Modelling of Flood Inundation due to Levee Breaches: Sensitivity of Flood Inundation against Breach Process Parameters

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Key Points:

- Flood inundation due to levee breach depends on the final dimensions of the levee breach and the breach location. The final breach dimensions affect the consequence of the levee breach as the amount of water that can flow into the hinterland is the function of the breach dimensions.
- The breach location affects the consequence of the levee breach even if the discharge through the breach is not significantly influenced.
- The process of the levee breaching (breach development) has marginal to no influence on the resulting flood inundation of the hinterland.

Abstract

This paper analyses the sensitivity of flood inundation due to levee breach against breach model parameters. A parametric levee breach model integrated into the 2D hydrodynamic numerical model Telemac-2D is used to simulate a levee breach flood event at Wonji, Ethiopia. Levee breach process parameters are systemically varied to find out their effect on the modelled flood inundation.The analysis of the model results show that the modelled flood inundation is sensitive to the final breach dimensions and breach location. However, the parameters describing the levee breach development have negligible influence on the modelled flood inundation. This implies that final breach dimension and breach location in an event of levee breach are the most important and decisive factors affecting the resulting inundation of the flood plain and as such should be given due consideration when creating flood inundation maps due to levee breach modelled by employing parametric breach model.

1 Introduction

Flood plains along rivers have been traditionally reclaimed since they provide the important resources needed for humans: water and loam soil. In the course of time many flood plains are changed to agricultural fields, residential villages, towns and cities, manufacturing and industrial areas including valuable historical heritages. In many cases such reclaimed flood plains are protected from flooding with levees. The levees, nonetheless, never guarantee absolute protection against floods (Merz et al. 2014) for several reasons. The first reason stems from the design concept. Levees are designed to protect against floods based on specific design values, e.g. a specific design water level, a specific design discharge or an event with as specific probability of occurrence. Independent of the individual design considerations, there is always some remaining probability of exceedance of the design values at any time in the future. In addition, extreme hydro-meteorological events are expected to increase in the future due to climate change. This makes the probability of occurrence of a given event and the exceedance of the design values higher. Secondly, levees, like any other civil structures, deteriorate (settlement etc.) over time (ASCE/EWRI 2011) even with good inspection and maintenance. Furthermore, levees are designed and constructed according to the state of art at the time of construction. And errors during design and construction are possible.

Flood disasters associated with levee breaches are not uncommon if not frequent (ASCE/EWRI 2011). Levee breach flooding often leads to unprecedented economic damage and loss of human lives. Some historical levee breach flood disasters reported in the literature include the 1996 Wonji levee breach flood in Ethiopia (Ahrens (1996), Tadesse & Fröhle (2015)), the 2005 New Orleans flood in USA (Sills et al. (2008), van Emelen et al. (2012)), the 2002 and 2013 levee breaches of Elbe River and its tributaries in Germany (Merz et al. 2014), among others.

This calls for authorities to be prepared for flood events (see e.g. European Flood Directive) and prepare flood hazard maps and emergency plans for communities, businesses, and other stakeholders living in flood plains behind levees so that consequences in case of flooding can be minimized. The preparation of flood hazard maps can be done with numerical flood inundation modelling for different levee breach scenarios. The modelling of flood inundation due to levee breach is better undertaken with a 2D hydrodynamic numerical model that has an integrated model for levee breaching (Dazzi et al. (2019), Apel et al. (2009)). While 2D hydrodynamic numerical flood modelling is a well-established subject, levee breach modelling is

still a research subject (Dazzi et al. (2019), ASCE/EWRI (2011)). Levee breach modelling approaches are described in chapter 2.2.1.

This investigation uses a parametric levee breach model investigated and integrated into the hydrodynamic model Telemac-2D by Tadesse & Fröhle (2015). Parametric breach models, as the name implies, rely on parameters to describe levee breaching processes. The parameters are often specified based on good engineering judgment. It is thus important to analyse the sensitivity of the flood inundation against the breach parameters. To the knowledge of the authors, there are only a handful of studies that deal with this subject. Singh & Snorrason (1984), for example, analysed the sensitivity of outflow discharge due to dam break against breach parameters.

