Reviewers #1

The authors highly appreciate the comments of the Reviewer, helping to improve the paper quality. The authors carefully went through all comments and incorporated relevant amendments in the paper. Below are also responses to issues raised. First the general comments are addressed, further the specific ones.

General comments

RC. This paper documents an important dam breach event (Niedów dam-breach) that occurred in 2010 during an extreme synoptic rainfall event that involved a large watershed. The test case is interesting although it could be better documented.

Thank you for this general and positive comment.

RC. Regarding the methods, some empirical equations are used to model the dam breach and a widely used 2D hydrodynamic model is used for the simulation of the flood wave in the long stretch of the river downstream of the dam. In spite of the interest of the test case, the paper is not well written and it is not clear what are its scientific reasons of interest.

We will clarify a number of issues following the specific comments. Concerning the scientific goals, they are:

To document and explain the causes of the dam breach of the Niedów dam as a case study; further to test several available empirical formula on the dam breach prediction, and finally to execute a numerical flood routing using a 2D model to determine the breach outflow hydrograph along with the flood propagation, relevant for the evaluation of the flood damage causes. A change is made in the introduction (L. 95-101):

The current work presents a case study of a catastrophic failure of the Niedów dam in Poland, in 2010. The goals of the study are to document and provide a detailed picture of the dam breach along with explanation of the failure mechanism in the case of an earth dam. This is made on the basis of own field survey, witnessed, stored, or restored data. The geographic, meteo- and hydrological conditions leading to this event are presented as well. The work further puts under the test several selected formulas with an attempt to assess the dam breach characteristics and the peak outflow. Finally, a two-dimensional (2D) hydrodynamic model is applied to more reliably determine the dam break outflow hydrograph and consequent flood wave propagation, in particular, quantifying the effect of the Niedów dam failure on the flooding downstream area.

The inadequacy of empirical dam breach formulas to represent such a complex situation is not a surprise…

The current dam breach development is specific, different from usually assumed schemes. In particular, the presence of the two-layered concrete slabs on the upstream dam slope as the impervious layer (seepage protection), combined with an asphalt road on top of the dam, hindered the breaching depth due to jamming of the slabs, with a consequence of relatively low outflow peak. On the other hand, the breaching width is larger, leading to different geometrical proportions of the breach, compared to those anticipated by the formula. This indicates that the breach forecast needs to carefully take into account the dam structure and search for a most suitable dam breach forecast method.

…and the meaning of the claim of "propagation with an (sic) reversed solution of the upper boundary hydrographs" is not clearly understandable, because too many points in the paper are presented in a rather confusing and non-reproducible way.

We have made multiple changes to a number of statements, including the description of the modelling approach, accordingly to specific comments. By 'reversed solution' it was meant that unknown upper boundary inputs were sought based on known low (output) boundary. For this an iterative search was executed. Nevertheless, the word 'reversed' is removed for not being pretentious.

Much of the reasoning is based on Eq. 1 that however is a wrong transient mass balance equation because it disregards storage in the floodplain flooded area, which must be relevant in the particular case. However, no variation of the stored water volume appears in this equation, where inflow hydrographs equate, at each time step, the output hydrograph.

Eq. 1. was written as problem specific for the unsteady flow simulation. The term for the valley retention was originally not added as the water mass conservation is inherent to the 2D model. Nevertheless, for formal reasons and following Reviewer's comment we will add a term for change of the retention in the river valley, *-dV(t)*.

In fact, the mass conservation was of prior importance of this flood routing (therefore a 2D model was applied after an initial 1D attempt), to ensure that the calculated low boundary discharge hydrograph is conform with the gauge recordings.

From a technical point of view, an undiscussed aspect is why they waited so long to open the gates (see Table 1 progression of the opening of gates I,II and III)

To control the water level in the reservoir, the crew initially followed the operational manual. The procedure was to elevate stepwise the gate by 0,2 m to maintain the desired water level. If necessary additional gate was elevated by 0,2 m too. In the course of this unpreceded water level rise the gate opening was accelerated, nevertheless not to avoid of flooding of the control room. Finally the crew tried to open more the gates manually from the dam crest. The flooding occurred on Sunday, and the crew was not fully aware on the dynamics and scale of escalating flood damage on the Czech Republic territory, including the damage of the Frýdland gauge station on the Smeda/Witka river upstream. In fact, this dam operation was a subject of prosecution and the court trial with judgment acquitting all the persons charged.

