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The Washout Mechanism of the Niedów Dam and its Impact on the Parameters of the Flood Wave

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Abstract

On 07 August 2010 a disaster occurred at the barrage Niedów located on the river Witka as a result of unprecedented rainfall in the Nysa Luzycka catchment area. The flood wave swept through the dam crest destroying almost the whole structure within a few dozen minutes. The only element that did not vanish was the concrete structure of the spillway and water-power station. The process of dam breach is not so far completely resolved and reliable predictive models of dam breaching are still challenging problem. Modelling of the process requires an evaluation of parameters affecting this phenomenon, namely, the breach size and the timescales for the breach formation. The latter is particularly important for characterizing the outflow hydrograph and, consequently, for the assessment of the damage risk of areas below the dam. The paper presents a mechanism of breaching of the Niedów dam which consists of three stages: the downstream slope wash out, the breaking down of the dam crest and the damaging of upstream slope revetments. The comparison of breach parameters with that evaluated based on several empirical formulas and numerical modelling is also presented.

Keywords: earth-fill dam; overtopped earth dam; dam failure; dam breach; numerical model.

1. Introduction

Studying the security of a dam consists in e.g. identifying the consequences of a dam breach which can be very detrimental to human lives and property on the area flooded with water from the reservoir. In the case of earth dams, the main cause of an earth dam failure is the overflowing of water above the crest of the dam or piping caused by concentrated seepage through the embankments leading to the washout of the dam body. The dam body washout mechanism has not been fully investigated yet because it depends on many variable features including soil characteristics, soil composition, and protective soil covers on slopes and crest in the form of vegetation, rock riprap, or hardened surfaces of concrete or asphalt. Evaluating the breach mechanism provides an insight into the evolution

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of the breach and calculations concerning the wave flowing from the breached reservoir. It has fundamental meaning for estimation of the risk of damage to the land downstream the dam and for development of evacuation plans.

Essentially, 3 groups of methods are used to predict dam breaching parameters and the volume of peak discharge from the breach. The first group involves empirical formulas based on case study data. Mac Donald and Langridge-Monopolis [1], Bureau of Reclamation [2], Froehlich [3, 4], Washington State [5], Xu & Zhang [6] and others worked to develop empirical formulas using multiple regression equation to determine such breaching parameters as breach depth, average breach width, failure time and dam breach peak discharge. The second group involves physical modelling based on sediment transport formulas. The existing 1D and 3D analytical and numerical models of dam breach and break were studied and presented by Murthy [7], Froehlich [3], Faeh [8], Pontillo et al [9], Wang and Bowles [10], Macchione [11] and others. The third group includes other methods such as application of the neural network algorithms by Nourani [12] and genetic algorithms by Sattar [13] for deriving an empirical formula.

The basic and indispensable component of all experimental studies and numerical simulations is validation of the same based on historical events. Hence, it is so important to identify the mechanism of breaches that actually occurred. This study aims to present progression of the breach of the Niedów dam on the Witka River, south-west Poland that occurred on 07.08.2010. It is a unique earth dam failure during which two breaches were formed: one on the left and one on the right side of the spillway structure. Based on photos and movies, 3 phases of the dam washout were identified and the flow wave from the reservoir was simulated using these inputs. Then, the data were compared to values calculated with empirical formulas, identifying the differences and their causes. Finally, calculations of the hydrograph of the flood wave were performed for the actual washout process data. The spatial coverage and depth of the flooding provided validation for the considerations and calculations performed.

1.1. Niedów reservoir and dam description

The Niedów reservoir on the Witka River in south-west Poland, near the Polish-Czech and Polish-German borders was built in 1962. It was designed to supply water to the Turów thermal power station and the town of Bogatynia. The capacity of the reservoir before the failure was 5.6 million m3 and the surface of the floor 183 hectares. The dam was formed from soil extracted from the reservoir bowl, mainly gravels and sand-gravel mixes. It was 16.70 m high at the tallest point. The dam consisted of 148.0 m long left part and 100.0 m right part separated with the concrete structure of spillway closed with segmented gates, bottom outlet and a hydropower plant. See Fig.1 for the cross-section of the dam.