This paper analyses the sensitivity of modelled flood inundation due to levee breach against the levee breach parameters. For this purpose, a historical levee breach flood event at Wonji, Ethiopia (Tadesse & Fröhle 2015) is modelled with a parametric breach model integrated into the hydrodynamic model Telemac-2D. To quantify the sensitivity of the modelled flood inundation against the breach parameters, a number of hydrodynamic numerical simulation for systematically varied breach parameters are carried out. The considered breach parameters are breach location, breach start time, final breach level, final breach width, breach duration, initial breach width, and type of breach (mode of breach).

The results of the analysis show that the modelled flood inundation is sensitive to breach location, final breach level and final breach width, but insensitive or marginally sensitive to breach duration, initial breach width and type of breach. This implies that the breach parameters – breach location, final breach level, and final breach width – need to be chosen for a given application with care and conservatively so as not to underestimate the resulting flood inundation extent and depth, which could have high implication on flood risk assessments.

2 Methodology

The aim of this work is to analyse the effect of levee breach processes on the inundation of flood plain due to levee breach. The approach for the analysis is shown schematically in Figure 1. A hydrodynamic numerical model is set up for levee breach case at Wonji, Ethiopia. A real levee breach case instead of a hypothetical breach case is preferred in order to base our analysis on a real world case. The 1996 Awash River levee breach at Wonji, Ethiopia is chosen as good documentation and description of the levee breaching processes was available to the authors.

For the investigation, the hydrodynamic numerical model Telemac-2D (Hervouet 2007) and a parametric levee breach model integrated into Telemac-2D by Tadesse & Fröhle (2015) are used. The sensitivity of the modelled flood inundation is quantified by systematically varying the breach parameters and comparing the corresponding modelled flood discharge through the breach and flood inundation depth at a selected location in the flood inundation area. In the following sections, further details on the hydrodynamic numerical model Telemac-2D and the breach model are given.

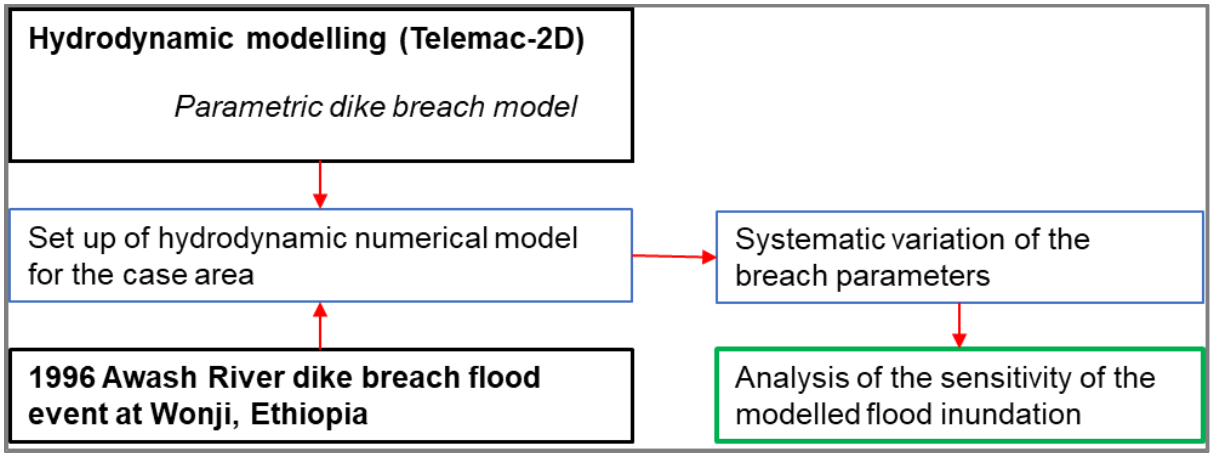

Figure 1. Schematic diagram of the proposed approach for the sensitivity analysis

2.1 Telemac-2D

Telemac-2D solves the hydrodynamic 2D flow equations in the horizontal directions (also known as shallow water equations). It is part of the open Telemac-Mascaret suite of solvers, which are in the public domain. The software suites are developed and maintained by consortium of organisations, of which Electricité de France Research and Development (EDF R&D) is the main developer and maintainer of Telemac-2D (TELEMAC 2019).

Telemac-2D gives the options to discretise the governing equations either with the finite volume or the finite element method. In this investigation, the finite element method in combination with semi-implicit time stepping scheme is used. The resulting algebraic equations are then solved with the conjugate gradient method. For further details on Telemac-2D, the reader is referred to the user manual (Ata 2018) and the book of the major developer of Telemac-2D (Hervouet 2007). Telemac-2D is suitable for modelling levee breach type flows (see Tadesse & Fröhle (2017), Ata (2018), TELEMAC (2019)).

