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STUDY OF DAM-BREAK DUE TO OVERTOPPING OF FOUR SMALL DAMS IN THE CZECH REPUBLIC

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Abstract

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Dam-break due to overtopping is one of the most common types of embankment dam failures. During the floods in August 2002 in the Czech Republic, several small dams collapsed due to overtopping. In this paper, an analysis of the dam break process at the Luh, Velký Bělčický, Melín, and Metelský dams breached during the 2002 flood is presented. Comprehensive identification and analysis of the dam shape, properties of dam material and failure scenarios were carried out after the flood event to assemble data for the calibration of a numerical dam break model. A simple one-dimensional mathematical model was proposed for use in dam breach simulation, and a computer code was compiled. The model was calibrated using the field data mentioned above. Comparison of the erodibility parameters gained from the model showed reasonable agreement with the results of other authors.

Keywords: dam-break, embankment dam, numerical model, dam overtopping, peak discharge

INTRODUCTION

Overtopping is the most common cause of the failure of embankment dams. The statistics for embankment dam failures shows that this failure mode represents approximately 40% of all embankment dam failures (Jandora and Říha, 2008). Even if both large and small embankment dams are not usually designed as overflow dams, it is necessary to take into account the fact that no dam design can ensure absolute protection against overtopping.

During the catastrophic floods in August 2002, which affected central and Western Europe, about 70 small dams were damaged in the Czech Republic. 10 of them were completely breached. During the flood events, the most frequent type of failure affecting small dams was breaching due to overtopping (Říha, 2004, 2006).

When conducting embankment dam breach analysis, the calculation of flow parameters has to be performed. In the case of practical studies, numerical methods are used. The common problem with the modelling of breaching is the estimation of the erodibility parameters of dam materials. The most reliable erodibility parameter values are obtained from the analysis of breaching processes occurring during past incidents. However, the frequency of dam incidents is not so great and thus it is rather difficult to compile statistics for breach parameters. In this paper, the authors contribute to this topic by discussing the failures of four small dams, these being the Luh, Velký Bělčický, Melín, and Metelský dams located in the Blatná region of the Czech Republic (Fig. 3).

In this paper, data from real cases are assembled and analysed. To simulate dam breach due to overtopping a one-dimensional mathematical model was proposed and computer code was compiled and tested. The erodibility parameters were determined via model calibration during which calculated and observed peak outflow discharges and breaching times were compared. The obtained parameter values were compared with previously published data.

State of the Art Review

A lot of publications about dam breaks have been made available since 1990. Singh (1996) presented a list of dam failures and their causes in his monograph. He provided an illustration of the dam breaching mechanism, introduced empirical models with dimensional and dimensionless solutions, mathematical models of dam breaching, and a comparative evaluation of dam breach models.

Wahl (1997) examined empirical procedures and numerical models used for the prediction of dam breach parameters and outlined a program for the development of an improved numerical model for the simulation of embankment dam breach events. Jun and Oh (1998) presented a simulation of dam-break processes during the overtopping of embankments and conducted flood routing analysis along downstream reaches using the NWS DAMBRK model. Good agreement with eyewitness evidence in terms of dam crest overtopping and the progression of breach formation was achieved. Other dam break simulations were presented by Tingsanchali and Chinnarasri (2001), who developed a one-dimensional numerical model for dam failure due to overtopping. In the model, one-dimensional equations of continuity and momentum for unsteady varied flow over steep bed slopes were solved together with sediment transport equations. The model was successfully calibrated and verified using laboratory experimental data. The use of experimental data to calibrate numerical models has been addressed in numerous studies. A set of laboratory experiments were carried out by Aureli *et al.* (2000) to verify a numerical model under test conditions, including shock formation, reverse flow and wetting and drying conditions. Říha and Daněček (2000) compared the results obtained from analytical solution for different breach channel shapes with the results of experiments carried out at an outdoor laboratory. Coleman *et al.* (2002) presented experimental results for small homogeneous embankments made of non-cohesive materials breached by overtopping under constant reservoir level conditions. Chinnarasri *et al.* (2003) investigated the flow patterns and progressive damage at a dike during overtopping after analysing the data obtained from nine experimental runs.

Results obtained from experimental tests were considered by Franca and Almeida (2004) for dam break modelling using two erosion parameters. The final breach geometry and dimensions were obtained from those experiments. Toledo *et al.* (2006) presented the results obtained from tests concerning the failure of rock-fill dams due to overtopping. A similar experimental study of embankment dam breaching was carried out by Dupont *et al.* (2007), and laboratory tests enabled validation of the study and the completion of a numerical approach. An overview of field tests and laboratory experiments was given by Morris *et al.* (2007), where the authors addressed the assessment and reduction of risks from extreme flooding caused by natural events or the failure of dams and flood defence structures. A series of tests investigating dike breaches due to overtopping were carried out by Schmocker and Hager (2009) to examine model limitations. Roger *et al.* (2009) compared data obtained from an experimental model (discharges, water level, and depth profiles of horizontal velocities) with the results of numerical computations of dike-break induced flows with a focus on the final steady state. Soares-Frazao *et al.* (2012) conducted experiments concerning two-dimensional dam-break flows over a sand bed at Université catholique de Louvain, Belgium.