As a final note, I would recommend using the term dam breach throughout the paper in place of dam break, a term that in the field of hydraulics is related to the impulsive collapse of a dam, as typical for reinforced concrete or masonry structures.

We use the term 'dam breach'.

Finally, even without being an English mother tongue, I would say without fail that this paper will certainly benefit from a careful and overall improvement of the text that, as it is, is not suitable for an *international Journal. In the following (see attached list) I provide a wide set of examples but many more are still present throughout the paper.*

The paper before submission was checked by a "very experienced translator with an English knowledge on level Native" from a translating office. Even the Authors feeling was that the work was not done very well…

English will be improved and checked by another native speaker familiar with technical and environmental topics.

Specific comments

Authors are grateful to the Reviewer for all numerous specific comments done. We took into account all of them, and here are the explanations (many corrections done in the text are not detailed here).

L. 64. *Do you mean analytical solutions ? I doubt. Accordingly I would say "physically based models"* Changed into: physically based models

L. 76. ANN: changed into: artificial neural networks models

L. 83. *Only in case of overtopping, I suppose, but not in case of siphoning.*

The sentence is modified: Such a construction, in case of overtopping, is likely to significantly affect the breach time and breach geometry, hence it affects the outflow hydrogram and peak.

L. 85-94. Modelling. The whole paragraph is corrected:

The evaluation of the dam failure consequences relies on testing of a number of catastrophic scenarios to further analyze and assess the consequences of the potential flood. The bases of such analysis are hydrologic simulations, numerical modelling of breaching processes, and flood plain flows as well as preparation of inundation maps using GIS systems (Altinakar, 2008; Cleary et al., 2012, 2015; Cannata and Marzocchi, 2012; Álvarez, 2017; Zhong, 2011). 1D models can predict the flood propagation in channels and narrow valleys with reasonable accuracy and good efficiency (Pilotti, 2011), but a 2D or hybrid1D/2D approach should be used in wide floodplains and complex terrain regions with elevated roads, secondary dikes, levees, buildings, and other obstacles (Vanderkimpen and Peeters, 2008). The 2D models gained applications due to significant computational power advances and air-born topographical data availability in recent years (Saberi et al., 2013; Yakti et al., 2018, Banasiak, 2021). In addition, hybrid models are used to derive 1D based breach outflow hydrographs, whereas the 2D model is used for flood plain modelling and generation of inundation maps downstream the dam (Shah et al., 2019).

L.99, 115,116 corrected.

L. 128-130 – *It would be useful to plot the discharge hydrograph at the two stations as far as available*

Drawing the discharge hydrograph was highly desirable but problematic for both stations given the high flow magnitude respective to the measured range and the local topography. There are water levels hydrographs available, but the Ręczyn gauge was destroyed soon after the dam breach started. The determination of the peak discharge for the Ostróżno station was difficult as mentioned in the paper, because of the lack of rating curve for the unpreceded high flow in a wide floodplain.

Fig. 1. *Improve: add numbers indicating Witka river, the Miedzianka river and Lusatian Neisse river which are mentioned in the paper but missing in the Figure Moreover, no indication appears here about the area studied in the 2D modeling, that pops up only in Figure 9*

The figure is modified, including an enlarged plan view of the reservoir. Additional figure showing the Lusatian Neisse river catchment and river network is provided:

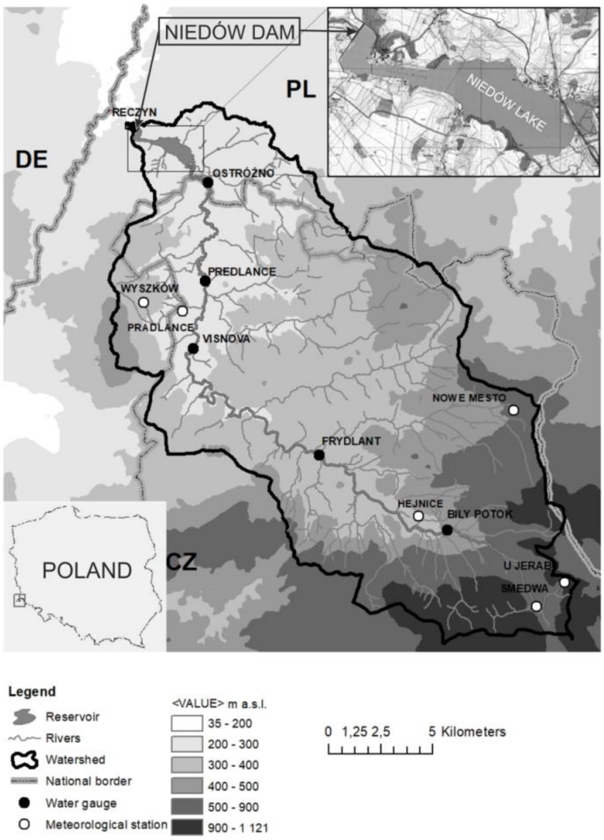


Fig. 1. The catchment of the Witka River

Fig. 2: The upper Lusatian Neisse catchment up to the Görlitz(DE)/Zgorzelec(PL) gauge station (source: IMGW, 2010).

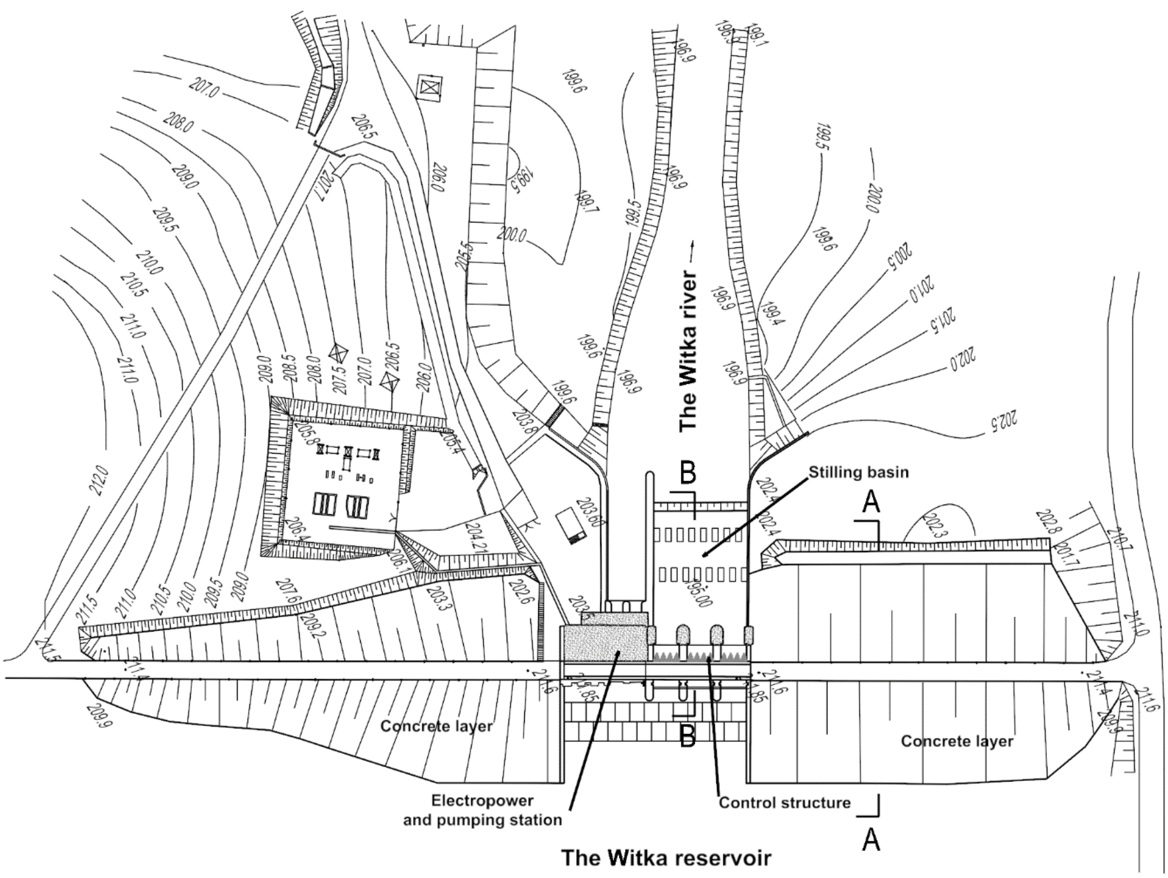
L.152. *Not clear. Do you mean that the maximum water elevation occurred due to the coincidence of two separate flood waves and in correspondence of the peak of the dam breach wave ?*

