franca and Almeida (2002). The block dimensions can be considered as representative of the ones used on real rockfill dams. A constant inflow hydrograph with a discharge value of 700 m^3/s was considered. The BREACH model was also used on the simulation of the dam breaking in order to compare the results. Two simulations were made with the BREACH model: one corresponding to the same dam used to test the RoDaB model; and another with the same dam dimensions but this time with a clay core.

Besides the data in Table 4, other parameters were necessary to run the models. For the rockfill dam: $D_{50} = 100$ mm; $D_{90}/D_{30} = 13.3$; unit weight of the rock blocks (26.5×10^3 N/m^3); porosity ratio (0.40); internal friction angle ($\alpha = 40.0°$); and cohesive strength (0 N/m^2). For the earth dam: $D_{50} = 0.01$ mm; unit weight of the rock blocks (18.9×10^3 N/m^3); porosity ratio (0.45); internal friction angle (20.0°); and cohesive strength (71.8×10^3 N/m^2). Table 5 presents the different model formulations for discharge over the crest, discharge through the breach, sediment transport and breach evolution.

5.2 Results

The main results from the three simulations, using RoDaB and BREACH models, are presented on the Table 6. The results are $Q_{P,B}$—maximum breach discharge; t_P—time to peak discharge; t_F—time of failure; h_B—breach final depth; and W_B—final breach average width. Figure 3 shows the three computed breach outflow hydrographs.

Table 4 Application example—data of the example dam.

Height (m)	Crest length (m)	Crest width (m)	Upstream slope (V : H)	Downstream slope (V : H)	Reservoir superficial area (km^2)	Inflow discharge (m^3/s)
25	100	10	1 : 1.5	1 : 1.5	0.50	700

Table 5 General presentation of RoDaB and BREACH basic equations.

RoDaB	BREACH
$Q_\mathrm{C} = C_\mathrm{C} L'_\mathrm{C}\sqrt{2g}(N_\mathrm{R} - N_\mathrm{C})^{1.5}$; $C_\mathrm{C} = 0.333 + 0.132\frac{N_\mathrm{R}-N_\mathrm{C}}{B}$	$Q_\mathrm{C} = 3L'_\mathrm{C}(N_\mathrm{R} - N_\mathrm{C})^{1.5}$
$Q_\mathrm{B} = C_\mathrm{B} W_\mathrm{B}(N_\mathrm{R} - N_\mathrm{B})^{1.5}$; $C_\mathrm{B} = 1, 3$	$Q_\mathrm{B} = 3W_\mathrm{B}(N_\mathrm{R} - N_\mathrm{C})^{1.5}$ or $Q_\mathrm{B} = 3W'_\mathrm{B}(N_\mathrm{R} - N_\mathrm{C})^{1.5} + 2\tan\alpha(N_\mathrm{R} - N_\mathrm{C})^{2.5}$
$q_\mathrm{s,b} = \alpha_\mathrm{s,b} U_\mathrm{B}^{\beta_\mathrm{s,b}}$	(Meyer–Peter–Muller modified by Smart)
$\frac{\Delta N_\mathrm{B}}{\Delta t} = -C_\mathrm{s,b}\frac{Q_\mathrm{B}^{\beta_\mathrm{s,b}}}{A_\mathrm{B}^{\beta_\mathrm{s,b}}}$	$\Delta N_\mathrm{B} = \Delta t\frac{Q_\mathrm{s}}{P_\mathrm{o} L_\mathrm{B}(1-p)}$
$\frac{\Delta W_\mathrm{B}}{\Delta t} = F\frac{\Delta N_\mathrm{B}}{\Delta t}$	$W_\mathrm{B} = W_\mathrm{B,o} + N_\mathrm{B}\sin\alpha$

Table 6 Results from the RoDaB and BREACH application.