2.2 Levee breach model

2.2.1 Major breach models

There are three major type of breach models: empirical, parametric and physically based breach models (Morris et al. (2009), ASCE/EWRI (2011)). The latter two can model the flow splitting during levee breaches into flow from the river to the hinterland and the flow in the main river channel (Roger et al. 2010) if they are integrated into a 2D hydrodynamic numerical model. In addition, these breach models are suitable for integration into a hydrodynamic numerical model for modelling the resulting flood inundation.

Empirical breach models are commonly used for reservoir dam breaks to calculate the discharge through the dam break as a function of water level in the reservoir, inflow into the reservoir, reservoir height, and area of the opening due to the dam break (ASCE/EWRI 2011). In case of levee breaches, the flow splits into 1) the flow through the breach and 2) the flow remaining in the river channel (Roger et al. (2010), Kamrath et al. (2006)). The flow through the breach is dependent on i) the breach dimensions ii) the river width influenced by the breach iii) the velocity in the river channel and iv) the flow depth in the river channel (Kamrath et al. 2006).

This complicated relationships makes it difficult to calculate the discharge through the breach empirically. 2D hydrodynamic numerical modelling of the flow field is imperative.

Parametric breach models describe levee breaching process by taking simplifying assumptions on the location, initiation, development, number, and shape of the levee breach often based on experience and historical levee breaches. The breach parameters are often number of breaches, breach location, final breach width, final breach level, initial breach width, breach duration (time taken for the breach to its final state), type of breach (piping versus overtopping), etc. (Tadesse & Fröhle (2015), Dazzi et al. (2019)).

Physically-based breach models model the levee breaching process using morphodynamic equations (Dazzi et al. (2019)) coupled with hydrodynamic numerical model. These models couple 2D shallow water equations with sediment transport equations (such as Meyer-Peter and Muller formula) and bed evolution equations (such as Exner equation). Such models are reported by Canelas et al. (2013), Murillo & García-Navarro (2010), Li & Duffy (2011), among others. Although physically based breach models are the right choice from the point of view of modelling the breaching process, their applicability to real cases is limited. The widely available sediment transport and bed evolution equations are not developed for levee breach flow conditions. Moreover, attempts to develop sediment transport and bed evolution equations (erosion laws) for levee breach conditions are limited to levee breaching due to overtopping flow (ASCE/EWRI 2011) and they often require use of parameters (e.g. erodibility parameter (Dazzi et al. 2019)). In this regard, parametric breach models are the suitable approach for practical applications of modelling of flood inundation due to levee breach. In addition, physically based models can be seen a special case of parametric breach models whose breach parameters are determined by a physical based approach (sediment transport and bed evolution equations).

2.2.2 The breach model used in this investigation

The levee breach model used for this investigation is a parametric breach model investigated by and then implemented in Telemac-2D by Tadesse & Fröhle (2015). The breach model simulates the breaching process via input information about the breach location, condition for breach initiation, erosion type, breach duration, initial breach width, final breach width and final breach level. Breach is created by lowering the elevation of the mesh nodes according to the parameters defining the breaching process.

In this model breaches are initiated at pre-defined locations if the conditions for breach initiation are fulfilled. The condition for breach initiation is given by one of the following three options. Breach is started a) at a specified time or b) when the water level is greater than the levee crest level or c) when the water level is greater than a specified value.

Once the breach is initiated, the levee breaching processes is imposed in two ways. The first option (Option 1) is the lowering of the levee level for the entire breach width (final breach width) without consideration of lateral breach growth over the breach duration. In the second option (Option 2), the lowering of the levee level takes place in two steps. First, the levee level for width beginning with the initial breach width at the centre of the final breach width is lowered in tenth of the breach duration to the final breach level. The horizontal breach expansion rate is the same as the second step. Then, horizontal expansion of the breach continuous until the

final breach width over the breach duration is attained. Figure 2 shows a sketch of the final breach dimensions of a levee breach at the indicated position.