One, two and three dimensional numerical dam break models are currently being developed by authors worldwide. Wang and Bowles (2006) developed an erosion and force equilibrium based three-dimensional model for the simulation of breaching of non-cohesive earthen dams due to overtopping and checked three-dimensional slope stability in the breach channel. Wang *et al.* (2006) developed a physically based numerical model to simulate the growth of a breach in an overtopped earth or rock-fill embankment. A one and two dimensional numerical dam-break flow model is provided by Galoie and Zenz (2011), in which the shallow water equations are solved by means of the finite differences method (FDM).

To obtain an idea about the breaching mechanism, breach opening dimensions and erodibility of soils, the development of a database of real cases and experimental results is necessary. Jun and Oh (1998), Aigner *et al.* (2002), Alcrudo and Mulet (2007) and other authors analysed real cases of embankment dam failures. Lemperiere *et al.* (2006) presented a database of real world case studies of embankment dam incidents and summarized the lessons learnt from dam failures by overtopping to propose a new empirical breach peak outflow formula. Løvoll (2006) presented the results of 3 field tests carried out on 6 m high embankment dams in Norway during the period of 2001–2003. Alcrudo and Mulet (2007) described the event that led to the breach of the Tous Dam in Spain which broke due to overtopping on October 20th 1982 and the effects of the flood. Goran and Goran (2009) presented the results of hydraulic analysis of the failure of two dams in Croatia with a special focus on the first stage of breach formation when water flows through the initial breach and accelerates eroding soil.

From this state of the art review it can be seen that a lot of studies deal with particular problems related to experimental and numerical dam break modelling. However, there is a lack of studies analysing real cases and comparing real incidents with numerical results. This is probably due to poor records available regarding the real circumstances of dam failure. This paper is contributing to the latter topic, discussing carefully compiled records from 4 failures of small embankment dams in the Czech Republic. A simple numerical model was calibrated using the data from the dams' sites.

The paper contains a proposal for a numerical procedure, including conceptual, mathematical and numerical models, descriptions of 4 dam failure cases, and finally a comparison of calculated dam break parameters with site data.

Conceptual Model

In general, the problem of dam failure due to overtopping involves an extremely turbulent three-dimensional flow comprising a mixture of water, air and soil, all with different densities. This fact creates theoretical and numerical difficulties when solving practical problems. Therefore, in this study the problem of dam failure due to overtopping is reduced from a three-dimensional (3D) problem to a one-dimensional (1D) one.

To calculate the breach discharge Q_b, it is necessary to suggest the probable shape of the breach, which is characterized by its shape and area A_b, the water level H in the reservoir and the breach bottom level Z above the reference level (above sea water level SWL). The trapezoidal breach opening is taken into account (Fig. 1). The slope of the breach opening sides is considered as constant during the breaching period.

The erosion capacity of flowing water can be described by empirical relations which express the changes in the breach parameters over time, namely breach bottom level $Z(t)$ and breach bottom width $b(t)$, as a function of water velocity and sediment discharge across the breach. The relations can be formally written as follows (Singh 1996, Jandora and Říha, 2008):

$$\frac{\mathrm{d}Z}{\mathrm{d}t} = f(v, q_s, \alpha_1, \beta_1), \tag{1}$$

$$\frac{\mathrm{d}b}{\mathrm{d}t} = f(v, q_s, \alpha_2, \beta_2), \tag{2}$$

where

v............................the cross-sectional velocity in the breach,

q_s...........................the specific discharge of sediment transport (dam material) and

$\alpha_1, \alpha_2, \beta_1$ and β_2...are empirical parameters expressing the erodibility of dam material.

Mathematical Model

Solving the problem of dam failure due to overtopping involves the determination of time dependent breach outflow discharge $Q_b(t)$. This breach discharge is affected by the dimensions of breach shape, water level and the volume of reservoir storage during the failure.

The unknown variables of the problem are as follows (Fig. 1):

$H(t)$ .....is the water level in the reservoir measured from a reference level,

$Q_b(t)$ ....is breach discharge,

$v(t)$.......is cross-sectional velocity in the breach,

$Z(t)$......is the breach bottom level measured from a reference level,

$b(t)$.......is the breach bottom width.

To determine these variables, the following five equations are used. The instantaneous change in the reservoir storage volume $V(t)$ as a function of the reservoir inflow Q_{in} and the outflow from the reservoir through the breach Q_b, and through appurtenant works Q_f (spillway and bottom outlet), is expressed as follows:

$$\frac{\mathrm{d}V}{\mathrm{d}t} = Q_{in} - Q_b - Q_f. \tag{3}$$

The instantaneous change in the reservoir storage volume can be also expressed as follows:

$$\mathrm{d}V(t) = A_s(H(t))\mathrm{d}H(t), \tag{4}$$

where $A_s(H(t))$ is the area of the water surface of the reservoir determined from the area-elevation curve (bathygraphic curve). When substituting equation (4) for equation (3) the following form is obtained:

$$A_s \frac{\mathrm{d}H}{\mathrm{d}t} = Q_{in} - Q_b - Q_f. \tag{5}$$

We can rearrange equation (5) to yield $\mathrm{d}H(t)$:

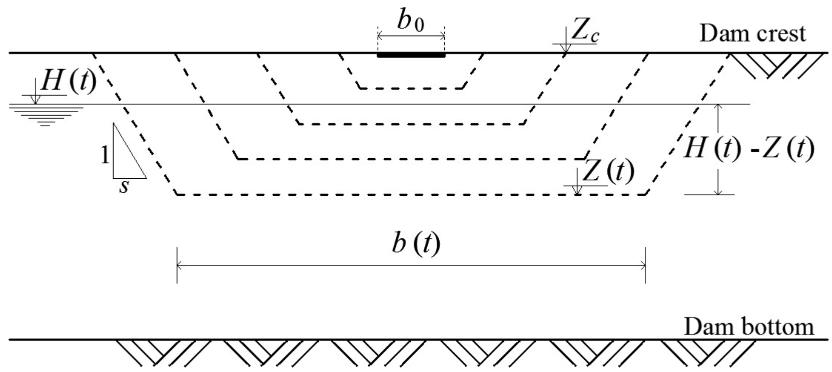
$$\frac{\mathrm{d}H}{\mathrm{d}t} = \frac{1}{A_s}(Q_{in} - Q_b - Q_f). \tag{6}$$

For the breach discharge Q_b, the following equation holds:

$$Q_b = vA_b, \tag{7}$$

where

A_b......the breach cross-sectional area.



1: *Proposed section of dam breach opening*

Breach discharge Q_b (in case of a trapezoidal breach shape) can be determined by the overflow equation for a broad crested weir:

$$Q_b = mb\sqrt{2g}(H-Z)^{3/2} + m_t s(H-Z)^{5/2}, \tag{8}$$

where
m............. the discharge coefficient for a rectangular weir,
m_t............ the discharge coefficient for a triangular weir,
b.............. the breach width at its bottom,
g.............. the gravitational acceleration,
s............... the average slope of the breach sides and,
$(H-Z)$.. the overflow head.

In the case of a trapezoidal breach shape the breach cross-sectional area is defined as:

$$A_b = b(H-Z) + s(H-Z)^2. \tag{9}$$

From equations (7), (8) and (9) we obtain the following equation:

$$v = \frac{Q_b}{A_b} = \frac{mb\sqrt{2g}(H-Z)^{3/2} + m_t s(H-Z)^{5/2}}{b(H-Z)+s(H-Z)^2}. \tag{10}$$

Equations (1) and (2) can be expressed by simple relations (Singh, 1996; Jandora and Říha, 2008):

$$\frac{\mathrm{d}Z}{\mathrm{d}t} = -\alpha_1 v^{\beta_1}, \text{ for } v > v_{cr}, \tag{11}$$

$$\frac{\mathrm{d}b}{\mathrm{d}t} = \alpha_2 v^{\beta_2}, \text{ for } v > v_{cr}, \tag{12}$$

where
$\alpha_1, \alpha_2, \beta_1$ and β_2...empirical erodibility parameters of dam material,
v_{cr} ....the critical value of flow velocity at which erosion of dam material starts (non-scouring velocity).

Initial Conditions

During the solution two significant instants are distinguished (Fig. 2):
$t_0 = 0$....instant of time corresponding to the beginning of dam overtopping,
t_b............instant of time corresponding to the beginning of dam breaching.

The initial conditions of the evolution problem are as follows:

$$\begin{aligned} b(t=t_0) &= b_0, \\ H(t=t_0) &= H_0 = Z_c, \\ Z(t=t_0) &= Z_0 = Z_c, \end{aligned} \tag{13}$$

where
b_0.........the initial breach width,
H_0........the initial water level in the reservoir,
Z_0 ....the initial level of the breach bottom,
Z_c..........the elevation of the dam crest at its lowest place.

The values of initial conditions are determined at the beginning of dam overtopping at time $t = t_0$ and when the non-scouring velocity is exceeded ($t = t_b$).

Numerical Solution

For the approximation of the numerical solution of equations (6), (10), (11) and (12), the Newton method has been used. Δt refers to time step and $t_{i+1} = t_i + \Delta t$ is discrete time.

From equation (6), the water level $H(t_{i+1})$ in the reservoir can be expressed using differences:

$$H(t_{i+1}) = H(t_i) + \frac{\Delta t}{A_s(H(t_i))}\left[Q_{in}(t_i) - Q_b(t_i) - Q_f(t_i)\right]. \tag{14}$$

For the cross-sectional velocity $v(t_i)$ in the breach (Eq. 10), the following equation holds:

$$v(t_i) = \frac{Q_b(t_i)}{A_b(t_i)}, \tag{15}$$

where:

$$Q_b(t_i) = mb(t_i)\sqrt{2g}\left[H(t_i) - Z(t_i)\right]^{3/2} + m_t s\left[H(t_i) - Z(t_i)\right]^{5/2}, \tag{16}$$

$$A_b(t_i) = b(t_i)[H(t_i) - Z(t_i)] + s[H(t_i) - Z(t_i)]^2 \tag{17}$$

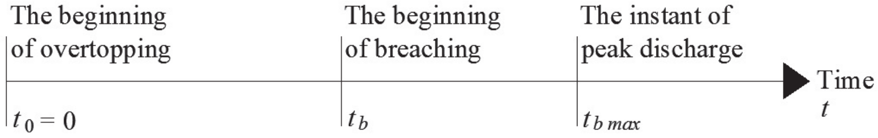
By the finite difference approximation of equations (11) and (12) we obtain:

$$\frac{\mathrm{d}Z}{\mathrm{d}t} \approx \frac{\Delta Z}{\Delta t} = \frac{Z(t_{i+1}) - Z(t_i)}{\Delta t} = -\alpha_1 v^{\beta_1}, \Rightarrow \tag{18}$$

$$Z(t_{i+1}) = Z(t_i) - \Delta t \alpha_1 v(t_i)^{\beta_1}, \text{ for } v(t_i) > v_{cr}, \tag{19}$$

$$\frac{\mathrm{d}b}{\mathrm{d}t} \approx \frac{\Delta b}{\Delta t} = \frac{b(t_{i+1}) - b(t_i)}{\Delta t} = \alpha_2 v^{\beta_2}, \Rightarrow \tag{20}$$

$$b(t_{i+1}) = b(t_i) + \Delta t \alpha_2 v(t_i)^{\beta_2}, \quad \text{for } v(t_i) > v_{cr}. \tag{21}$$



2: *Time axis of the failure process*

CASE STUDY

When calibrating the erodibility parameters of the proposed numerical model, the observed characteristics of real breached dams were used. In the case study, the field data from 4 small dams breached during the flood in 2002 in the Czech Republic were used.

The August 2002 Flood

On August 5th 2002, a cyclone developed above the Western Mediterranean, which during August 6th proceeded to the northeast and reached the eastern Alps. At that time it supplied the south of Bohemia with heavy rainfall featuring local showers of high intensity, which temporarily ceased on August 8th in the morning. On Saturday August 10th, the cyclone regenerated above Italy and continued to the north again. During 11th and 12th August it reached the Czech Republic, where long lasting precipitation struck almost the whole country with daily totals exceeding 200 mm and with a peak daily total of 321 mm. In the area of interest (Fig. 3), the three-day total rainfall (from 11th to 13th August) was about 160 mm. The total rainfall during both rainfall episodes was 305 mm.

Some Details of the Studied Dams

In the area of interest (Blatná region), 4 small dams were breached: the Luh, Velký Bělčický, Melín and Metelský dams. The Luh and Velký Bělčický dams are located about 8 km north of the town of Blatná and about 2 km west of the village of Belcice, both are situated on the Závišínský stream (Fig. 4). The Melín and Metelský dams are located about 12 km to the northwest of Blatná, and situated on the Metelský stream (Fig. 5).

Both the Luh and the Velký Bělčický dams have a heterogeneous body (Fig. 6a) comprising a sandy clay upstream blanket and a downstream shoulder, which in case of the Luh dam is composed of clayey sand, while sand mixed with fine-grained soil has been used for the shoulder at the Velký Bělčický dam (Fig. 7). The Luh dam is equipped with two spillways at the right and left side of the dam and the Velký Bělčický dam is equipped with one spillway at the right abutment.

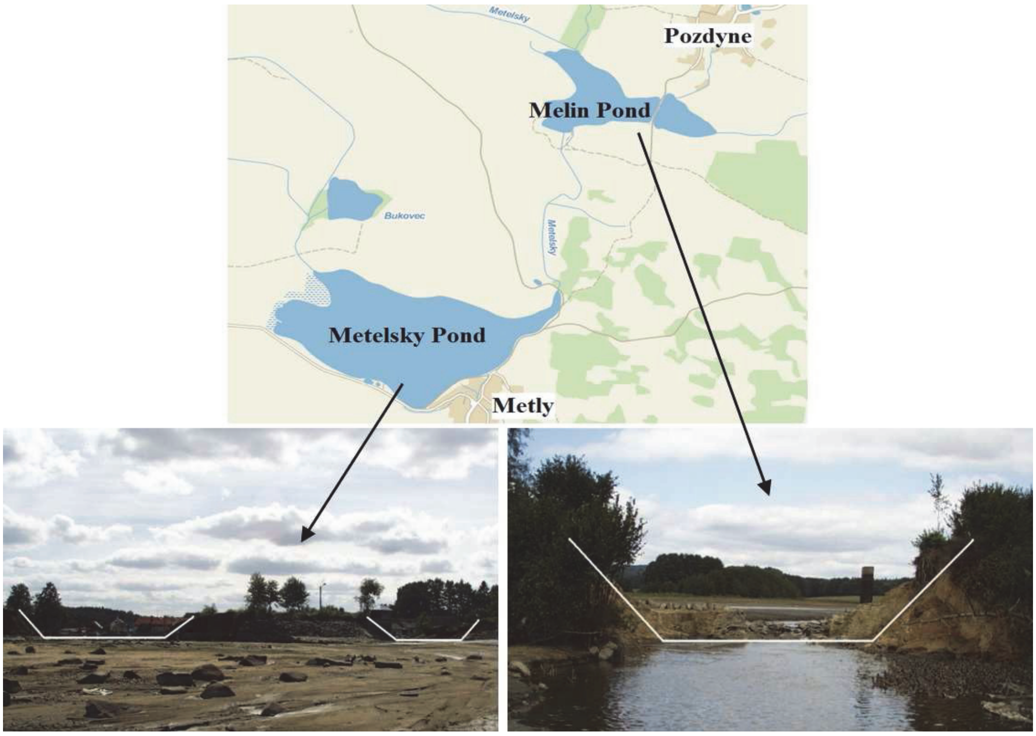
The Melín dam has a practically homogeneous body (Fig. 6b) made of sandy clay (Fig. 8). This dam is equipped with two emergency spillways at the left and right abutments and one wooden bottom outlet with a service shaft located at its upstream end.

