Author response to Review comment by Martin Mergili for “Formation, breaching and flood consequences of a landslide dam near Bujumbura, Burundi”

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In this document, the Reviewer comments are shown in the boxes and are directly followed by the authors' response.

General comment

> The authors present an interesting study on the possible impact of the dynamics of a landslide on flooding downstream, thereby considering the effects of breaching of a landslide dam. Much of the manuscript is well written, structured, and illustrated. Particularly the evaluation of the stability conditions of the landslide is very well described. However, coupling of landslide and flood is, in my opinion, insufficiently covered. Particularly in this context I have identified some important issues requiring improvements and therefore recommend **major revisions**. I now outline my suggestions and comments in the order of decreasing priority:

Thank you for the attention paid to our manuscript.

Specific comments

> 2.1. One of my major concerns relates to the fact that only flooding by water is considered. Breach of the landslide dam would release a huge amount of solid material (most probably deeply weathered tropical soil) that would be incorporated in the flow and could possibly lead to completely different characteristics and downstream impact of flooding, compared to clear water flow. This issue is not even discussed at all. I see two possibilities to face this challenge: (i) incorporating sediment load in the flow simulation; or (ii) a thorough argumentation and discussion why this is not necessary. Either (i) or (ii) should be an absolute requirement for the acceptance of the manuscript.

We indeed only included water in the flood wave computation, while breaching of the landslide will release a substantial amount of solid material. The resulting flow will have an intermediate behavior between clear-water flow and debris or granular flow. This issue will be acknowledged explicitly in the revised manuscript, and its implications will be discussed in detail, as proposed hereafter.

Table R1. Some recent studies of flooding induced by the breaching of landslide dams, and of debris flow routing.

	Model dimensions	Morphodynamics	Flow rheology	Available observations
Present study	2D	No	Clear water (turbulent flow)	None
Fan et al. (2012)	1D for river flow, 2D for overland flow	No	Turbulent flow	Peak discharge, peak arrival time …
Yang et al. (2013)	Sobek-1D and -2D	No	Turbulent flow	Flooding occurrences
Shrestha and Nakagawa (2016)	1D for river flow	Yes	Granular, hyper-concentrated and turbulent flow	Observed flood discharge
Li et al. (2011)	1D for river flow, 2D sediment transport	Yes	Empirical equations for Mohr-Coulomb, viscous and turbulent shear stresses	Downstream hydrograph, observed sediment depths …
Mergili et al. (2012a) [NHESS]	2D, considering bottom curvature and steep slope effects	Deposition of granular material represented explicitly	Granular flow (Savage-Hutter type model)	Focused on avalanche flows, not flooding due to dam breaching
Mergili et al. (2012b) [Nat. Hazards]	2D	Sediment detachment by runoff and routing of debris flow	Semi-deterministic two-parameter friction model	Debris flow travel distance, shape of deposits …

"As summarized in Table R1, some recent studies neglected sediment transport in the analysis of floods induced by the breaching of landslide dams (Fan et al., 2012; Yang et al., 2013), while others did take sediment transport into account (Li et al., 2011; Shrestha and Nakagawa, 2016). Indeed, sediment transport may have considerable implications on the volume of mobilized material as well as on morphodynamic evolutions of the valley bottom (e.g., sediment deposition). Nonetheless, in the particular context of the present study, going for more complexity in the modelling framework (i.e. including sediment transport) would not substantially reduce the overall level of uncertainty mainly because validation data are neither available for our case study nor for any similar one in the region, which remains largely understudied. Table R1 shows that previous studies which considered sediment transport benefited from available validation data, such as observed flood discharges or depths of sediment deposits."

In the revised manuscript, we will additionally handle the issue of sediment transport through a comprehensive sensitivity analysis, aiming at appreciating successively the effect of *(i)* the volume of mobilized material and *(ii)* the consequences of morphodynamic evolutions (erosion, deposition).

Effect of volume involved in the flow

The volume V_d of the landslide dam is about 16,000 m³, while the volume V_l of water impounded behind the landslide dam prior to dam breaching is roughly 55,000 m³. Table R2 provides a estimate of the ratio between the volume of dam material and the total volume of water contributing to dam erosion in the various considered scenarios. Table R2 suggests that only in the case of a 20- or a 50-year flood and a slow erosion of the dam (in hours), the volume of dam material could reasonably be neglected compared to the volume of water, as in this case, the volume of water contributing to the dam erosion is approximately twenty to thirty times larger than the volume of the dam material. In all other cases, the volume of dam material ranges between 12 % and 30 % of the water volume and is therefore not negligible.

We propose to address this in the revised manuscript by conducting additional simulations in which the dam material is assumed “fluidized” as the breaching develops, instead of being “removed” from the simulations as it is the case now (in accordance with common practice in risk analysis of engineered dams). This will provide some hints on the influence of the overall volume of material (although assumed fluid) involved in the flow.

Table R2. Estimated volume of water released at the dam over the breaching duration, evaluated as $V_l + T_c \times Q_r$. Notation V_l refers to the volume of water initially impounded behind the landslide dam, Q_r to the river discharge before dam breaching and T_c is a characteristic time, taken equal to 60 s for the extreme scenario of instantaneous dam breaching and equal to T_f (breaching duration) in the other cases. Notation V_d designates the volume of the dam.

		Dam breach scenario		
Hydrological scenario	River discharge Q_r before dam breaching	“Instantaneous” dam breaching	Breaching duration of 600 s	Breaching duration of 3600 s
Mean discharge	3 m³/s	$5.5\ 10^4$ m³ ≈ 3.5 V_d	$5.7\ 10^4$ m³ ≈ 3.6 V_d	$6.6\ 10^4$ m³ ≈ 4.1 V_d
20-year flood	60 m³/s	$5.8\ 10^4$ m³ ≈ 3.7 V_d	$9.1\ 10^4$ m³ ≈ 5.7 V_d	$2.7\ 10^5$ m³ ≈ 17 V_d
50-year flood	120 m³/s	$6.2\ 10^4$ m³ ≈ 3.9 V_d	$1.3\ 10^4$ m³ ≈ 8.0 V_d	$4.9\ 10^5$ m³ ≈ 31 V_d

Effect of morphodynamic evolutions

In our reply to comment 2.5, we propose to report in the revised manuscript on an additional set of simulations to test the sensitivity of the modelling results to the use of a different DEM (derived from field survey). This additional simulation will also give some insights into the effect of changes in the river bathymetry (e.g. as could be obtained as a result of erosion / deposition, which will not be modelled explicitly) and we suggest to link this to the present comment. We may also consider running additional simulations in which we include changes in the DEM to mimic plausible deposits in the downstream (e.g., where the longitudinal slope decreases sharply). The results of these additional simulations will enable appreciating the influence of possible deposits on flooding.

Thanks to the sensitivity analysis conducted based on the proposed additional model runs, we will assess in the revised manuscript which parts of our conclusions are strong despite the existing uncertainties and which ones are more affected by the modelling uncertainties linked to sediment transport. In addition, we will clearly indicate as a perspective in the Conclusion section of the revised manuscript that the present study should be further continued using more advanced debris flow / granular flow modelling tools such as presented by Mergili et al. (2012a, 2012b, 2017) or others, and adapted to channelized debris flow.

2.2. I do not fully understand the work flow of the flood modelling: in the first step, do you (i) simulate the base flow without the dam incorporated, or do you (ii) fill the lake behind the dam to let it flow out in the second step? The description in Sect. 2.4.3 is confusing and has to be improved.

In the first step, we fill the lake behind the dam to let it flow in the second step. This will be clarified in the revised manuscript, by introducing a new table (Table R3) and by rewording section 2.4.3 as detailed hereafter:

"The hydraulic simulations aim at evaluating the impact of the dam failure as a result of the water impoundment behind it and the river overflowing the dam crest. Thus, the initial step of hydraulic modeling considers a filled reservoir and a steady flow of water over the crest of the dam before failure. In line with Dewals et al. (2011), the modelling procedure involves two steps:

- step 1: a pre-failure steady flow is computed in the river, under three different hydrological scenarios (steady flow corresponding to the mean discharge in the river or to a 20-year flood, or a 50-year flood);
- step 2: using the result of step 1 as initial condition, the flow induced by the breaching of the dam is computed.