RoDaB					BREACH—rockfill dam					BREACH—earth dam				
$Q_\mathrm{P,B}$ (m^3/s)	t_P (h)	t_F (h)	h_B (m)	W_B (m)	$Q_\mathrm{P,B}$ (m^3/s)	t_P (h)	t_F (h)	h_B (m)	W_B (m)	$Q_\mathrm{P,B}$ (m^3/s)	t_P (h)	t_F (h)	h_B (m)	W_B (m)
3736	0.84	0.85	20.0	42.5	8125	0.15	0.15	25.0	42.2	6925	0.13	0.13	25.0	31.4[a]

[a] Rectangular breach.

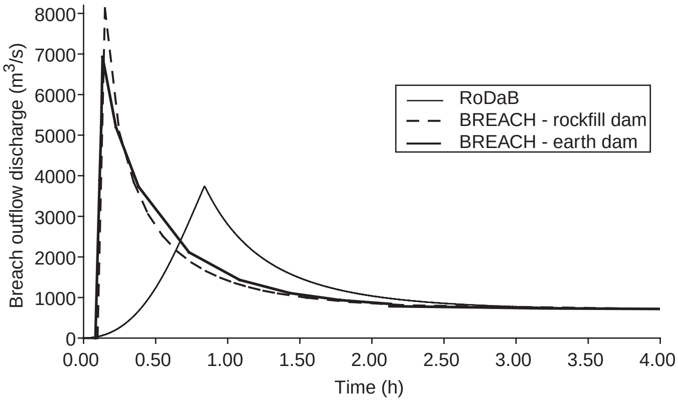

Figure 3 Example case—breach outflow hydrographs obtained by different techniques.

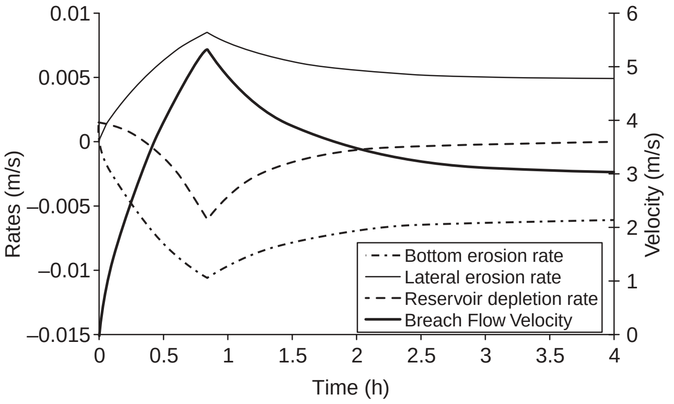

Figure 4 Erosion and depletion rates and average velocity in the breach—example of a RoDaB simulation.

Analysing the outflow hydrographs given by the BREACH model, contrary of what was expected, the peak discharge correspondent to the rockfill dam is higher. The breach cross-section is one of the causes for this discrepancy; in fact the model computes a rectangular breach in the earth dam case while in the rockfill dam it calculates a trapezoidal breach with a larger average width. These unexpected results might be related with the lower internal friction angle of the earth (about a half) and with the inadequacy of the Meyer–Peter–Muller formula, used on the BREACH model, to simulate the sediment transport of large blocks—Graf and Altinakar (1998) refer the value of 28.6 mm as the maximum diameter to which the formula was established. As a consequence, the BREACH model seems to be more appropriate for the simulation of an earth dam breach.

Considering now the maximum discharge obtained for the earth and the rockfill cases (this latter one computed with the RoDaB model), the RoDaB result is half of the peak value given by the BREACH model. The time to peak discharge in the earth dam is about 15% of the time corresponding to the rockfill dam. The results obtained for the rockfill dam rupture correspond to a situation less catastrophic to the downstream valley due to the lower peak discharge and to the more extended time to its occurrence.

Figures 4 and 5 show some typical results obtained with the RoDaB model applied to a rockfill dam breaching.