3: *Map of the Czech Republic with the area of interest*

I: *Geometric data for the studied dams*

Name of dam	Dam height [m]	Length at the dam crest [m]	Width of the dam crest [m]	Upstream slope	Downstream slope
Luh	3.8	202	7–7.8	1:1.7	1:1.5
Velký Bělčický	6.7	296	7.4	1:1.4	1:2.1
Melín	6.2	195	2.5–3.5	1:1.5	1:1.5
Metelský	8.5	433	4–6.85	1:1.2	1:2

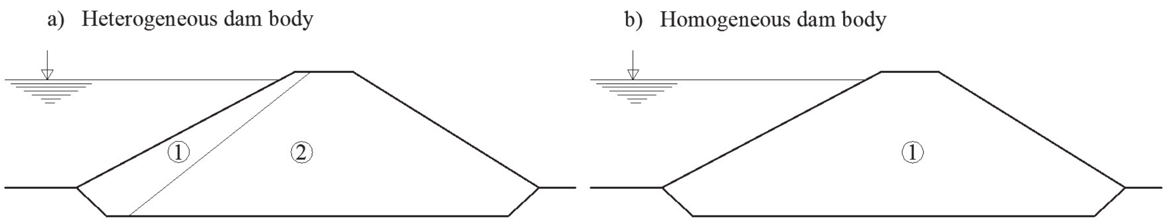
4: *The Luh (left) and Velký Bělčický (right) dam breaches*



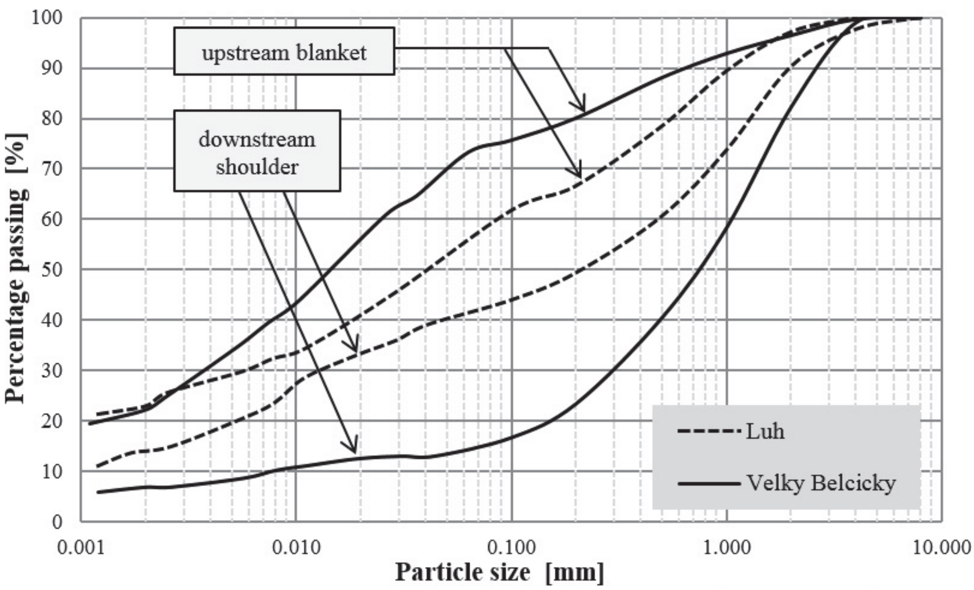
5: *The Metelský (left) and Melín (right) dam breaches*

The Metelský dam has a heterogeneous body (Fig. 6a) comprising a sandy clay upstream blanket and a sandy downstream shoulder (Fig. 8). The dam was equipped with two wooden bottom outlets and an emergency spillway.

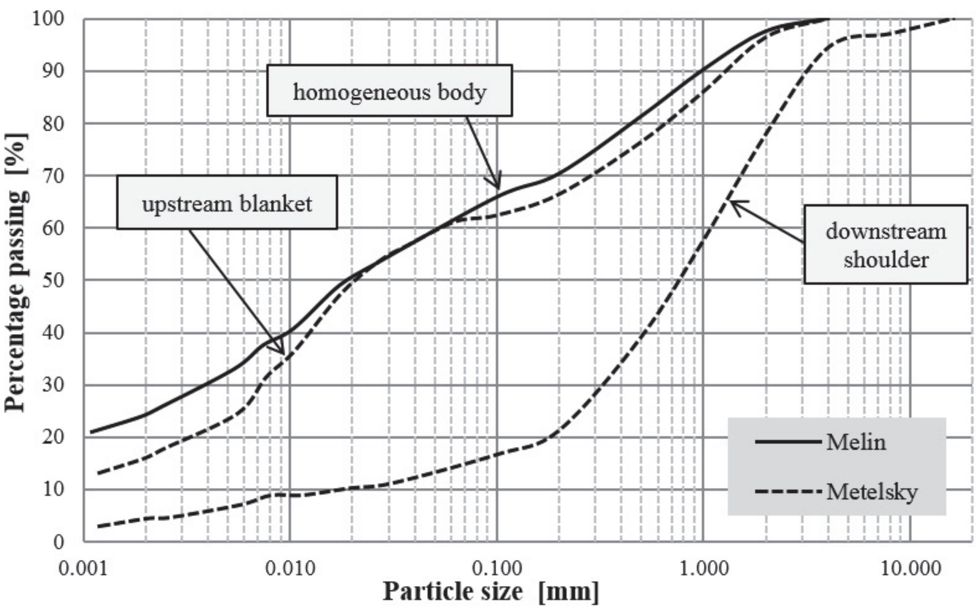
The dam crest of the Luh dam serves as a public asphalt road but the dam crests of the other dams are overgrown with grass and trees. The upstream slopes of all dams are covered with stone pavement and the downstream slopes are covered with grass, bushes and trees (Figs. 4 and 5).