In Step 1, the dam geometry is incorporated in the topographic data used for flow computation. This means that the dynamics of material sliding into the river is not explicitly reproduced in the hydraulic modelling. As it is not possible to anticipate when the landslide dam breaching might occur, we consider three different pre-failure flow conditions: base flow, 20-year flood and 50-year flood.

In Step 2, using a parametric description of the breaching, the dam is gradually removed from the topography, so that the water impounded behind the dam is released. The model computes the unsteady propagation of the flood wave in the downstream valley."

Examples of results of Step 1 and Step 2 are displayed in Fig. R1 and Figs R2 to R5, respectively.

More details on the parametric description of the dam breaching are given in our reply to comment 2.3 below.

Table R3. Two-step hydraulic modelling protocol

	Hydraulic computation	Dam
Step 1	Steady-state simulation	Incorporated in the DEM used for the simulation
Step 2	Unsteady simulation	Gradually removed from the DEM (time-dependent topography)

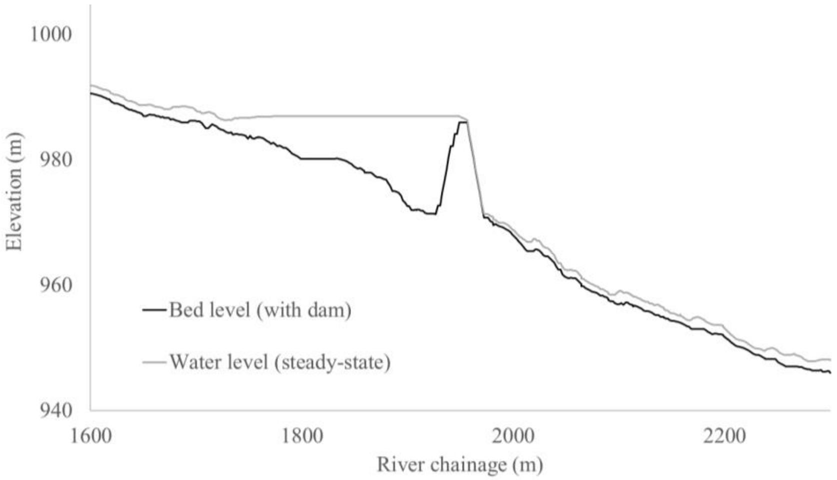


Figure R1. Longitudinal profile (in the dam area) of the bed and water levels for a steady discharge of 120 m³/s, as computed in Step 1 of the hydraulic modelling procedure (k_s = 0.3 m).

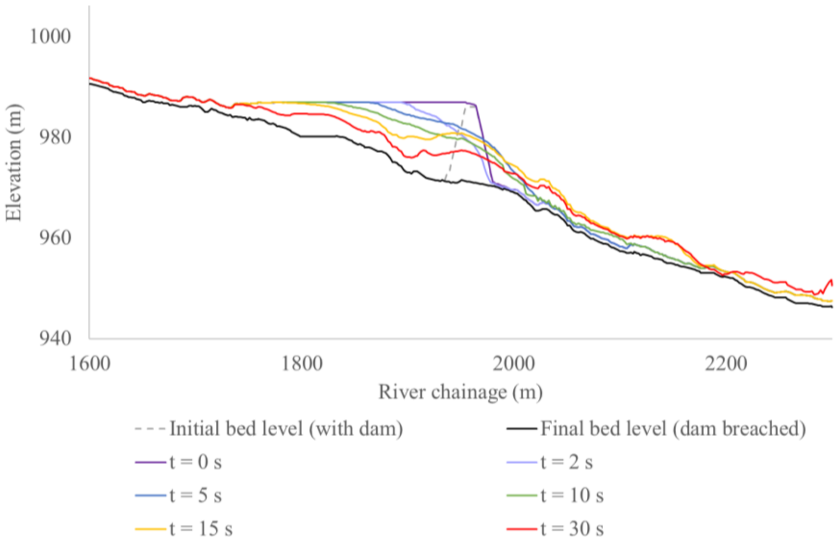


Figure R2. Longitudinal profiles of water levels computed in Step 2 of the hydraulic modelling procedure, assuming an instantaneous breaching of the dam (extreme case) and a flow rate of 120 m³/s in the river prior to dam breaching (k_s = 0.3 m).

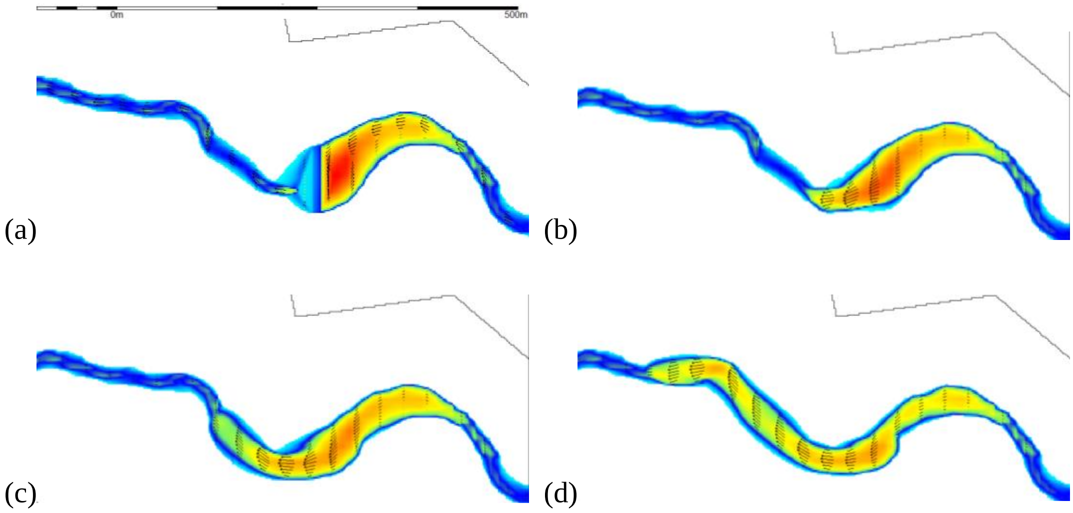


Figure R3. Water depth distribution and velocity profiles before the breaching (a) as well as after 5 s (b), 10 s (c) and 20 s (d), as computed in Step 2 of the hydraulic modelling procedure. This computation assumes an instantaneous breaching of the dam (extreme case) and a flow rate of 120 m³/s in the river prior to dam breaching (k_s = 0.3 m).

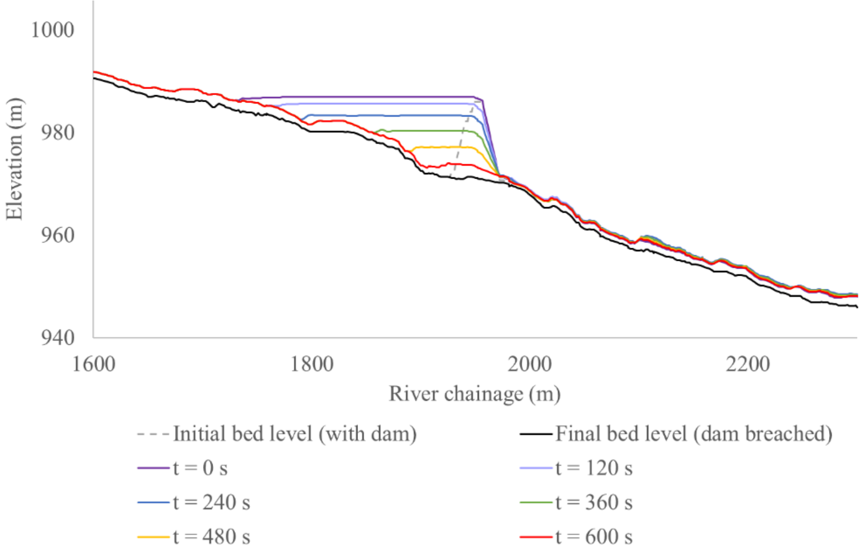


Figure R4. Longitudinal profiles of water levels computed in Step 2 of the hydraulic modelling procedure, assuming a breaching duration of 600 s and a flow rate of 120 m³/s in the river prior to dam breaching (k_s = 0.3 m).