Figure 4 shows the dependence between the erosion rates and the average flow velocity on the breach. The instant of maximum discharge occurs when the maximum breach flow area occurs. At this moment an inflexion point on the reservoir level curve occurs (Fig. 5), which is in accordance with the changing of the sign of the derivative from the reservoir depletion rate (Fig. 4).

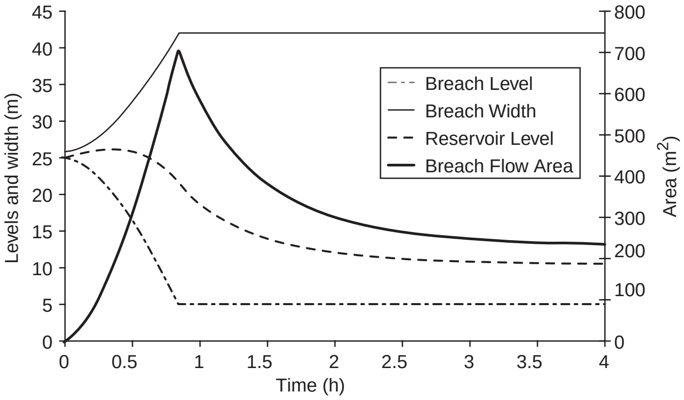

Figure 5 Breach evolution—example of a RoDaB simulation.

6 Conclusions

The formulation here presented represents a simplified lumped method for the numerical simulation of reservoir depletion due to the rockfill dam breaching and the resulting downstream flood hydrograph. As mentioned before, the model is valid within certain simplifications of the reality, that once more are stressed: (i) gradual dam rupture, (ii) seepage induced instabilities during the breach evolution are considered in the experimental behaviour observed in the dam models, and (iii) downstream backwater effects does not influence the rupture process. The empirical results presented in Section 2 are considered in the model to fill the lack of knowledge about the breaching mechanism on rockfill dams.

According to our experiments, and to other studies on rockfill dam failure modes, we believe that seepage is a very important factor that induces the instability and failure of the downstream dam face. Critical shear and erosion seems not to be the key factor in rockfill dams as can be during preliminary rupture phases of earthfill dams. In recent essays (2001–2002), including our set of experiments and some made in Norway and England (Impact Project), the rockfill material is displaced and moves downstream but it is not completely removed as we can see in earthfill dams after rupture. We understand that there is a residual capability of non-structured stabilization that seems to be very important during the failure. This stabilization effect smoothes the breach opening and the outflow hydrograph.

Taking into consideration the mentioned assumptions, the RoDaB model depends mainly on two dam empirical erosion parameters ($C_{\mathrm{s,b}}$ and $\beta_{\mathrm{s,b}}$). The simple dependence of the material transport on the average flow velocity, allows an easy implementation and resolution of the erosion and Exner equations. The authors, based on the approximate indirect calibration process described in this work, suggest that the pair of values for these parameters of the RoDaB model, suitable for rockfill dams, is $C_{\mathrm{s,b}} = 2.0 \times 10^{-3}$ and $\beta_{\mathrm{s,b}} = 1$.

The erosion parameters are characteristic of the dam body and have the advantage to be estimated mostly as a function of the block size of the material. Actually, being the RoDaB model based on these two constants, it is an improvement compared with the models that require the 'knowledge' *a priori* of the dam failure time, since these parameters can be estimated within a reasonable range of values.

Comparisons between RoDaB and BEED models (in terms of the erosion coefficients) and BREACH model (looking at the results to a case study), leads to a belief that the destructive potential of a rockfill dam failure is of less order of magnitude than the one from an earth dam. In fact, a rockfill dam will breach in a different way of an earth dam and the mixed empirical-numerical model developed by the authors is a first step to obtain a practical tool for this type of dam break analysis. RoDaB model background is based on laboratory work and its erosion parameters are calibrated with data from historical cases.