6: *Schematic cross-sections of the dam body*



7: *Grain size distribution curves for the Luh and Velký Bělčický dams*



8: *Grain size distribution curves for the Melín and Metelský dams*

The grain size distribution curves of the above mentioned dams are shown in Figs. 7 and 8. Some further geometric data are summarized in Tab. I.

Method of Solution (Methodology)

Erodibility parameters (α_1, α_2, β_1 and β_2) used in the numerical model were determined based on data obtained from real dams breached due to overtopping, and those parameters were evaluated by comparison the results obtained from the numerical model with the results of model calibration, namely in terms of the maximum breach discharge and instant of peak discharge. For the model calibration the trial-and-error method was used during the performance of backward analysis.

The solution procedure can be illustrated in the following steps:

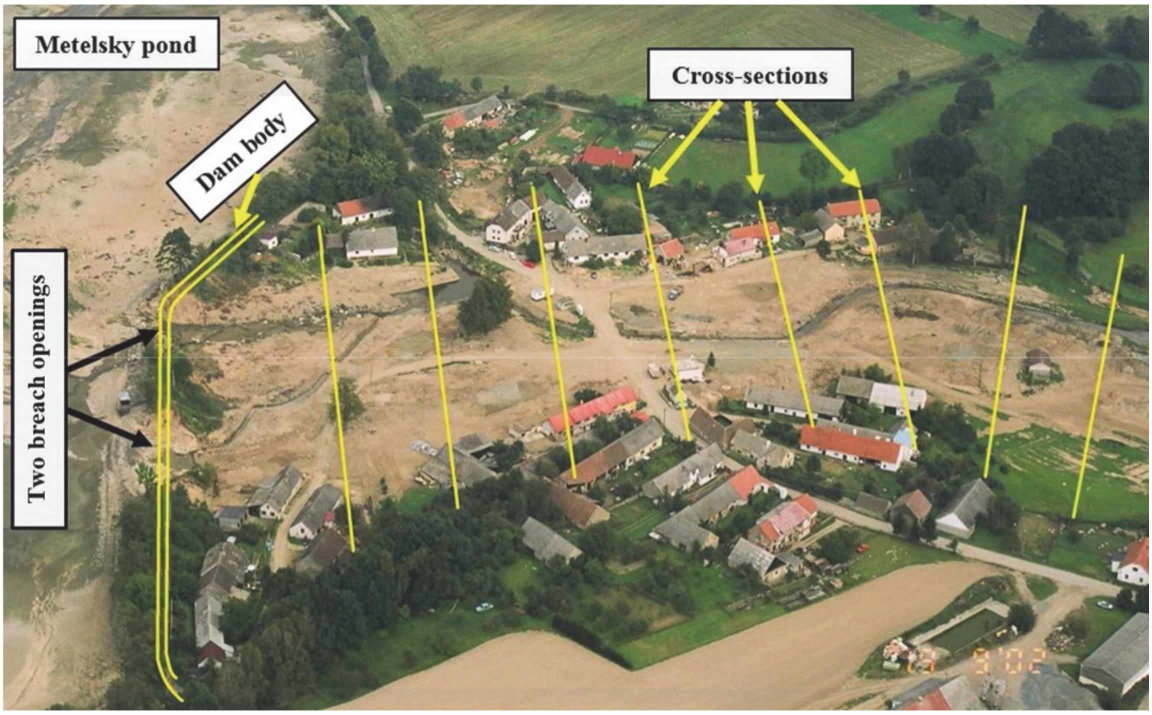
1. Eyewitness testimony provided by local inhabitants was used to reconstruct the flood and dam break events. Based on assembled information, the initial breach width (b_0) was determined together with the instants corresponding to the beginning of dam overtopping (t_0) and the maximum breach discharge (t_{bmax}). These data were used for the model calibration.
2. Site investigations were carried out to determine the final breach bottom width (b) and the average slope of the breach sides (s) by measuring the breach opening geometry from which the maximum breach discharge ($Q_{b\max}$) was estimated using Eq. (8). The traces left by the water level in the downstream valley, on the trees and houses downstream of the dams were indicated (Fig. 9). These data enabled reliable estimation of the maximum breach discharges during the events (see next step).
3. The maximum breach discharge was determined by comparing the value determined from Eq. (8) based on the geometry of the breach opening (Figs. 4, 5) with the value obtained from hydraulic calculations in the valley downstream of the breached dam (Fig. 9). HEC-RAS software was used.
4. The inflow into the reservoirs $Q_{in}(t)$ was determined using the rainfall-runoff model with the use of information about the temporal progression of precipitation during the flood event (provided by the Czech Hydrometeorological Institute). At the same time, the necessary hydraulic calculations were carried out for appurtenant works (bottom outlets, spillways) of the mentioned dams to obtain $Q_f(H(t))$.
5. The event was reconstructed by the numerical model described above. During the model calibration, the hydraulic and erodibility parameters were derived via a trial-and-error procedure.