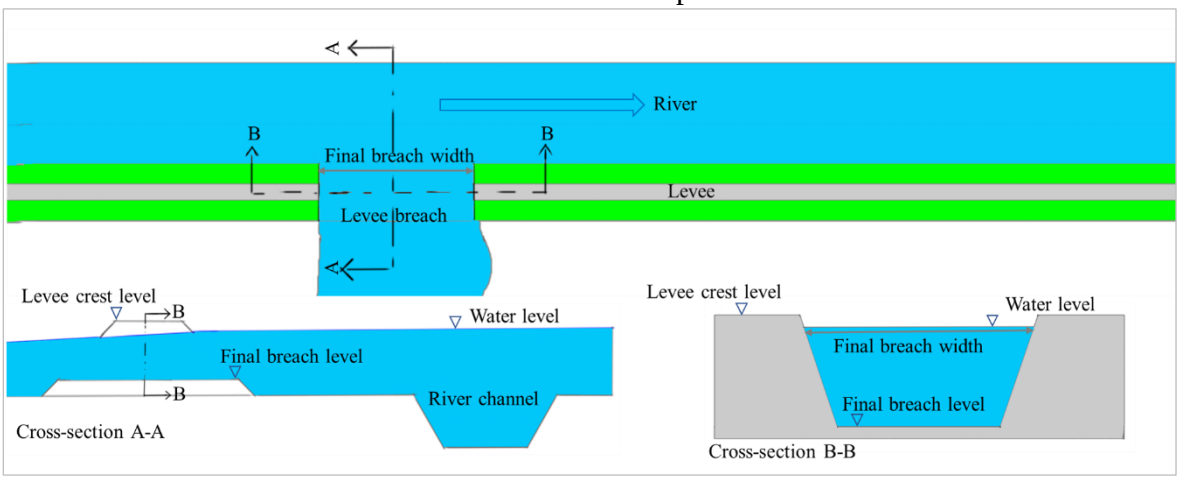

Figure 2. Sketch: Definition of levee breach (not to scale)

2.3 The levee breach case: the 1996 Awash River levee breach at Wonji

2.3.1 The study area

This study uses the 1996 levee breach flood event of Awash River at Wonji, Ethiopia. The flood event inundated Wonji Shoa Sugar Factory (WSSF) and WSSF's sugar cane plantation. WSSF and its sugar cane plantation is found in the upper Awash River basin downstream of Koka Hydropower Dam (KHD) some 100 km south of the capital Addis Ababa near the town of Adama, Ethiopia (Figure 3).

Since Wonji is located only about 20 km downstream of KHD, Awash River flow at Wonji is directly related to water release from the KHD reservoir. In its normal operation the KHD regulates the Awash River flow downstream. However, in exceptional cases of high water inflow into KHD reservoir, water level in the reservoir exceeds the design water level and flood gates of the dam need to be opened to avoid a rather catastrophic phenomena of dam failure. High-water level in the reservoir usually occurs following days-long heavy rainfall in the upstream catchment. Besides, the KHD reservoir has lost much of its capacity to sedimentation (Abebe H. (2001), SHAHIN (1993)). This leads to higher water levels in the reservoir even if the inflow into the reservoir and the outflow from the reservoir are unchanged.

Opening of flood gates during high-water levels in KHD reservoir results in flood flows in Awash River downstream of KHD. This often poses high probability of flooding at Wonji. To decrease the probability of flooding at Wonji, levees have been constructed along the banks of Awash River. About 20 km critical reach of Awash River at Wonji are protected by levees on both banks of the river (Halcrow et al. 2005, p. 34). The levees are simple construction from earth material available in the vicinity with approximate 1:2 (vertical:horizontal) side slopes, bottom width of 9 m, height of 3 m and top width of 3 m (Halcrow et al. 2005, p. 17).

