Dam Break Flood Analysis for small Dams, Example of Use – Problems and Approaches

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Abstract

In this case study a dam-break flood analysis, according to the ICOLD Bulletin 111, for a dam with the height of 10 m and with a volume of 36.000m³ is performed.
Thereby, the following three scenarios are investigated: overtopping, sabotage and piping. In the present study the authors decided to use a computer program, which was developed and tested during the EC Research Project (Project Reference No. EVG1-CT2001-00037): “Investigation of extreme flood processes & uncertainty”, to calculate a temporal high-resolution dam failure hydrograph. Several soil parameters, which were determined during the dam erection, are used as input parameters for the computer program.
With the dam failure hydrograph as an input, the flood wave propagation is calculated with the 2D numerical flow simulation program Hydro_As_2d. Based on airborne laser scanning data (1 point/m²) and land use data, which is used to estimate the specific roughness, the terrain model is built.
The result of the present study is the chronology of the flood wave propagation with water depths and velocities. Based on this, the hazard/emergency planning for this dam on the owner’s side and for the municipality involved is adapted.

Introduction

Based on the size of the dams, different hazard evaluations have to be carried out in Austria. For example, for dams taller than 15 m or dams with a volume of more than 500.000 m³, flood wave propagations have to be accomplished. In this case study, a dam-break flood analysis, according to the ICOLD Bulletin 111, for a dam with the height of 10 m and with a volume of 36.000m³ is carried out due to this dam’s hazard potential.
The dam is used as an equalizing reservoir. The catchment area is very small; thereby the only inflow to the dam is the headrace tunnel, which has the same capacity as the outflow (pressure tunnel – turbines) or even the spillway. During an inspection of the reservoir a bypass is used to operate the hydropower plant. The dam is an embankment dam with impervious coating on the upstream side of the dam.

Operating Modes and breakdowns

The operating mode 1 is the normal case. The water flows through the headrace tunnel into the reservoir and further into the pressure tunnel to the turbines.
During an inspection of the reservoir (operating mode 2) the water, which comes out of the headrace tunnel, is directed into a bypass to the headrace tunnel and to the turbines.
If there is a marginal increase of the water level (e.g. during a break down of the turbines), the whole amount of water which is transported through the headrace tunnel, can run off through the spillway without endangering the dam crest.
If there is any break down of the spillway, the water runs through the pressure tunnel or alternative through the bypass.

Dam failure hydrograph

According to the ICOLD Bulletin 111 [1], the following three scenarios have to be investigated: overtopping, sabotage, and piping. As mentioned before, for this dam overtopping is nearly impossible because in case of an emergency one outflow possibility is always available.
After consultation ATCOLD, sabotage can be ignored because a large amount of explosives is required; thus, the transport of these explosives to the dam and the deposit of the explosives at the dam will be recognized.
Consequently, piping is the only threat left. Thereby, according to the ICOLD Bullentin 111, the peak value of the flood wave can be estimated very well using different formulas (e.g. Froehlich, Costa, or Molinaro). In addition, using these formulas, the moment of the peak value can be calculated as well.
First of all in the numerical model for piping is an initial pipe due to internal erosion (a) which increases (Figure 1). When the roof above the pipe collapses (b) the dam erosion process proceeds like overtopping the dam (c)

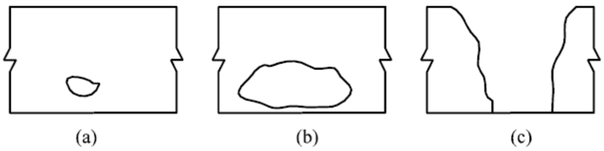

Figure 1: Collapse of the dam due to piping [2]

In literature can be found several different parameter models to calculate the dam failure hydrograph, e.g.: THIRRIOT (1975), the program BREACH by FREAD (1988), the program BEED by SINGH/SCARLATOS (1989), the program by VISSER (1998), and the program by HASSAN (2002) as well as the program DEICH by BROICH. The differences between the mentioned parameter models are the used sediment transport formulas and the breakage development. Strictly speaking, all models can only be used for non-cohesive material but due to the lack of alternatives, these formulas are also used for cohesive material. The influence of the cohesion is represented in the modified breach developing process represented [2].

In the present study, the authors decided to use the computer program DEICH 1.15, which was developed and tested during the EC Research Project (Project Reference No. EVG1-CT2001-00037): “Investigation of extreme flood processes & uncertainty”, to calculate a temporal high-resolution dam failure hydrograph.

Several soil parameters, which were determined during the dam erection, are used as input parameters for the program. For the initial condition, the amount of water in the basin and the available amount of water in the headrace tunnel have to be estimated. In addition, a safety margin for a delayed closing of a gate (18 minutes) is added. Altogether 106.000 m³ of water are used in the calculation.

The program calculates the dam erosion process and based on this, the dam failure hydrograph is computed. The initial pipe is widened within 200 s so that 121 m³/s water leaves the dam (Figure 2). After 300 s the dam above the pipe collapses and due to that there is another additional peak (118 m³/s) in the dam failure hydrograph. Further, the dam failure hydrograph gets more complanate up to 32,5 m³/s which is the capacity of the headrace tunnel. After 40 minutes the whole reservoir and the headrace tunnel are empty. The time spread up to the initial pipe cannot be estimated in the current calculation.