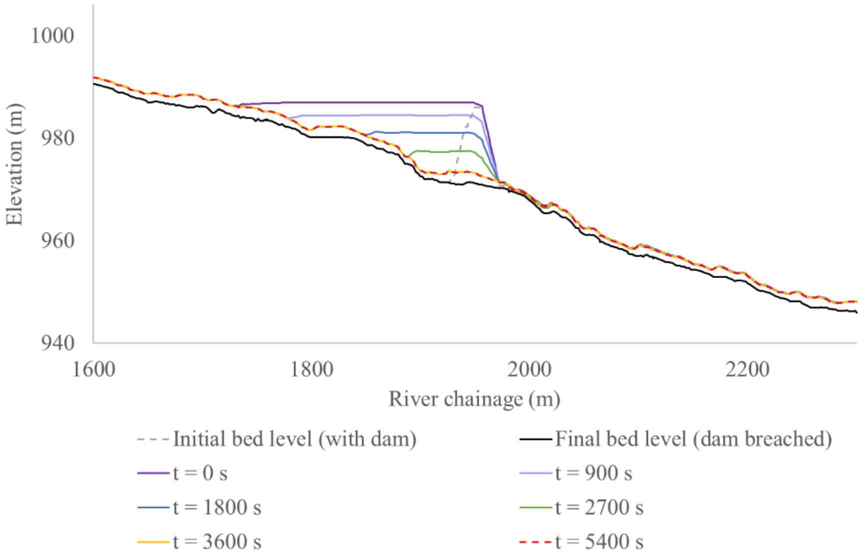


Figure R5. Longitudinal profiles of water levels computed in Step 2 of the hydraulic modelling procedure, assuming a breaching duration of 3600 s and a flow rate of 120 m³/s in the river prior to dam breaching (k_s = 0.3 m).

2.3. A highly critical issue is also the consideration of dam breach (lowering of the dam crest and release of the impounded water) – how does this work? Please explain! I have the feeling that you spend a lot of effort in describing base flow and lower boundary conditions at a high level of detail, but do not explain some of the really important aspects at all.

As stated in the initial manuscript, we closely followed the procedure proposed by Dewals et al. (2011) for representing the dam breaching. Nonetheless, as pointed out by the Reviewer, we agree that this procedure deserves more explanations and more discussion in the manuscript, since it is indeed an important step of our study.

In our response below,

- we explicitly describe how the dam breaching is represented in the model;
- we also present and discuss the modelling results for an additional scenario of gradual dam breaching.

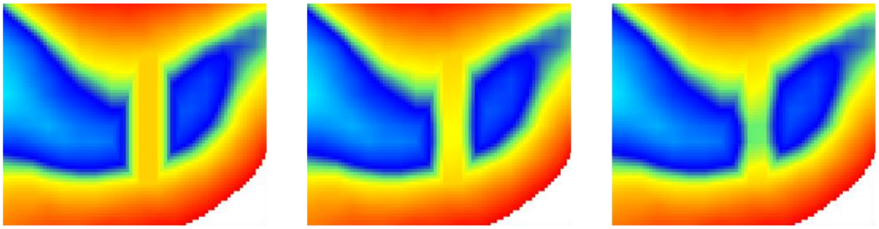
The text in the revised manuscript will be updated accordingly.

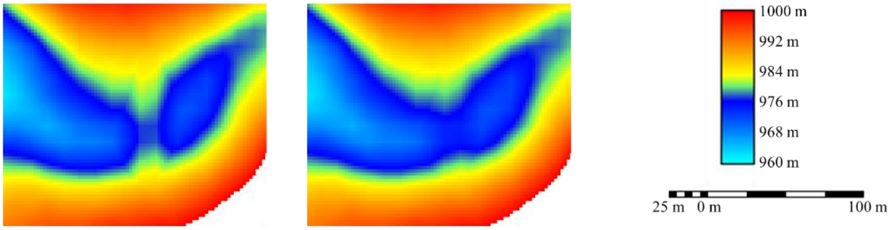
“The mechanisms of breaching of natural dams are complex, highly variable and incompletely understood. Hence, the modelling of the dam breaching may be a substantial source of uncertainty.

In the present study, process-oriented modelling of the breaching was not considered as a viable option, mainly due to the lack of detailed information on the dam material (graded, non-homogeneous material), the complexity of the breaching of natural dams and the absence of validation data from similar case studies in the region. Instead, we opted for a simpler *parametric description* of the dam breaching which appears more consistent with the quality of available data and the overall level of uncertainty affecting the present study.

Among the various possible failure modes, we chose to represent dam *overtopping*, which is the most frequent failure mode for landslide dams. Failure induced by dam overtopping was reported for over 90 % of all landslide dams reviewed by Costa and Schuster (1988) and for 131 out of 144 cases reviewed by Peng and Zhang (2012).

As sketched in Fig. R6, the parametric breach model was implemented in the 2D flow model by means of a time varying topography. The breach outflow is thus explicitly computed by the flow model, enabling the representation of the hydraulic coupling between reservoir depletion, flow through the breach and possible backwater effects. This procedure requires a user-defined initial dam geometry (Fig. R6a) and a user-defined final geometry corresponding to the breached dam (Fig. R6e). In-between these two geometries, the algorithm performs a linear interpolation in time (Dewals et al. 2011). The breaching duration also needs to be prescribed by the user."


(a) Initial state (with dam) (b) $t = 0.25\ T_f$ (c) $t = 0.50\ T_f$



(d) $t = 0.75\ T_f$ (e) Final state (without dam, $t = T_f$)

Figure R6. Plane view of the topography evolution in the near-field of the landslide dam as a function of time (T_f stands for the breach formation time).

Several prediction formulae have been tested for estimating the breaching duration (Froelich 2008, Peng and Zhang 2012, BREACH model …). They lead to scattered values, ranging in-between 10 min and one or two hours. Such discrepancies result from the limited number of real-world case studies for which information on breaching duration is available. For instance, out of a total of 1,239 cases reported by Peng and Zhang (2012), only 52 contain detailed information on the breaching and only 14 cases have records of breaching duration. Moreover, inconsistencies exist in these records, so that the regression results for breaching duration are generally less satisfactory (in terms of R^2) than for other breach parameters. These are the reasons why, in the revised manuscript, we will discuss the results obtained based on a range of plausible assumptions on the breaching duration: 10 min (Fig. R4) and 1 h (Fig. R5). One extreme assumption will also be considered (instantaneous dam failure) to characterize the envelope of possible results. The latter scenario could also correspond to an almost instantaneous breaching following an earthquake.

While the initial manuscript detailed only the results for the most extreme case (instantaneous dam failure), the revised version of the manuscript will include a detailed presentation of the results obtained for the other two breaching durations (10 min and 1 h). The text and all figures in section 3.3 will be revised accordingly. For instance, Figs. 11 to 13 and Tabs. 7 to 9 in the original manuscript will be replaced by the following figures and tables in the revised manuscript. The discussion will also be adapted, as the results reveal a substantial influence of the breaching duration in the upper part of the valley; while this influence becomes much smaller in the urban area of interest.