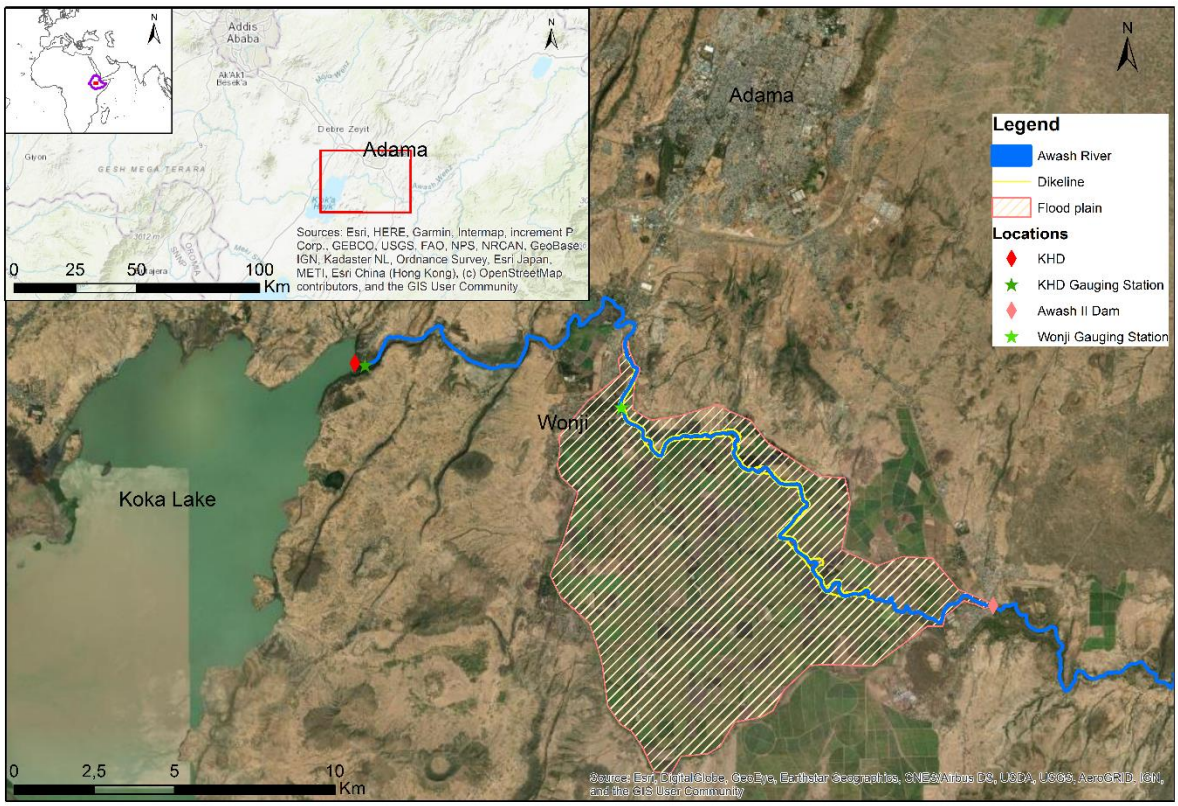

Figure 3. Location of the study area – Wonji-Shoa Sugar Factory and its sugar cane plantation (background map courtesy of ESRI®, ArcGIS online service)

2.3.2 The 1996 Awash River levee breach at Wonji, Ethiopia

Following days-long heavy rainfall in the catchment of KHD in August 1996, the flood level of KHD reservoir reached critical level. To prevent catastrophic failure of the dam, the flood gates of the dam were opened. The released flow caused high flow level in Awash River downstream of KHD and breached levee at Wonji. The levee breach lead to widespread flooding affecting WSSF's sugar cane plantation, residential houses of employees of the factory, offices of the factory and private agricultural works on the left bank of the river.

Data on the levee breach are compiled from personal communication with levee foreman of WSSF (Mr. Desta), reports given by international disaster relief organizations and the government, and news outlets. According to Mr. Desta, an eyewitness of the flood and levee foreman of WSSF, one levee breach occurred during the 1996 Wonji levee breach upstream of the office area (see Figure 3). The levee breach had an approximate breach width 100m and was eroded to the ground level. The breach occurred on 24 August 1996 (Associated Press 1996) and the time taken for the breach to develop to its final state is not documented. A breach time of about an hour is considered realistic. Following the levee breach extensive flooding of WSSF and its sugar cane plantation is reported (Ahrens (1996), Associated Press (1996)). It is documented that flood marks in the office buildings of WSSF reached window beam levels (see Figure 3 for the location of the office building) (Mr. Desta, pers. comm.), which are

approximately 0.8 m above the ground level. In addition, Ahrens (1996) reported wide spread flooding on both banks of the river on 27 August 1996 with increasing water level.

3 Model set up

Hydrodynamic numerical model is set up for the Awash River reach from KHD to Awash II Dam at Awash Melkasa town. The model area encompasses the river channel, the Wonji flood plains on both sides of the river, and the levees on both banks of the river (see Figure 3). Data required for the model set up are obtained from Ethiopian Mapping Agency, The Ministry of Water, Irrigation and Electricity of Ethiopia and WSSF.