The sentence is modified:

This information helps to reconstruct the flood wave hydrograph, quantify the argued effect of coincidence flood waves from the two rivers as well as to define upper boundary conditions for the hydrodynamic model.

Figure 3. *Add major dimensions on the blueprint and dashed lines to show the location of the cross section shown in Fig. 4 and 5*

Fig 3. s adapted as below:



L. 167. *Only sand with no clay core present ? Is this a common practice or something that deserve some additional comments ?*

The body of the dam was well compacted sand without a clay core. To protect against water filtration the upstream dam slope was covered by two layers of concrete slabs with a bitumen sealing. cf. Fig. 5.

L. 176-178. *Without a better layout description this statement sounds not realistic. What evidence do you have for this ? I guess that the kinetic component in a reservoir is negligible and the radius of curvature very large and, accordingly, the bend superelevation in a curve must be undetectable. Here, as in several other points of the paper, the description is to vague and unprecise for the reader to really understand what process is in action. For sure an enlarged map of the reservoir upstream of the dam is needed.*

We have to agree with the reviewer's opinion that the kinematic component in this case can be undetectable because of a relatively large cross-section area upstream of the dam, so this claim has been removed from the paper. We tried to find out the reason for overflow first on the left dam side, even also considering a side overflow effect but this is still speculative. An additional 2D modelling for the reservoir area could provide more detail. Nevertheless we reviewed the available data on the dam crest elevation, and concluded that the uneven elevation of the dam crest is the primary reason for the start of the overflow. In addition, a plan view of the reservoir is enlarged in Fig. 1.

L.179. *'lighting foundations 180'. What is it ? What do you mean ?*

Corrected to: “around the lamp post foundations”

L. 188. Right - the left side of the earth dam

L.197. *From the figure one has not this piece of information. Is it the sum of the left and of the right breach widths ?*

The sentence is modified: The final width of the breach of the right dam was 58 m. Fig. 8 illustrates the complete dam breach.

Table 1. *At 17:42 dam – 40 m on the left side and 30 m on the right - What do you mean ?*

It was meant the length of the collapsed road on the top of the dam.
Corrected to: 17:42: Washing-out of the lee side of the dam, destruction of the road on top of the dam – the dam breach width of 40 m on the left side and 30 m on the right side.

L.225 *This part provides the computed discharge. Apart that a steady state equation is used to represent a transient phenomena and that two parallel weirs are in action at the same time, no data (e.g., discharge coefficient….) are provided to really understand how the computation was accomplished . Moreover the evaluation of the breach height using the 2D model is badly explained and potentially totally arbitrary.*

Since the breach for the left and right embankment developed differently, it was necessary to calculate or estimate the input parameters, i.e. the reservoir storage at the crest level and water volume above the breach bottom at the time of failure regarding both embankments. The time-varying outflow of the control structure (all three tainter gates) and the two breach outflows were calculated separately using a well-known hydraulic formula. To calculate the outflow through the tainter gates, the discharge coefficients used are in accordance with those obtained from the laboratory physical model testing for the Niedów dam (Herrera-Granados and Kostecki, 2016). For the dam breaches, first a broad crested weir formula was applied until the road on the dam crest collapses, with a discharge coefficient $C=2/3 \cdot C_d \cdot (2g)^{0.5} = 1.70$. Next, a formula for the sharp crest weir inclined to the horizontal at an angle of 18.4 degrees (as the inclination of the concrete slabs) were adopted from Shesha Prakash et al., 2011, and finally a formula for no crest weir for the situation of breaching to the ground floor were used, with a discharge coefficient C= 1.70, according to USBR, 1987. The growing width of the breach was interpolated based on the photographs and films made during the catastrophe. The breach depth in specific moments, being the most difficult parameter to reconstruct, was calculated making use of the discharge-water level relationship for a section closely downstream the dam. This relationship was obtained in the course of the 2D hydrodynamic modelling. The breach depth was searched as to match the resulting outflow discharge to the discharge obtained in the course of 2D modelling. In case of differences the breach depth was adapted. Finally, based on the outflow volumes of the left and right breach values of V and V_w are estimated. The following results and comments are presented in Section 4.