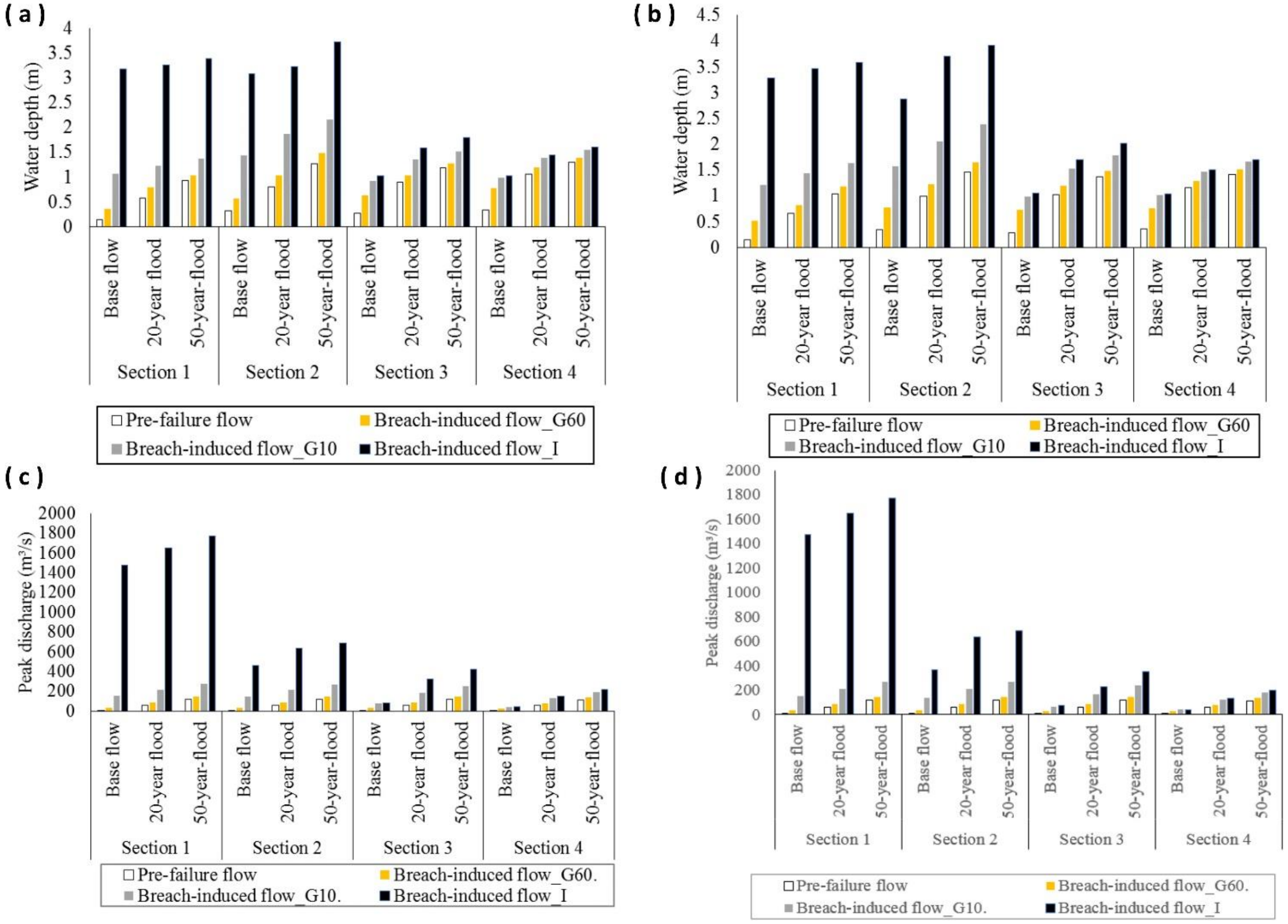


Figure 11. Computed water depths (a, b) and discharge (c, d) for various pre-failure flow conditions (base flow, 20- and 50-year floods), and corresponding maximum water depths (a, b) and peak discharges (c, d) after dam breaching, in cross-sections 1 to 4 and for a roughness height k_s = 0.1 m (a, c) and 0.3 m (b, d). ‘Breach-induced flow_G10’, ‘Breach-induced flow_G60’ and ‘Breach-induced flow_I’ stand for ‘Breach-induced.flow_gradual with 10 minute as breaching time’, ‘Breach-induced.flow_gradual with 60 minute as breaching time’ and ‘Breach-induced.flow_instantaneous’.

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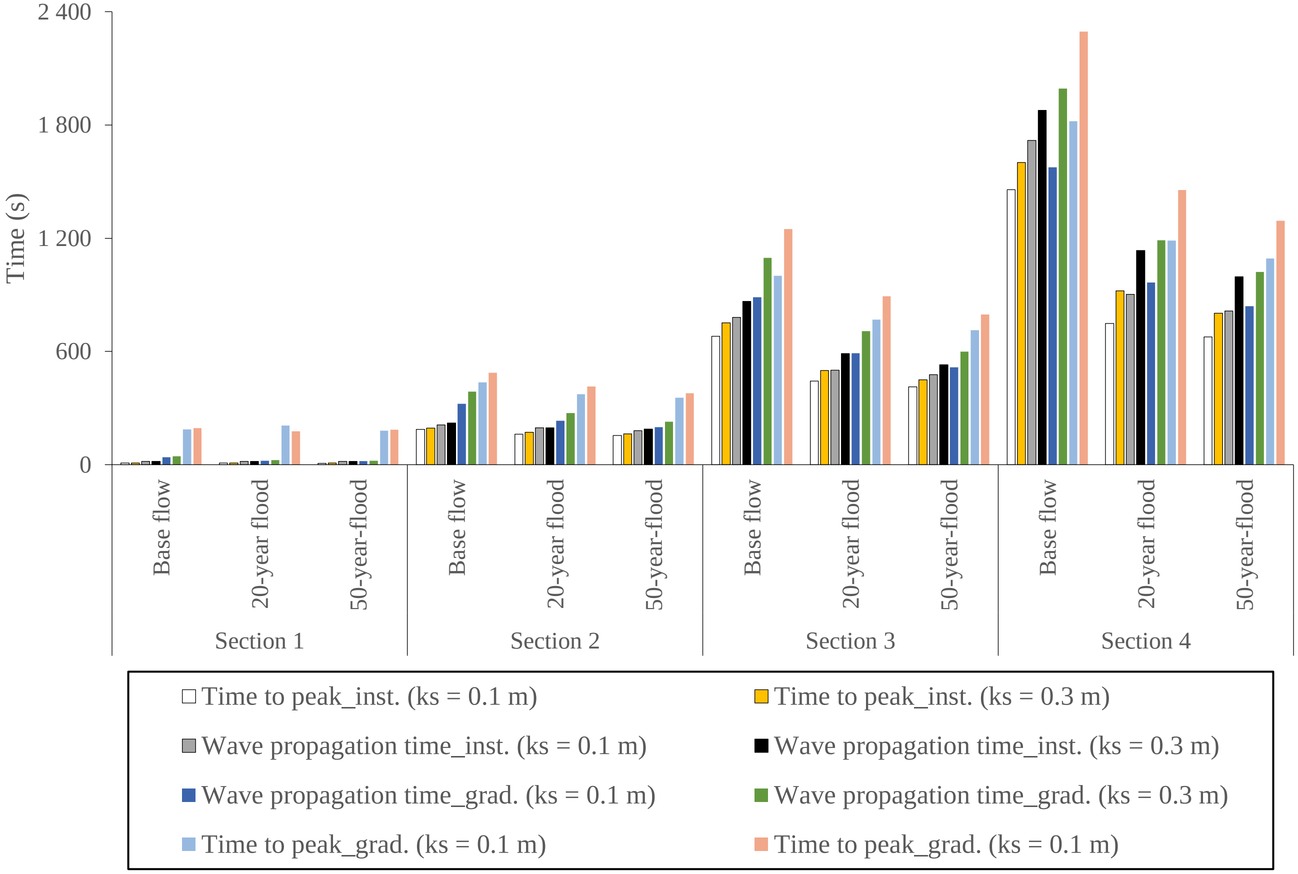