3.1 Computational mesh

The model area is discretised into triangular mesh with spatially varying resolution. The levee is discretised by at least three mesh nodes between the levee foot and crest. The river is discretised by more than ten mesh nodes over the cross-section. The flood plain is discretised with mesh element size of about 1.5 ha. In total the computational mesh has about 725 000 triangular mesh elements. Figure 4 shows excerpts of the computational mesh.

Topographic data of the model area is obtained from two sources. For the flood plains, geo-referenced topographic maps are obtained from Ethiopian Mapping Agency. According to this topographic map, the flood plains in Wonji lie approximately at 1538 m a.m.s.l.. For Awash River and the levees, river cross-sections of Awash River for about 20 km length of the river in Wonji area (for the reach where levees are built) are obtained from Civil Engineering section of WSSF (see Figure 3). For the part of the river without cross-section data, the data gap is filled by interpolation respectively extrapolation of the existing cross-section data.

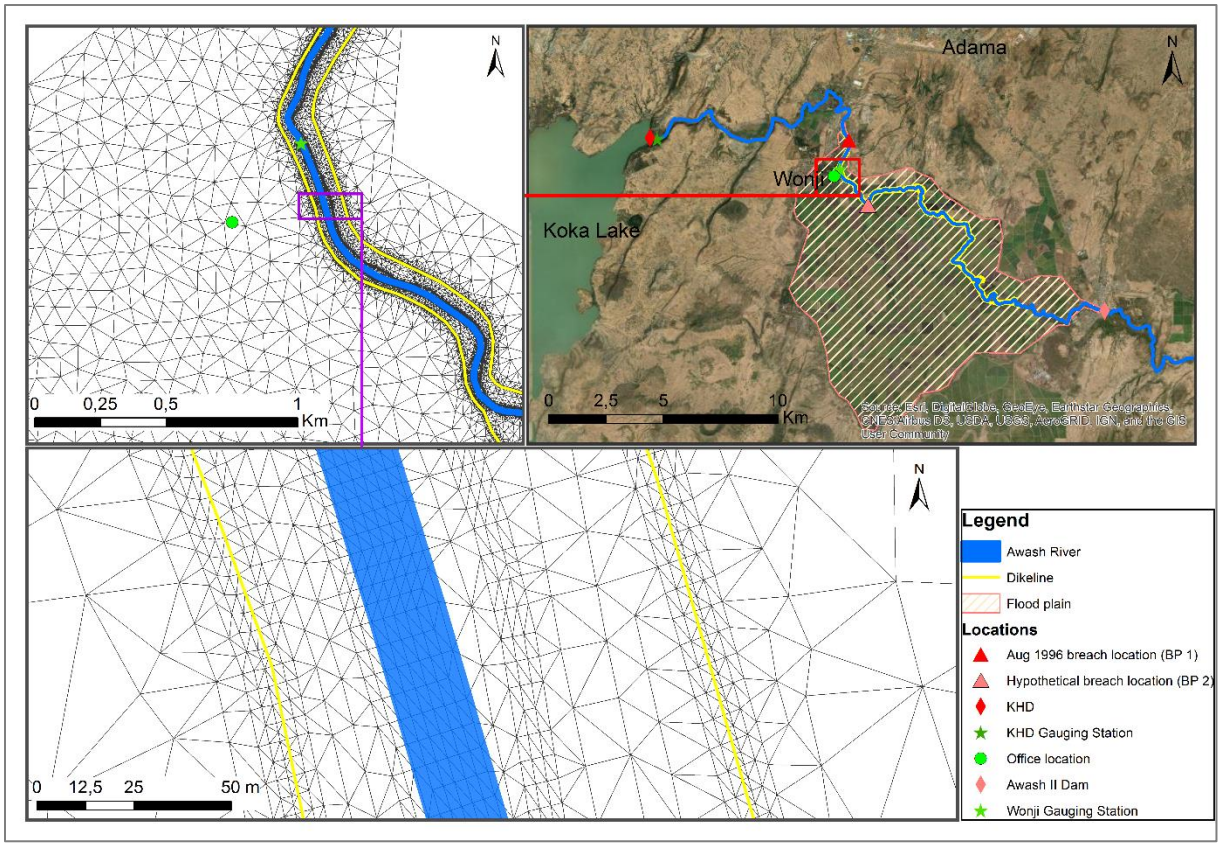

Figure 4. Excerpt of the computational mesh

3.2 Boundary and initial conditions

The upstream boundary of the model area is KHD (see Figure 3). Discharge measurement at KHD gauging station is used as the upstream boundary condition. Figure 5a – c show hydrographs of three flood events at KHD as well as Wonji gauging stations.