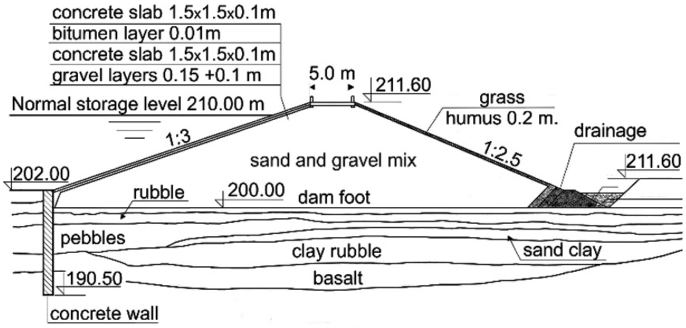


Fig. 1. Cross-section of the Niedów dam.

1.2. Breach description

On August 7, 2010, a rain unprecedented within several decades fell in the catchment of the Witka River (Czech: Smeda) on the territory of the Czech Republic and produced a sudden freshet. The flood wave flew over the crest of the dam, washed out its body and produced water outflow from the reservoir approx. 1,300 $m^3 \cdot s^{-1}$.

The body washout mechanism was as follows:

- sod tear-off;

- downstream body side washout;
- devastation of the asphalt road on the crest;
- gradual devastation of the overfall concrete slabs;
- destabilization of the retaining wall on the left side of spillway;
- complete body washout and reservoir emptying.

The washout progressed through 3 phases featuring different breach development conditions. In the 1st phase, water flew over the crest and the downstream slope with high velocity. Within minutes water tore off the compact layer of sod, starting from the weakest spots around the light posts, and the washout of the dam body followed (Fig. 2a).

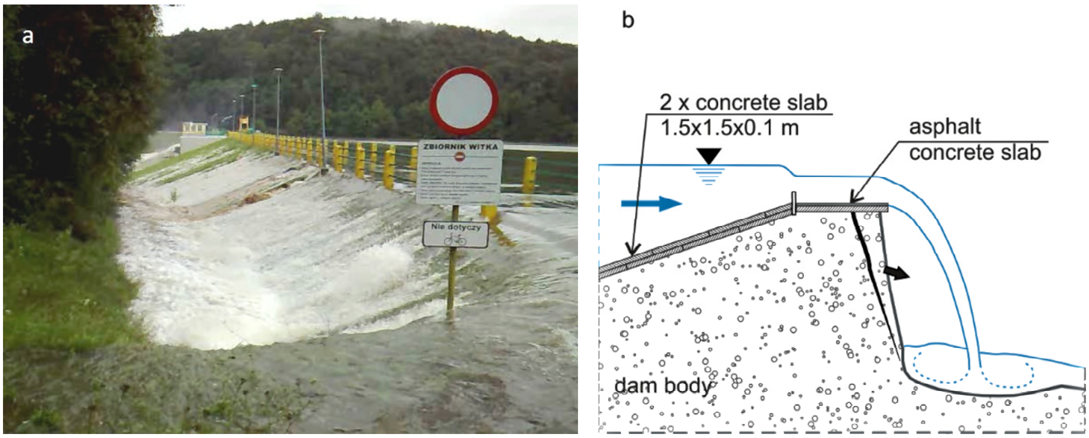


Fig. 2. (a) Phase 1: time 5:10 PM, water level H = 211.60 m a.s.l.; (b) Mechanism of phase 2 of the slope washout.

Phase 2 devastated the dam body from the downstream side. Water falling from the crest with high velocity hollowed the body, forming the characteristic breach shown in Figs. 2b) and 3a). The growth of the breach destabilized body, which collapsed into the gap and were washed out and down the river. Within approx. 25 minutes the concrete crest lost its base, with different rates at various cross-section points, and then broke down, collapsed and was washed out, which triggered the next washout phase (Fig. 3b).

Fig. 3. Phase 2: (a) right side of the dam, 5:42 PM, water level H = 212.00 m a.s.l., (b) left side of the dam, 5:56 PM, H = 211.60 m a.s.l.

Phase 3 took from approx. 5:35 PM to the complete drainage of the reservoir as the concrete slabs of the upstream slope resisted longer. The upstream slope had a two layers of slabs, the top ones offset by 0.5 of their widths and heights relative to the bottom slabs, bound together with bituminous sealant. This made the concrete cover of slope more rigid and resistant to overturning by the force of water. Once the next slice of the body was washed out, the slabs broke one horizontal row after another and were washed down by water (Fig. 4a). Given the progressing decline of the dam height and high water velocity on the overfall, the range of the stream grew, which significantly extended the duration of the slab bottom washout process (Fig. 4b).