Figure 12a. Computed wave propagation time and time-to-peak in sections 1 to 4, for various pre-failure flow conditions (base flow, 20- and 50-year flood) and for two different roughness heights (k_s = 0.1 m and k_s = 0.3 m). The gradual failure time is 10 minutes. 185

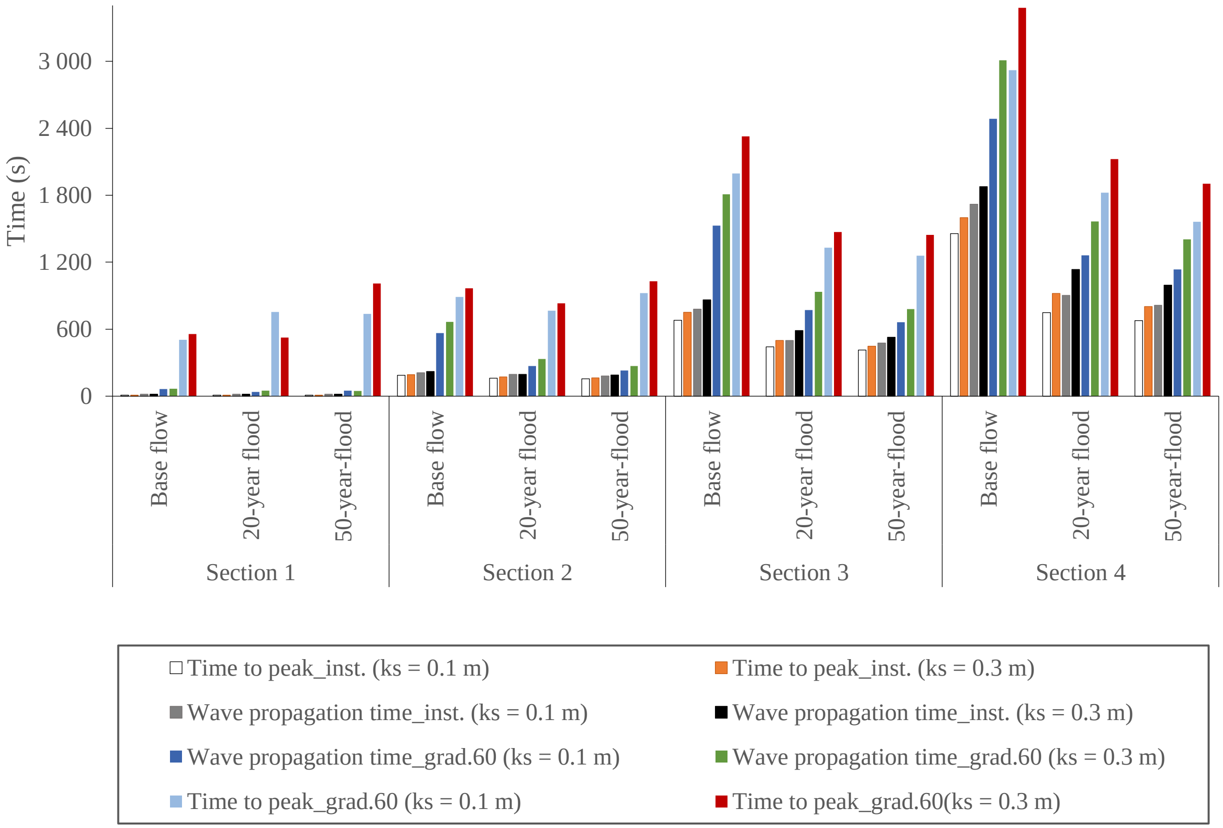

Figure 12b. Computed wave propagation time and time-to-peak in sections 1 to 4, for various pre-failure flow conditions (base flow, 20- and 50-year flood) and for two different roughness heights (k_s = 0.1 m and k_s = 0.3 m). The gradual failure time is 60 minutes.

Moreover, Table 7, 8 and 9 are modified as follows:

Table 7. Ratio between the maximum water depth following dam breaching and the water depth in the pre-failure flow conditions in sections 1 to 4, considering two different roughness heights (k_s = 0.1 m and k_s = 0.3 m) and various pre-failure flows (base flow, 20-year flood and 50-year flood). I and G_{10} and G_{60} stand for instantaneous, 10 minutes-gradual breaching and 60 minutes-gradual breaching respectively.

Section	ks	Failure mode	Hmax ratio		
			Base flow	20-year flood	50-year flood
Section 1	0.1	I	22.60	5.70	3.60
		G_{10}	7.57	2.16	1.47
		G_{60}	2.57	1.39	1.12
	0.3	I	23.50	5.30	3.50
		G_{10}	8.64	2.18	1.58
		G_{60}	3.64	1.24	1.15
Section 2	0.1	I	9.60	4.00	2.90
		G_{10}	4.50	2.34	1.69
		G_{60}	1.75	1.29	1.17
	0.3	I	8.50	3.70	2.70
		G_{10}	4.62	2.07	1.64
		G_{60}	2.26	1.23	1.12
Section 3	0.1	I	3.80	1.80	1.50
		G_{10}	3.41	1.53	1.29
		G_{60}	2.37	1.17	1.08
	0.3	I	3.80	1.70	1.50
		G_{10}	3.54	1.50	1.31
		G_{60}	2.57	1.18	1.08
Section 4	0.1	I	3.10	1.40	1.20
		G_{10}	2.97	1.30	1.18
		G_{60}	2.33	1.12	1.06
	0.3	I	3.00	1.30	1.20
		G_{10}	2.91	1.26	1.17
		G_{60}	2.14	1.11	1.06

Table 8. Ratio between the peak discharge following dam breaching and the discharge in the pre-failure flow conditions in sections 1 to 4, considering two different roughness heights (k_s = 0.1 m and k_s = 0.3 m) and various pre-failure flows (base flow, 20-year flood and 50-year flood).

Section	ks	Failure mode	Qmax ratio		
			Base flow	20-year flood	50-year flood
Section 1	0.1	I	490.0	28.0	15.0
		G_{10}	51.5	3.5	2.3
		G_{60}	11.1	1.5	1.5
	0.3	I	490.0	28.0	15.0
		G_{10}	51.6	3.5	2.3
		G_{60}	11.1	1.5	1.2
Section 2	0.1	I	150.0	11.0	5.7
		G_{10}	47.6	3.5	2.2
		G_{60}	11.1	1.5	1.2
	0.3	I	120.0	11.0	5.7
		G_{10}	45.3	3.5	2.2
		G_{60}	10.9	1.5	1.2
Section 3	0.1	I	27.0	5.4	3.5
		G_{10}	24.7	3.0	2.1
		G_{60}	9.5	1.5	1.2
	0.3	I	25.0	3.8	2.9
		G_{10}	20.9	2.7	2.0
		G_{60}	9.0	1.5	1.2
Section 4	0.1	I	15.0	2.6	2.0
		G_{10}	15.5	2.3	1.7
		G_{60}	8.9	1.4	1.2
	0.3	I	14.0	2.2	1.8
		G_{10}	13.0	2.0	1.6
		G_{60}	8.1	1.4	1.2

Table 9. Predicted change in terms of flooded area due to the landslide induced dam breaching for roughness = 0.1 m and 0.3 m.