In fact, use of the formulas to asses all the dam breach parameters is not simple and need several assumptions with inherent uncertainty as discussed later on in the paper.

L. 226. *These documents should be ordered and made available as additional material.*

We considered providing additional material (photos and a video) as well, but the authorship issues are related which are quite difficult to be resolved.

L. 243. *This mass balance equation is wrong because it disregards storage in the flooded area.* …

Eq. 1. was written as problem specific for the unsteady flow simulation. The term for the valley retention was not added as the water mass conservation is an inherent feature of this 2D model. Water flowing into the Berzdorfer Lake is 'lost' from the flood routing domain, as it does not enter back the river valley. Nevertheless, for formal reasons and following Reviewer comment we add a term for change of the retention in the river valley, *dV(t)*. Eq. 1 is now:

$$Q_{NL,in}(t) + Q_{ND}(t) + Q_P(t) + Q_{CW}(t) - V_B - dV(t) = Q_{NL,Z}(t) \qquad (0.1)$$

Moreover Eq (1) shows the mass balance introduced to solve "iteratively" *the unknowns present in the numerical model. In the following lines only the term $Q_{ND}(t)$ is classified as unknown and the other terms being known by the authors at some time (the discharge into the lake should be computed automatically by MIKE 21 using the floodplain topography), so what exactly is the iterative procedure used in the numerical model is not clear.*

The iterative procedure included the adaptation of the hydrograph shape for both upstream boundaries, i.e. for the Lusatian Neisse and the Witka river/outflow from the Niedów dam, as well as finding proper roughness coefficients to obtain a good match between computed and observed hydrographs at Zgorzelec cross-section (timing and peak flow rate). In addition, the total overflow to the Berzdorfer lake had to be ensured conform to data provided by the German party. This was quite complex and, in fact, consecutive iterations were numerous and time consuming taking into account the computer effort of two days for a single run at that time (2011).

L. 264 *Detail better. Provide a map with roughness coefficient.*

Both the bathymetry and roughness raster (here: the velocity coefficient) will be added as supplementary files. The description now is:

v) preparation of the <u>initial</u> roughness raster from the land cover based on aerial photographs; (in total 15 roughness classes were distinguished – for the main channel and open surface waters, grassland and tree areas, bushes, paved surfaces, roads, etc.);

L.265. *I suppose that the flow is subcritical at this cross section and you have a stage-discharge relationship that was further enriched by a measurement at the peak of the event at Zgorzelec cross–*

section Why didn't you use the measured stage-discharge curve, Q(h), as a downstream boundary condition in place of a normal depth that could be unjustified ? Actually how can you be sure that you have not any backwater effect from downstream at the Zgorzelec cross–section ?

MIKE21 v.2010 at hand was not equipped with a stage-discharge relationship closing boundary. Instead, the water elevation in a function of time was used. This is simply the hydrograph of the Zgorzelec gauge station, as the modelled area ends in its cross-section.

The text reads now:

Since the Lusatian Neisse was modeled as an open-ended reach, the downstream boundary condition was set as the water elevation in a function of time. This was based on the Zgorzelec gauge observations directly as the modelled area ends in its cross-section.

L. 268. *What is the proper value of Courant number in your case?*

Text amendment made: The size of modelled area was 13.3 km by 5.0 km and the total number of grid cells was 2.65 million, the computation step was from 0.5 to 0.75 sec. to limit the Courant number to a value of one (although MIKE21 is capable dealing with larger values, DHI, 2011) and to achieve numerical stability and accuracy on the one hand, and feasible computation time on the other hand.