Fig.4. Phase 3 of the right side: (a) 6:22 PM, water level H = 210.70 m a.s.l.; (b) 7:45 PM, right side, empty reservoir.

Collapse of the retaining wall of the spillway was critical for the left dam. Once embankment close to the wall was washed out, the anchoring of the retaining wall in the foundation (concrete cut-off wall) and in the abutment was insufficient to provide stability. The retaining wall was overturned and washed down. The collapse of this wall resulted in a significant growth of the outflow rate on the left side of the spillway, which caused complete devastation of the left side of the dam. The left side washout process ended at 6:30 PM. Starting from the collapse of the retaining wall, water level in the reservoir dropped and the outflow ended at 7:30 PM. See Table 1 for the duration of each phase.

Table 1. Washout time of left and right side of the dam in each phase.

Earth dam	Left side	Right side
Time (minutes), phases 1 + 2 + 3 =	5+20+45=70	5+25+105=135

2. Breach data based on empirical formulas

Typically, empirical methods determine the breach data (average depth and width, slope inclination, development time and peak outflow) based on the dam shape, water surface elevation, reservoir capacity and the soil type. Three formulas for the determination of washout of a non-cohesive earthen dam caused by crest over-flowing – developed by McDonald & Langridge-Monopolis [1] (1), Washington State [5] (2), Froelich [4] (3), recommended by the Colorado State [14] – and the formulas for concrete faced dams built from non-cohesive soil, the design corresponding to that of the Niedów dam (4), proposed by Xu and Zhang [6] were used for comparing the results of empirical calculations to the actual values for the Niedów dam:

$$V_{er} = 0.026\left(H_w V_w\right)^{0.77};\; B_{avg} = V_{er}\big/\left(H_b W_{avg}\right);\; Z_b = 0.5;\; T_f = 0.0176 V_{er}^{\;0.364} \tag{1}$$

$$V_{er} = 0.30\left(H_w V_w\right)^{0.77};\; B_{avg} = V_{er}\big/\left(H_b W_{avg}\right);\; T_f = 0.022 V_{er}^{\;0.36} \tag{2}$$

$$B_{avg} = 0.27 k_0 V_w^{\;0.32} H_b^{0.04};\; Z_b = 1;\; T_f = 63.2\left(V_w \big/ g H_b^{\;2}\right)^{0.5} \quad k_0 = 1.3 \; \textit{for overtopping} \tag{3}$$

$$\frac{B_t}{H_b} = 1.062\alpha^{0.092}\beta^{0.508}e^{0.798}\ ;\ \frac{B_{avg}}{H_b} = 0.787\alpha^{0.133}\beta^{0.652}e^{0.466}\ ;\ \frac{T_f}{T_r} = 0.304\alpha^{0.707}\beta^{1.228}e^{-2.458}\ ;\ \alpha = \frac{H_d}{H_r}\ ;\ \beta = \frac{V_w^{\ 1/3}}{H_w} \quad (4)$$

Where: H_b is the breach height (m), H_w is the maximum water depth upstream the dam (m), V_w is the water volume in the reservoir (m^3) corresponding to depth H_w, V_{er} is the eroded capacity of the dam (m^3), B_{avg} is the average breach width (m), W_{avg} is the dam body width at the center of its height (m), Z_b is the breach slope inclination and T_f is the breach development time (h). The breach data for the Niedów dam were calculated based on foregoing formulas and the following inputs: H_w = 10.00 m, W_{avg} = 37.0 m, V_w = 7,000,000 m^3, H_b = 9.60 m. See Table 2 for the results

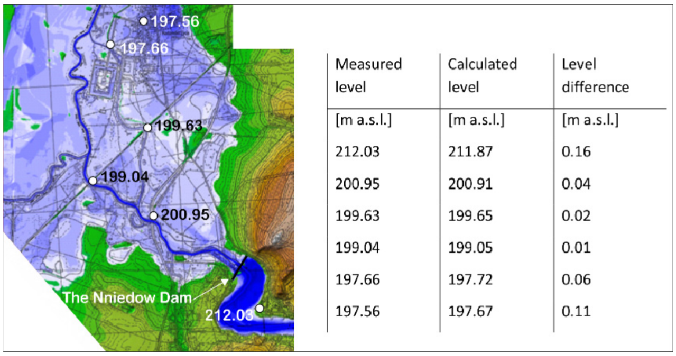
Table 2. Niedów dam breach data calculated using empirical formulas.