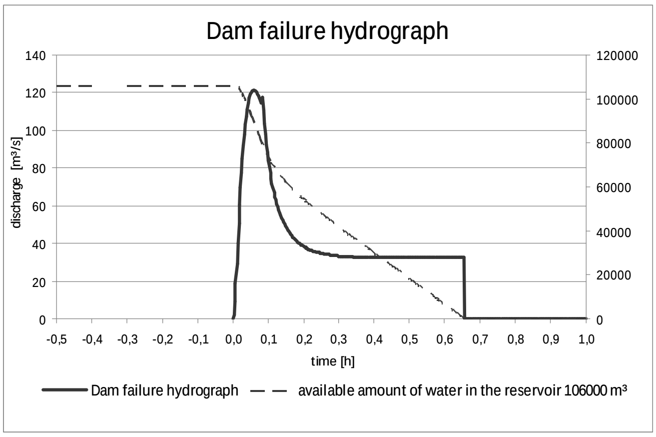

Figure 2: Dam failure hydrograph out of DEICH 1.15, the time spread up to the initial pipe cannot be estimated

The peak value and the time of peak out of this computation are compared to the values from the formulas according to the ICOLD Bulletin 111 [1].

TABLE 1: ESTIMATING PEAK VALUES

Author	Peak value
Froehlich	213 m³/s
Costa	208 m³/s
Molinaro	234 m³/s

With the help of the formula of Froehlich, the time spread to the peak value of the dam failure hydrograph can be estimated with 367 seconds.

All mentioned formulas out of [1] are based on dam failure hydrographs which happened in the past. The dam crest of these dams were higher and there was also much more water in the reservoir than in the dam used in the presented study. So the regression curve out of these studies delivers unusual high values for small dams. Based on the facts above the calculated dam failure hydrograph in the presented study is a realistic curve for this dam.

Flood wave propagation

With the dam failure hydrograph as an input, the flood wave propagation is calculated with the 2D numerical flow simulation program Hydro_As_2d. Based on airborne laser scanning data (1 point/m²) and land use data, to estimate the specific roughness, the terrain model is built (Figure 3). To calculate the most devastating scenario for the flood wave propagation, several starting points of the flood wave at the dam are reviewed.

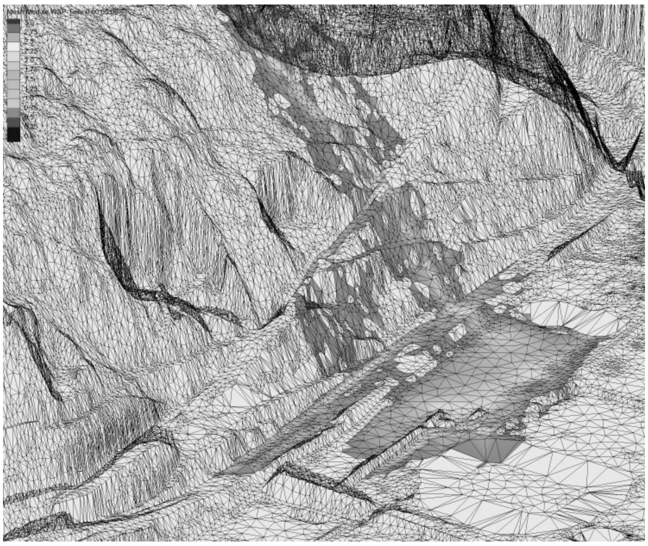

Figure 3: Water flowing down on the triangulated terrain model – endangered area

Results

The result of the present study is the chronology of the flood wave propagation with water depths and velocities. The arrival times of the flood wave are nearly the same for all scenarios. Within the first 15 minutes, very high velocities occur mostly in inhabited areas. After 15 minutes, due to the terrain slope, the velocities and the speed of the flood wave propagation decrease.

After 6 hours of simulation, most of the water out of the dam (106.000 m³) is stored in the flooded area (approx. 3.000.000 m², 1-2 % with basements, 30.000-60.000 m³) or transported through the sewer system out of the area (capacity 2-3 m³/s, 40.000-65.000 m³). Due to the arguments mentioned above the calculation is aborted after 6 hours.

Based on the results of the flood wave propagation, the hazard/emergency planning for this dam on the owner's side and for the municipality involved is adapted.

Conclusion

The presented study was based on the ICOLD Bulletin 111 [1], which is very good guideline for dam break analyses. For small dams the formulas mentioned in [1] lead to high peak value of the dam failure hydrograph. The time spread up to the peak is also estimated too long for small dams. Furthermore, it is difficult to take care about the impervious coating on the upstream side of the dam in the used software because this material has a very high cohesion. At least in the 2d numerical model of the flood wave propagation the amount of stored water in basements or the out flown water through the sewer system has to be estimated because the model was not able to include these settings.

To sum up, the ICOLD Bulletin 111 gives good advice for a dam break analyses. This study shows that even with software, not only a reasonable value but first of all a temporal high-resolution dam failure hydrograph can be calculated. With the help of the 2d numerical model, the flood wave propagation can be estimated, so that the emergency response organizations get a rough overview for their hazard planning. A dam break analysis, as shown in this study, is absolutely necessary for dams to show the potential danger and in further consequence to configure the safety equipment of the dam to prevent a dam break.

Acknowledgements

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