L.285. *Real life ? Do you mean the values computed using the weir equation at line 225 ?*

Modified to: As presented in Table 9, also in this case there are significant discrepancies with our case estimate of 1380 m3/s, based on the flood routing.

L.285. *Why do you not consider that a major discrepancy arises from the complex layout of the breach in this case ? Actually in your case you have two parallel and independent breaches developing at the same time. I doubt that any of the empirical equation considered makes explicit reference to such a complex situation.*

We agree with the Reviewer opinion. Indeed the breaching process under concern was compound and complex, especially the influence of concrete shield was important, hence the deviations between the calculated and assessed peak outflow (the last based on the flood routing) is not surprising. Nevertheless we consider this formula testing is worth presenting not only for their reliability assessment but also for the usage restrictions, as mentioned in the text. We used all formulas to calculated breach characteristics for both dam sides separately.

L. 293. *The sentence reported is vague and does not provide a complete explanation of the reason of the difference between the left and right embankment in terms of the breach width, which, as stated in the current work, is very important in all the reported calculations. A more precise explanation is needed in my opinion.*

Thank you for this indication again. We have corrected this to read:

After the dam crest collapsed, the upstream concrete slabs hindered the growth of the breach down, but less affected the breach horizontal development. The resistance of the concrete slabs may also explain, along with a time delay of breaching inception, why the breach width on the right side was a half of that on the left side.

Fig. 9. The figure is corrected, the size of the modelled area is 13.3 by 5.0 km2 (2660 by 1000 cells 5x5 m2 each). Numbers are added to the text.

Fig. 10. The flow velocity unit is added. We tested several display setting in MIKEView (no color shading for velocity) and make unchanged the visualization of both water depth and velocity vectors at once as a compromise of content and clearance.

L. 315. '*The solution of the problem is iterative.' This is not clear. The problem is not clearly set*

The problem setting is now:

The flooding along the Lusatian Neisse in the studied case is a combination of two major flood waves originating from the upstream river section and the Niedów reservoir outflow. These are two upper boundaries of the modelled domain (see Fig. 10) with hydrographs to be reconstructed.

The reconstruction of these hydrographs was iterative, relying on a series of computations executed with adjusted shapes of hydrographs, including change of timing and rate of peak flows, to satisfy Eq. 1.

L. 320-322. *These are two separate issues: the fixed bed hypothesis and the uncertainty of the local roughness. What do you mean with this statement ? Again, a phrase without a proper explanation. Clearly, any measurement is affected by uncertainty but what do you exactly mean ?*

In fact this is an important issue. In other words, the model was not 'forced 'to be fully conform with several uncertain water level marks. Further, it can be discussed whether the fixed/eroded bed and roughness related uncertainty are two separate issues, or they are linked. To clarify, Lines 320-324 have been changed to:

ii) the cross-sections taken during low or medium flows may not be representative for high flow conditions. The local velocity coefficients M were kept in a range of 36 m1/3/s. This was the case in the section near the Zgorzelec city. Yet, one also needs to bear in mind that high water marks collected after the flood passage, against which the model is calibrated, are affected by (significant) uncertainty. Therefore, a balance on confidence between the model results and field data is sought. The author's being a field surveyor and modeler in one person is an advantage here

L. 325. *Is there a plot that compare measurements and modelled water elevations ?*

The water level hydrographs were drawn in each high water mark location, however here we limit the information to maximum values only. Table 10 is added: Comparison between estimated and calculated water levels.

Table 10. Comparison between estimated and calculated water levels

Water marks	H (m)	H calculated (m)	Difference (m)
WW1	200.73	200.656	-0.074
WW2	199.04	199.121	0.081

WW3	199.63	199.539	-0.091
WW4	197.72	197.618	-0.102
WW5	198.03	197.47	-0.46
WW6	197.66	197.382	-0.278
WW7	197.56	197.383	-0.177
WW8	197.22	197.327	0.107
WW9	195.17	195.09	-0.08
WW10	193.91	193.942	0.032
WW11	191.53	191.312	-0.218
WW12	191.36	191.125	-0.235
WW13	190.22	190.008	-0.212
WW14	189.09	189.203	0.113
WW15	188.31	188.466	0.156
WW16	188.53	188.463	-0.067
WW17	188.77	188.727	-0.043
WW18	185.43	185.498	0.068
WW19	184.79	184.658	-0.132
WW20	182.77	182.754	-0.0

L. 329. *On what basis one can conclude that the inflow is close to reality ?*

The text now: The calculated overflow volume to the Berzdorfer lake amounted 3,783 mln m^3, which is in accordance with the data provided by the German party, assessed based on the water level increase in the lake before and after the flood passage.