Washout data	McDonald & Langridge-Monopolis (1)	Washington State (2)	Froelich (3)	Xu & Zhang (4)	Niedów dam	
					Left	Right
V_{er} (m^3)	28512	32758	24090	35024	24001	23135
B_{avg} (m)	71	81	60	87	122	58
Z_b (-)	0.5	-	1	2	-	-
T_f (h)	0.74	0.97	1.40	0.76	1.17	2.25

The empirical calculations authors did not use formulas for the determination of the outflow wave peak as they were unable to identify the actual outflow rate for the left and right sides of the dam. This is a consequence of the collapse of the retaining wall of the left dam and much higher percentage of outflow on this side. In addition, water flew out through the open spillway gates. The foregoing table shows that, for the Niedów dam breach, the formulas of Froelich provide the best estimation of the eroded material quantity and breach width and development time. However, there are quite a lot of errors, which results from the formation of two breaches of the dam and the features of the dam washout mechanism. Given the resistance of the concrete slabs, the washout time of the right dam is approx. 40 % longer and the total width of the gaps is approx. twice as large the value calculated with the empirical formulas. Also the total volume of eroded material is approx. 100% larger than the output of the foregoing formulas. The complex nature of the dam destruction process made it very difficult to estimate the volume of outflow from the breach. Therefore, the authors of the study decided to use numerical modelling of the outflow based on the actual breach development data.

3. Numerical calculation of the outflow rate

Calculations of the results of the dam failure in the form of the wave reach time and flooding height are typically performed in 2 steps, the first one consisting in calculation of the outflow wave hydrograph using the numerical or empirical method, the other one consisting in a procedure of transforming the wave in the river valley. Software packages such as Boss Dambreak, Telemac, Mike 11 or Mike 21 are generally used for this purpose. In addition, it is common to use the Mike 11 or HecRAS application to transform the wave. For the Niedów dam failure, the MIKE 11 application by DHI was used for modelling the outflow from the breach. For calculating the shape of breach, initial and final breach widths and levels are required to be specified. The linear or erosion-based failure mechanisms can be used. As the latter one fails to address concrete faced dams, the authors decided to perform calculations for the linear damage mechanisms based on the actual data on the geometry and timing of the breach development. Using MIKE 11 authors performed calculations of the outflow hydrograph due to the dam breach, flow of the flood wave through the downstream valley to get the water level in time and the flood zone on the base of digital terrain model. Because the breaches on the left and right sides of the spillway structure developed at different rates, difficult to represent in the MIKE 11 application, the authors decided to model the Niedów dam failure as a single averaged process, considering the outflow through the spillway. This approach provided very similar outflow wave shape and size values to the actual ones. The model was validated against the water surface levels measured during the flood downstream the dam. Fig. 6 presents the calculated flood range below the dam and points where modelling accuracy was checked. The modelled peak discharge rate below the dam was 1,355 $m^3 \cdot s^{-1}$.



Measured level	Calculated level	Level difference
[m a.s.l.]	[m a.s.l.]	[m a.s.l.]
212.03	211.87	0.16
200.95	200.91	0.04
199.63	199.65	0.02
199.04	199.05	0.01
197.66	197.72	0.06
197.56	197.67	0.11

Fig. 5. Simulated maximum flooding area downstream the Niedów dam on 07.08.2010, 7:10:00 PM. Points with measured water surface elevation marked and the calculated and measured water elevation compared.

4. Conclusions

The process of the Niedów dam destruction characterized by variable dynamics of breach formation. There were 3 phases of the process: (1) grass cover washout and initiation of downstream slope destruction, (2) total devastation of the downstream part of the dam body and the start of destruction of top of the dam, (3) total destruction of the asphalt road on the crest and concrete slabs of the upstream slope leading to the washout of the dam. The process of dam breach development shows its nonlinearity in cascades due to the combined effects of scouring and concrete slope collapse.

When the soil composition of dam is significantly inhomogeneous and these parameters will vary greatly in space, especially when the dam slopes are covered by concrete or asphalt, better results in solution of breaching evolution parameters are on the basis of the empirical formulas, then the numerical simulation. The numerical dam breach models used assume homogeneity of the dam material and constancy of the initial conditions of the process. They also require that the final state of the breach is defined. Therefore, they are not relevant for most of dam designs for which the dam body erosion processes are typically strongly dependent on the structural components of the dam, which are not covered by mathematical models.

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