Pre-failure flow	Pre-failure flooded area (m^2)	Flooded area after dam failure (m^2)			Relative increase in flooded area as a result of dam breaching (%)		
		Instantaneous	Gradual (10 min)	Gradual (60 min)	Instantaneous	Gradual (10 min)	Gradual (60 min)
Roughness height k_s = 0.1 m							
Base flow	447660	601184	577108	539536	34.29	28.92	20.52
20-Year	529204	695236	632712	590280	31.37	19.56	11.54
50-Year	556816	757300	707024	637320	36.01	26.98	14.46
Roughness height k_s = 0.3 m							
Base flow	493028	635484	599948	561700	28.89	21.69	13.93
20-Year	604988	741964	689388	636916	22.64	13.95	5.28
50-Year	747764	898048	859004	824928	20.10	14.88	10.31

Figure 13 will be improved, including results of the gradual failure. The modified figure 13 in the revised manuscript will include results of a gradual failure corresponding to 10 minutes and 60 minutes as breaching time. That means that 2 more columns (6 maps) are to be added. However, adding new elements (12 maps instead of 6) in the same figure significantly reduces the visibility of the legend and the other texts that are incorporated into it. That is why Fig. 13 will consist of 2 figures: Fig. 13a (including only one more column for failure scenarios with 10 minutes as breaching time) and Fig. 13b (including failure scenarios with 60 minutes as breaching time).

A revised version of Fig. 13 is proposed bellow:

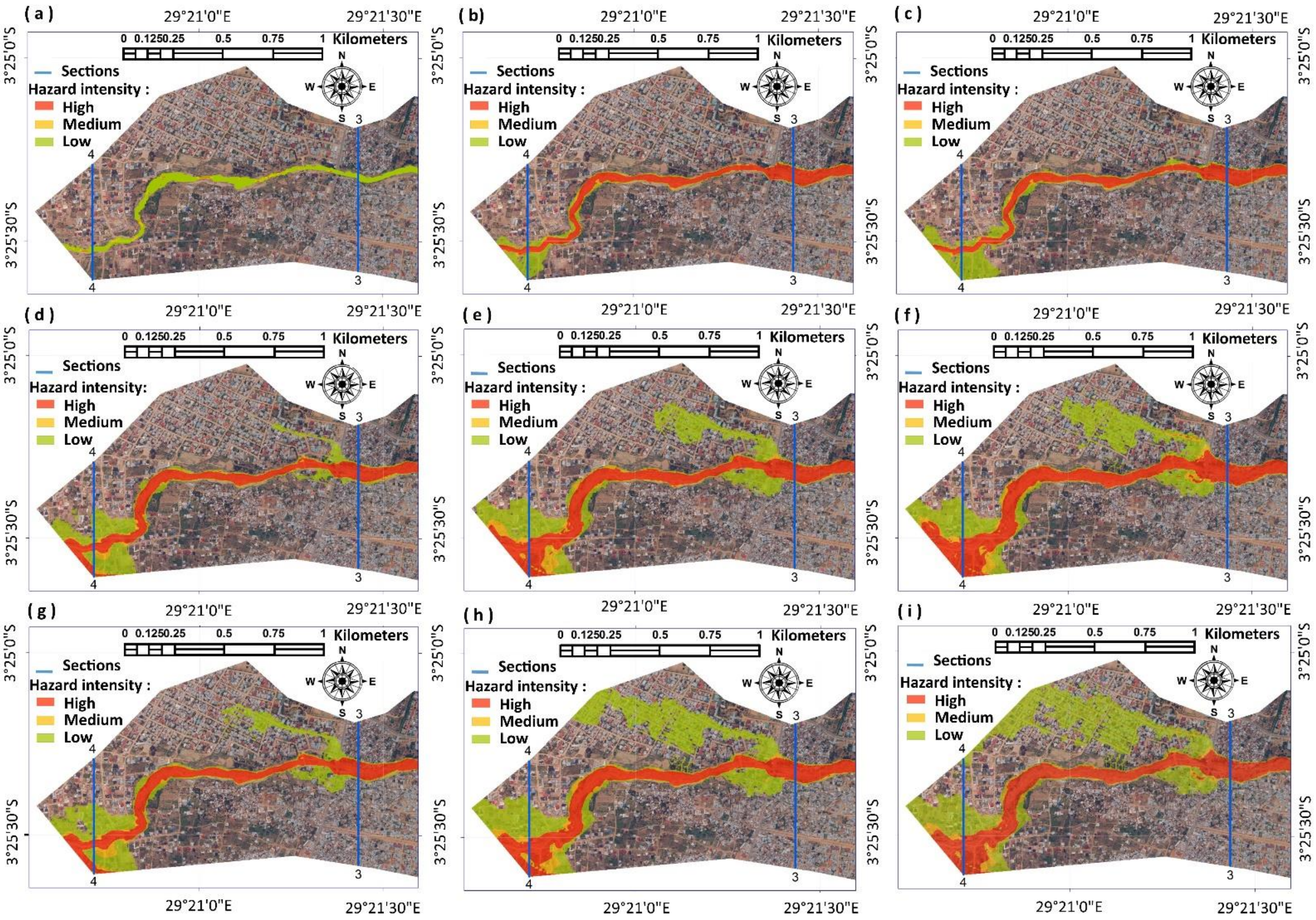

Figure 13a. Hazard intensity maps for different initial steady discharges and roughness: the first column (a, d, g) corresponds to the pre-failure scenarios while the second (b, e, h) and third (c, f, i) columns relate to the gradual (10 minutes as breaching time) and instantaneous breaching. The first line (a, b, c) is based on the base flow and a roughness height of 0.1 m. The scenarios of the second line (d, e, f) are simulated using a 50 years-flow and a roughness of 0.1 m. The third line (g, h, i) is similar to the second one, but considers a roughness height of 0.3 m.