RESULTS AND DISCUSSION

As mentioned above, flow and erodibility parameters were obtained from the model calibration. The outputs from the calculations were dam break hydrographs for all 4 dams.

Tab. II summarizes the values of initial breach width (b_0), the average slope (s) of the breach opening sides, the discharge coefficients (m and m_t) and erodibility parameters (α_1, α_2, β_1 and β_2) used in the calibrated numerical model.

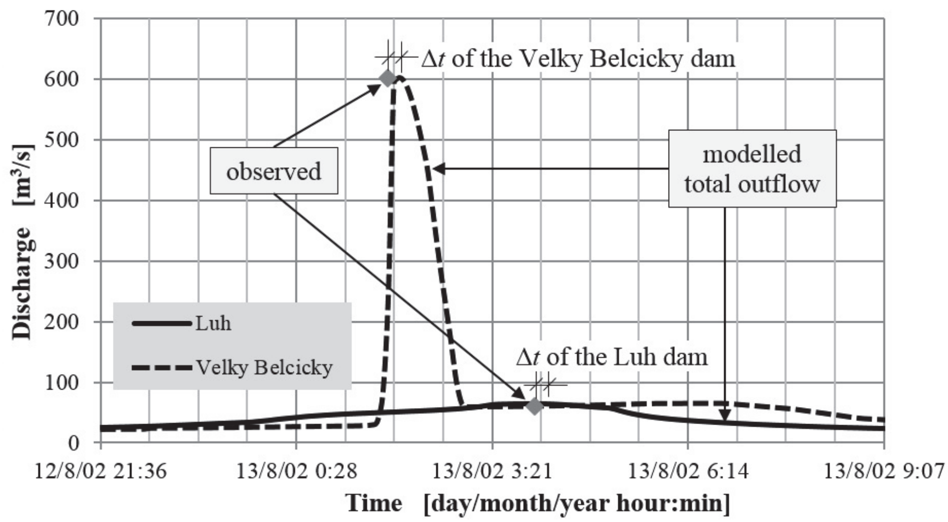
The final results are presented in Figs. 10 and 11, where the modelled peak discharges of the breached dams are compared with values obtained from site investigations, and the time difference between the



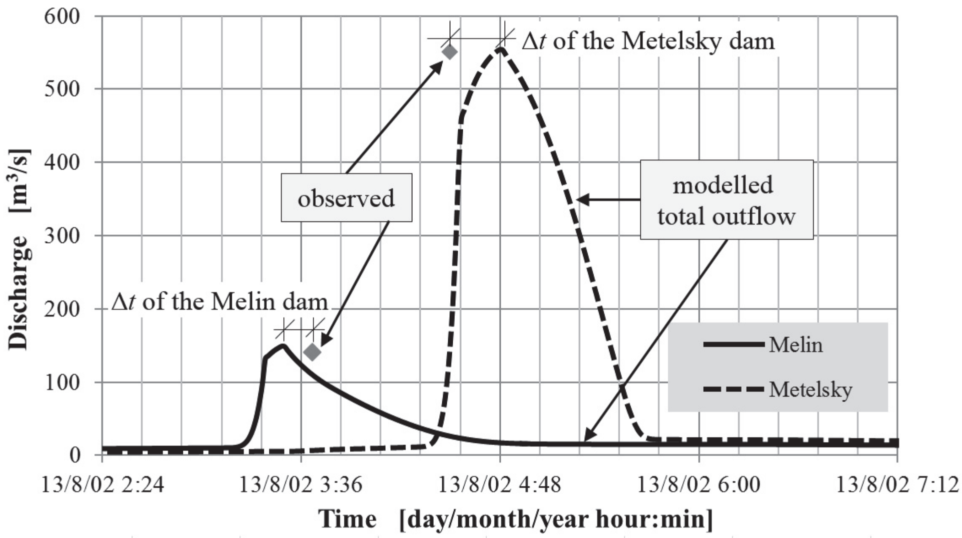
9: *Area below the Metelský dam after the flood*

II: *Input data for the mathematical model (numerical solution)*

Name of dam	b_0 [m]	s	m [-]	m_t [-]	α_1	α_2	β_1	β_2
Luh	3	1.00	0.33	0.35	0.0001	0.001	1	1
Velký Bělčický	12	0.80	0.35	0.35	0.003	0.003	1	1
Melín	1	0.80	0.35	0.35	0.003	0.003	1	1
Metelský	1	0.95	0.35	0.35	0.003	0.003	1	1



10: *Flood hydrograph and model calibration for the Luh and Velký Bělčický dam failures*



11: *Flood hydrograph and model calibration for the Melín and Metelský dam failures*

III: *Results of model calibration and the mathematical model*

Name of dam	Average breach width [m]		Q_{bmax} [m³/s]	Q_{bmax} [m³/s]	Δt *
	observed	calculated	observed	calculated	[hour:min]
Luh	17	15	58	64.1	0:05
Velký Bělčický	42	38	600	602.1	0:10
Melín	17	9.23	150	149	0:10
Metelský	42 + 30 **	35 + 15 **	550	554	0:15

* Time difference between the peak discharges (Figs. 10 and 11).
** Two breach openings (Fig. 5, left).

modelled and observed peak discharges are stated. The obtained values are listed in Tab. III.