The downstream boundary of the model area is Awash II Dam (see Figure 3 and Figure 4). Awash II Dam is a concrete gravity dam about 16 m high and 88 m long, and has a storage capacity of 6 Mm³ (International Bank for Reconstruction and Development 1964). The water level at maximum capacity is 1539.04 m a.m.s.l. The dam operates like a weir and is overflown when the water level exceeds the maximum level. Thus, the downstream boundary condition of the model is a stage-discharge relationship. Since exact stage-discharge curve for the weir is unavailable, stage-discharge relationship is derived using the overflow formula for broad-crested weirs given by equation (1) (Jiang et al. (2018), Tracy (1957)).

$$Q = \frac{2}{3} bCd\sqrt{g}h_o^{3/2} \quad (1)$$

In equation (1), Q, b, Cd, g and h_o stand respectively for overflow discharge, width of the weir, discharge coefficient, acceleration due to gravity and upstream total head. The discharge coefficient considers losses due to friction, the effect of the upstream slope of the weir, shape of the weir crest, among other factors. Experiments show that the discharge coefficient for broad-crested weir can be very low (Jiang et al. (2018). An average discharge coefficient value of 0.55

is adopted. The upstream total head h_o is approximated by the upstream water depth over the crest level. The width of the dam is 88m. Therefore, the stage-discharge relationship at Awash II Dam is established by calculating the values of stage (depth over crest plus the crest level of the dam) for various Q values and is shown in Figure 5d.