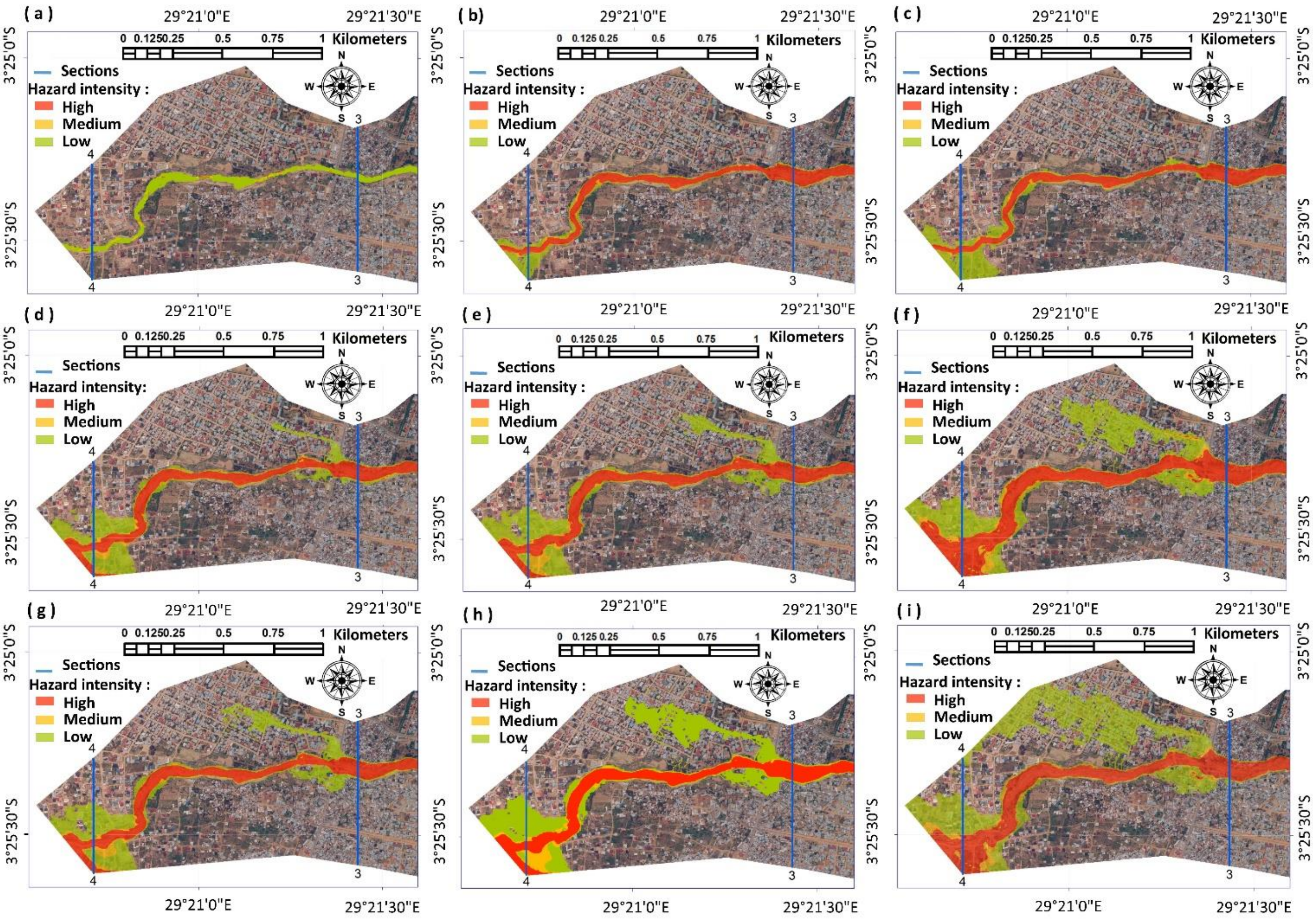

Figure 13b. Hazard intensity maps for different initial steady discharges and roughness: the first column (a, d, g) corresponds to the pre-failure scenarios while the second (b, e, h) and third (c, f, i) columns relate to the gradual (60 minutes as breaching time) and instantaneous breaching. The first line (a, b, c) is based on the base flow and a roughness height of 0.1 m. The scenarios of the second line (d, e, f) are simulated using a 50 years-flow and a roughness of 0.1 m. The third line (g, h, i) is similar to the second one, but considers a roughness height of 0.3 m.

The text in the revised manuscript will be updated accordingly.

> 2.4. You claim to consider flood hazard – however, it is only the possible intensity which is used for the preparation of the maps – hazard would also have to include a measure for frequency. Some rewording (e.g. flood intensity indication map?) will be necessary.

We agree and we will revise the terminology throughout the manuscript, as it is already done in the legend of the maps within Fig. 13a and Fig. 13b proposed above.

2.5. You resample the 10x10 m DEM to 2x2 m. it is absolutely clear to me that this is necessary for numerical reasons – still, it does not increase the level of topographic detail. How wide is the river, i.e. is an effective 10x10 m cell size sufficient to capture the topographic patterns governing the flow? Please discuss.

In our response below, we first highlight that the study was conducted in a data-scarce context. Next, we report on a field survey conducted in the study area, which enables assessing the DEM we used for hydraulic modelling.

The average width of the river is about 20 m for a discharge of 3 m³/s, 32 m for 60 m³/s (20-year flood) and 40 m for 120 m³/s (50-year flood). Hence, a computational spacing of 2 m (obtained after resampling) is certainly fine enough to represent the flow field over the width of the river, since the number of computational cells over the width of the river is in-between 10 and 20. Nonetheless, the Reviewer is of course right that only the topographic details already present in the initial DEM (10 m × 10 m) are captured in the topography used for hydraulic modelling. This situation stems from the data-scarce environment in which this study was conducted, as also acknowledged by Reviewer 2.

In developed countries, light detection and ranging (LiDAR) elevation data are generally available at a high resolution (up to 0.5 m horizontally). In contrast, data for the study area are particularly scarce. Data scarcity is a common challenge in many regions in Africa. This reality was emphasized by various authors such as Jacobs et al. (2016) or Alvarez et al. (2017). Based on available elevation data (usually SRTM with a horizontal resolution of approximately 30 m), these authors performed hydraulic simulations leading to conclusions considered as scientifically relevant and recently published in leading international journals. This suggests that using medium- or low-resolution products remains a valuable intermediate step to advance our understanding of flood risk in data-scarce areas in Africa, provided that the results are interpreted in light of the uncertainties in input data. In this context, a 10 m resolution is among the best in the region, especially when compared to SRTM and ASTER GDEM provided by USGS. This is the reason why the 10 m × 10 m DEM was used in this manuscript (Section 2.2).

Besides, we conducted field surveys during the dry season (June-September) in 2014 and in 2015. The surveys covered the main riverbed and part of the floodplains (band of 10-20 m) of Kanyosha River, from 500 m upstream of the dam down to Lake Tanganyika. Fig. R7 shows the extent of the field survey, compared to the position of the banks of the river and to the limits of the 10 m × 10 m DEM used for hydraulic modelling. The available equipment did not allow measurements in the lake (this is the reason why we present in the manuscript a sensitivity analysis with respect to the downstream boundary condition).

As shown in Fig. R8, the differences between the DEM used in our hydraulic simulations and data from the field survey remain moderate, as they range generally between – 0.5 m and + 0.5 m. The median and mean differences are both - 7 cm. The RMS error between DTM 10 m × 10 m and field measurements is 65 cm and seems reasonable.

Most significant differences are obtained near the river banks, which may result from discretization errors and/or from the instability of the banks due to planform evolution of the riverbed over the period from 2012 (when the 10 m × 10 m DEM was produced) to 2014 (field survey in the main riverbed).

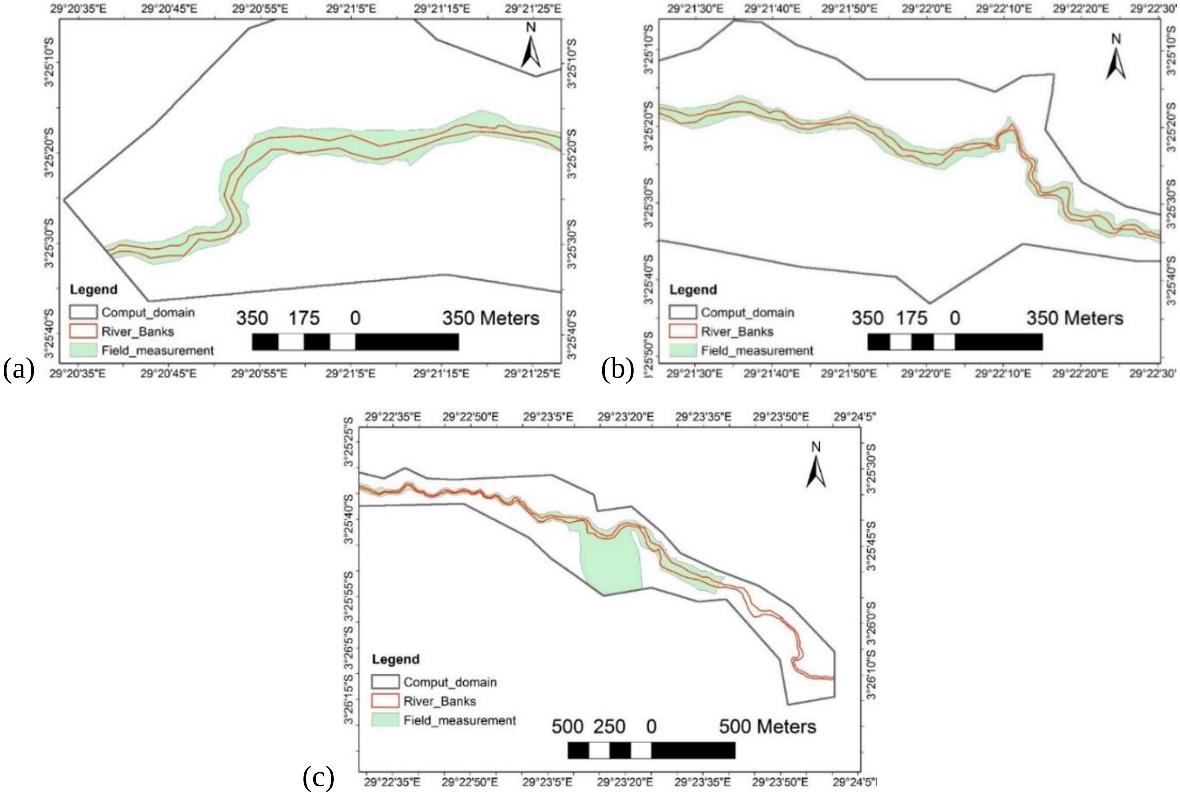

Figure R7. Extent of the field measurements (🟩), of the river banks (—) and of the computational domain: (a) lower part of the valley, (b) middle part and (c) upper part.