The results (Tab. II) show that the breach opening slopes are not far from $s = 1$. This corresponds to the results described in older literature sources such as ICOLD (1974), USACE (1980) and DSG (1992). However, experimental results show that during breaching slopes can be significantly steeper, especially in the case of cohesive materials. The slopes close to $s = 1$ reflect the state after breaching when the side slopes of a breach opening gradually slide due to their instability, together with weather conditions, namely rain, etc.

Model calibration provides approximately identical discharge coefficients during breaching for all of the studied dams. The value $m = 0.35$ corresponds to the broad crested weir and is also reported by other authors (Singh, 1996). The slightly smaller coefficient of $m = 0.33$ at the Luh dam resulted from the upstream bottom outlet tower remaining in the centre of the breach opening.

The values of erodibility parameters (α_1, α_2, β_1 and β_2) are governed dominantly by the properties of the upstream blanket materials (Wahl 1998), which influence the rate of dam crest decreasing due to erosion, see Eq. (11). The upstream clayey material does not differ significantly in the case of the Velký Bělčický, Melín (homogeneous dam body) and Metelský dams, so the erodibility parameters $\alpha_1 = \alpha_2 = 0.003$ are valid for all 3 dams. The significantly lower value $\alpha_1 = 0.0001$ for the Luh dam is attributed to the effect of the asphalt road on the dam crest, which produces much higher resistance (Wahl, 1998). This situation at the Luh dam crest can be seen in Fig. 12. The values $\beta_1 = \beta_2 = 1$ indicate a linear relation where according to Eq. (11) and (12) the breach opening size increases proportionally with the cross- sectional velocity in the breach.

Generally, the values of erodibility parameter α_1 fit the range of values $\alpha_1 \in (0.001; 0.008)$ referenced by Singh (1996) for embankment dams of various types. Especially dams of similar parameters exhibit relatively good agreement with the values in Tab. II:

- Frankfurt, Germany, height 10 m, reservoir volume 350 000 m^3, $\alpha_1 = 0.001$,
- Kelly Barnes, USA, height 11.5 m, reservoir volume 505 000 m^3, $\alpha_1 = 0.005$,
- Lake Latonka, USA, height 13 m, reservoir volume 1 590 000 m^3, $\alpha_1 = 0.001$.

12: *Higher resistance of asphaltic pavement on the dam crest*

CONCLUSION

In this paper an analysis of the dam break process that occurred at 4 small dams is described. Comprehensive site investigations, measurements of the breached dams' parameters and analyses of the dams' properties and failure scenarios were carried out.

The peak outflow discharges were estimated by backward analysis by comparing the outflows through the breach openings obtained from Eq. (8) and the flows downstream from the dams using HEC-RAS software and the data collected from site investigations and geodetic measurements.

A simple one-dimensional numerical model for dam break simulation was developed and computer code was compiled to simulate the dam breaching process. A trial-and-error calibration procedure was used for the determination of the parameters of the numerical model. Quite good agreement was reached between observed and measured final average breach width and the maximum breach outflow from the dam during its collapse. A relatively greater difference was found between identified and calculated peak outflow times; this difference resulted from uncertainties in witness testimonies and also the rainfall-runoff process and flood routing through the reservoirs.

The model parameters (m, α_1 and α_2) obtained from backward analysis agreed quite well with data published by other authors. The differences (namely in erodibility parameters) can be explained by the differences in dam types, dimensions and materials, and also by general uncertainties in the estimation of breach parameters in the field.

The mathematical model and the parameters can be used for simulations of the breaching process of small embankment dams with similar structures and materials. For other dams the model can be used with special care and attention. Comparison with a wider range of real cases (Singh, 1996) is strongly recommended. The uncertainties in modelling should be dealt with by performing sensitivity and probability analyses using e.g. random sampling (Jandora, Říha, 2008).

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List of symbols

A_b	breach cross-sectional area	[m^2]
A_s	water surface area of the reservoir	[m^2]
b	breach width at its bottom	[m]
b_0	initial breach width	[m]
g	gravitational acceleration	[m.s^{-2}]
H	water level in the reservoir	[m]
H_0	initial water level in the reservoir	[m]
m	discharge coefficient for a rectangular weir	[-]
m_t	discharge coefficient for a triangular weir	[-]
Q_b	breach discharge	[m^3.s^{-1}]
Q_{bmax}	maximum breach discharge	[m^3.s^{-1}]
Q_f	outflow from the reservoir through appurtenant works	[m^3.s^{-1}]
Q_{in}	reservoir inflow	[m^3.s^{-1}]
q_s	specific discharge of sediment transport	[m^2.s^{-1}]
s	average slope of the breach sides	[-]
t	time	[s]
t_0	instant corresponding to the beginning of dam overtopping	[s]
t_b	instant corresponding to the beginning of dam breaching	[s]
t_{bmax}	instant corresponding to the maximum breach discharge	[-]
V	volume of the reservoir storage	[m^3]
v	cross-sectional velocity in the breach	[m.s^{-1}]
v_{cr}	critical value of flow velocity at which erosion of dam material starts	[m.s^{-1}]
Z	breach bottom level	[m]
Z_0	initial level of breach bottom	[m]
Z_c	elevation of dam crest	[m]
α_i, β_i	empirical erodibility parameters of dam material	[-]

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