L. 330. All figure numbers are checked taking into account the additional one (Fig. 2).

Fig. 12. *I do not understand the physical reason of the discharge plateau in the outflowing dam breach hydrograph of Figure 12. If the Witka entering discharge is growing in time and the outflowing discharge is constant, it means that the level in the reservoir is growing. Why the outflowing dam breach hydrograph does not grow with the water level in the reservoir ?*

Figure 9 The indicated plateau is a results of the yield of water of the control structures. Please remind, that the outflow occurs underneath the gates, so the flow increase with water level is not that large as for free surface overflows.

In the figure there are red marks underneath the names of the locations reported typical of a grammatical check tool, their presence is unessential. Furthermore the schematics provided is very helpful in understanding the domain in which the numerical method is applied, but the lack of a geographical counterpart of the same scheme (a map with all the locations highlighted) damages the understanding of the spatial dimensions involved in the simulation. As previously noted, a map is required to help the reader to orient himself in the various locations described in the current work.

Figure 9 is corrected (below) and a catchment map is added as Fig 2.

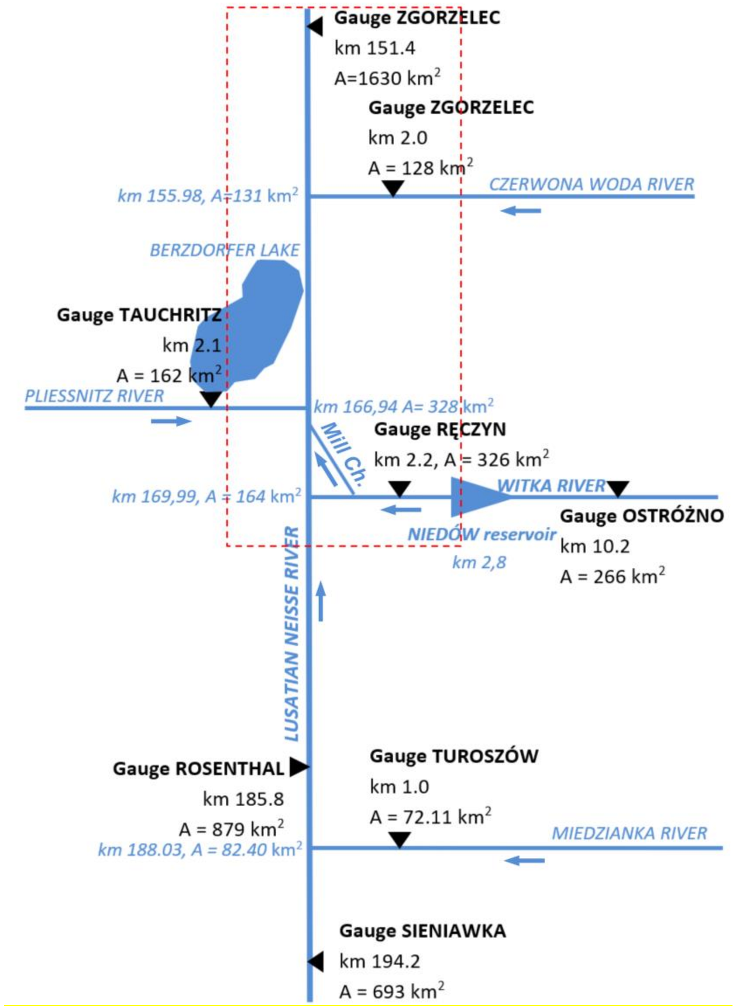


Fig.12. *Colors used to depict the two hydrographs are too similar and generate confusion*

We made them more distinguishable, as below.