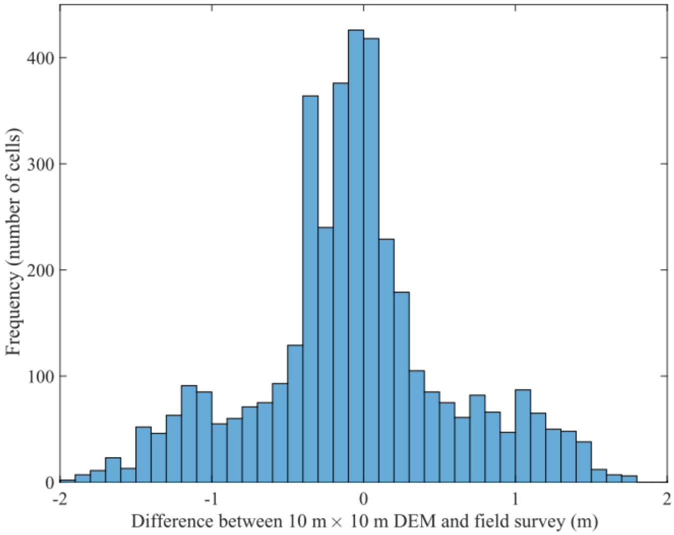

Figure R8. Elevation difference between the topography from field measurement and the resampled 2 m × 2 m DEM used for hydraulic modelling.

We also expect that in the upper part of the valley, which shows a distinctive V-shape with relatively steep lateral slopes, as the flow tends to concentrate in the main canal and its vicinity, the hydraulic modelling results are less affected by small inaccuracies in the DEM than further downstream.

In the revised version of the manuscript, we will:

- discuss the resolution of the original and resampled DEMs with respect to the river width;
- highlight that this study was conducted in a "data-scarce" context and discuss the implications in terms of reliability of the results in both the upper and lower parts of the valley;
- refer to the field survey to appreciate the reliability of the topographic data;
- explicitly state that using higher resolution and updated elevation data (particularly for the river bathymetry) is a necessary next step of this research.

These important points will be added in Section 2.2 (topographic and geophysical data) and in section 4.3. of the revised version of the manuscript. We also propose to report in the revised manuscript on additional simulations performed based on the surveyed topographic data instead of the original 10 m × 10 m DEM for appreciating the sensitivity of the simulation results (peak discharge, inundation extent, water depths) to the inaccuracy in the topographic data.

2.6. The discussion on the uncertainties involved is very short, given that the uncertainty issue is very important when it comes to computer simulations. Some further considerations would be desirable (geotechnical parameters, hydrograph, sediment transport, ...).

In the revised manuscript, we will substantially expand our discussion on the model uncertainties (section 4.3 in the original manuscript), so that it becomes more representative of the whole spectrum of sources of uncertainties. To make the discussion more structured, we will categorize the various uncertainties affecting our results as a function of their cause: *(i)* input data, *(ii)* model structure (i.e. processes which are incompletely represented in the model), *(iii)* model parameters and *(iv)* scenarios. Among others, we will refer to the aspects detailed hereafter.

- The influence of the *topographic and bathymetric data* will be discussed, in line with our response to comment 2.5 above.

- Another major and specific local challenge relates to the *planform variations of the river channel*. The banks of the Kanyosha River, like those of other rivers in Bujumbura, are not stabilized and frequently undergo strong changes due to erosion and anthropogenic disturbances. This results in changes of the river cross section and may affect the flow dynamics.

- The influence of sediment transport and morphodynamics will be discussed in line with our response to comment 2.1.

- Moreover, the characteristic size of the bottom irregularities was observed to vary along the river channel. Therefore, although we tested different values of the friction coefficient in our simulations, uncertainties remain regarding the effect of the *spatial variability in bottom roughness*.

- In our simulations, we assume that the reservoir behind the dam is completely filled when the failure starts. The actual situation could be different, as the breaching may occur before the complete filling of the reservoir. However, in such a case, the severity of the induced flooding would be lower, so that our assumption makes sense from the perspective of risk management. Filling of the reservoir takes about

5.5 hours, 17 minutes and 9 minutes in, respectively, the base flow scenario, the 20-year flood scenario, and the 50-year flood scenario.

- In addition, the *dam breaching mechanism and dynamics* depends on a series of factors related to the resistance of the natural dam. The detailed prediction of this resistance is out of the scope of the present study (in which we *assume* a breach formation time); but it may considerably affect the actual breaching and the induced flood wave. Therefore, in the revised manuscript, we will detail three failure scenarios:

  - relatively slow gradual failure of the dam (60 min), initiated by the flow overtopping the dam after filling of the reservoir;
  - relatively fast gradual failure of the dam (10 min);
  - instantaneous failure (extreme scenario), resulting for example from the occurrence of a major disturbance like an earthquake.

  Intermediate scenarios may also be considered if deemed relevant.

2.7. Flow depth x velocity does not result in m/s^2, but in m^2/s.

Indeed, this was a mistake and it will be corrected in the revised version of the manuscript ("m/s^2" will be replaced by "m^2/s" at Line 200 and Line 202 of the manuscript).

References

Costa, J. E., Schuster, R. L.: Formation and failure of natural dams, Bulletin of the Geological Society of America, 100(7), 1054-1068, 1988.

Fan, X., Tang, C.X., Van Westen, C.J., and Alkema, D.: Simulating dam-breach flood scenarios of the Tangjiashan landslide dam induced by theWenchuan Earthquake, Nat. Hazards Earth Syst. Sci., 12, 3031-3044, 2012.

Froehlich, D.C.: Embankment dam breach parameters and their uncertainties, Journal of Hydraulic Engineering, 134(12), 1708-1721, 2008.

Li, M.-H., Sung, R.-T., Dong, J.-J., Lee, C.-T. and Chen, C.-C.: The formation and breaching of a short-lived landslide dam at Hsiaolin Village, Taiwan— Part II: Simulation of debris flow with landslide dam breach, Engineering Geology, 123 (1-2), 60–71, 2011.

Mergili, M., Schratz, K., Ostermann, A. and Fellin, W.: Physically-based modelling of granular flows with Open Source GIS, Nat. Hazards Earth Syst. Sci., 12, 187-200, 2012a.

Mergili, M., Fellin, W., Moreiras, S.M. and Stötter, J.: Simulation of debris flows in the Central Andes based on Open Source GIS: Possibilities, limitations, and parameter sensitivity, Natural Hazard, 61(3), 1051-1081, 2012b.

Mergili, M., Fischer, J.-T., Krenn, J., Pudasaini, S.P.: R.avaflow v1, an advanced open-source computational framework for the propagation and interaction of two-phase mass flows, Geoscientific Model Development, 10(2), 553-569, 2017.

Peng, M., Zhang, L.M.: Breaching parameters of landslide dams, Landslides, 9(1), 13-31, 2012.

Shrestha, B. and Nakagawa, H.: Hazard assessment of the formation and failure of the Sunkoshi landslide dam in Nepal, Natural Hazards, 82(3), 2029-2049, 2016.

Yang, S.H., Pan, Y.W., Dong, J.J., Yeh, K.C. and Liao, J.J.: A systematic approach for the assessment of flooding hazard and risk associated with a landslide dam, Nat Hazards, 65, 41-62, 2013.