In order to carry out to a more accurate breach model based on the equations presented on this paper, more experimental research is needed in order to clarify the following aspects:

- Flow characterization through the breach, eventually with a time variable discharge coefficient that takes into account the variation of the hydraulic head, the influence of lateral effects and of the displaced rock block effects.
- Better calibration of the erosion parameters based on more rockfill dams.
- Use of a specific equation to calculate the lateral breach erosion based on a simple formulation as the one presented for the bottom erosion.
- Evaluation of the final breach geometry in a more general way, using a dimensionless analysis in order to obtain general relations between the breach dimensions and the dam and rock characteristics.
- Implementation of a routine that verifies the conditions for the occurrence of rockslides due to the seepage and that computes the breach enlargement and its consequences to the outflow discharge due to these ones.
- Direct calibration of the model parameters based on both controlled rock erosion experiments and (or) full-scale rockfill dam failures.

It is believed that this contribution can be a helpful aid for the risk assessment and emergency planning in valleys downstream of rockfill dams.

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Notation

$A_{\mathrm{B}} =$ Breach flow cross area $[\mathrm{L}^2]$
$A_{\mathrm{R}} =$ Surface reservoir area $[\mathrm{L}^2]$
$B =$ Crest width [L]
$C_{\mathrm{B}} =$ Breach discharge coefficient [–]
$C_{\mathrm{C}} =$ Crest discharge coefficient [–]
$C_{\mathrm{s,b}} =$ Erosion coefficient $[\mathrm{L}^{1-\beta}\mathrm{L}^{\beta-1}]$
$D_{50} =$ Average block size diameter [L]
$g =$ Gravity acceleration $[\mathrm{LT}^{-2}]$
$h_{\mathrm{B}} =$ Breach final depth [L]
$h_{\mathrm{B,sim}} =$ Simulated breach final depth [L]
$L'_{\mathrm{C}} =$ Overtopped crest length [L]
$N_{\mathrm{B}} =$ Breach bottom level [L]
$N_{\mathrm{C}} =$ Crest level [L]
$N_{\mathrm{R}} =$ Water level in the reservoir [L]
$p =$ Embankment porosity [–]
$q_{\mathrm{s,b}} =$ Specific solid discharge from the bottom material $[\mathrm{L}^2\mathrm{T}^{-1}]$
$Q_{\mathrm{B}} =$ Discharge through the breach $[\mathrm{L}^3\mathrm{T}^{-1}]$
$Q_{\mathrm{C}} =$ Discharge over the dam crest $[\mathrm{L}^3\mathrm{T}^{-1}]$
$Q_i =$ Flow due to the upstream hydrograph $[\mathrm{L}^3\mathrm{T}^{-1}]$
$Q_{\mathrm{P,B}} =$ Maximum breach discharge $[\mathrm{L}^3\mathrm{T}^{-1}]$
$Q_{\mathrm{P,B,sim}} =$ Simulated maximum breach discharge $[\mathrm{L}^3\mathrm{T}^{-1}]$
$t_{\mathrm{F}} =$ Time of failure [T]
$t_{\mathrm{F,sim}} =$ Time of simulated failure [T]
$t_{\mathrm{P}} =$ Time to peak discharge [T]
$U_{\mathrm{B}} =$ Average flow velocity on the breach $[\mathrm{LT}^{-1}]$
$V_{\mathrm{R}} =$ Volume of the reservoir $[\mathrm{L}^3]$
$W_{\mathrm{B}} =$ Average breach width [L]
$W_{\mathrm{B,sim}} =$ Simulated average breach width [L]
$\alpha =$ Internal friction angle [–]
$\alpha_{\mathrm{s,b}} =$ Erosion coefficient $[\mathrm{L}^{2-\beta}\mathrm{T}^{\beta-1}]$
$\beta_{\mathrm{s,b}} =$ Erosion exponent [–]
$\lambda_{\mathrm{R}} =$ Dynamic reservoir response coefficient $[\mathrm{L}^{-2}]$

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