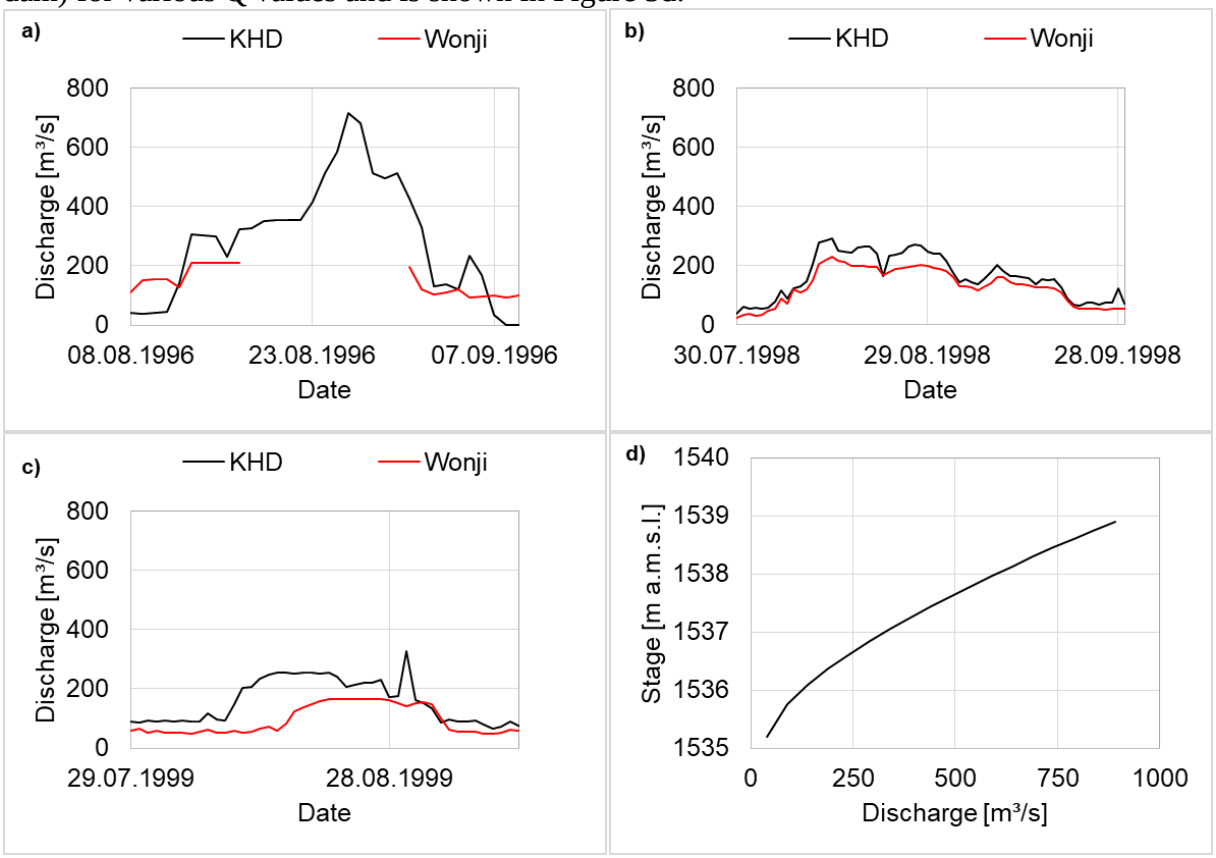

Figure 5. Awash River flow data a) hydrograph of the August 1996 flood flow at KHD and Wonji gauging stations b) hydrograph of the summer 1998 flood flow at KHD and Wonji gauging stations c) hydrograph of the August 1999 flood flow at KHD and Wonji gauging stations d) stage-discharge relationship of Awash River at Awash II derived using overflow equation for broad-crested weir.

Besides the boundary conditions, initial conditions need to be specified. Initial conditions specify the values of the variables (depth and velocity of flow) at the start of the simulation. For all simulations in this work, water depth of 4 m in the river channel and 0 m for the rest of the model domain, and velocity of 0 m/s for the entire domain is set as initial condition. All simulations are run with a sufficient warm up phase of several days to minimize the effect of the initial conditions.

3.3 Model calibration and validation

The model is calibrated against the flood event in August 1998. The model calibration parameters are bottom resistance and turbulence loss coefficients. For the simulations bottom resistance loss law of Nikuradse and constant eddy viscosity turbulence model are used. In the process of calibration, the river bottom roughness (Ks-value) and the eddy viscosity coefficient (Vis.) are varied until good agreement between the measured and the simulated discharge at

Wonji gauging station are achieved. Figure 6a shows the simulated discharge hydrographs for different Ks-values and the measured discharge at Wonji gauging station, and Figure 6b shows the simulated discharge hydrographs for different eddy viscosity coefficients and the measured discharge. As can be observed from the figures, the model replicates the measured discharge very well for this flood event. For further use, we adapted Ks-value of 0.1 m and eddy viscosity value of 2.0 m²/s.

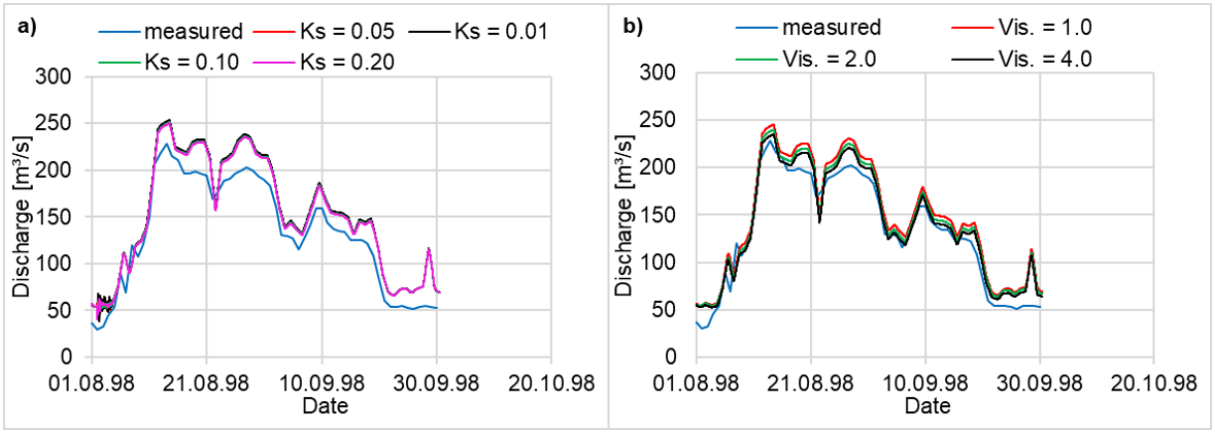

Figure 6. Model calibration: measured discharge hydrograph of summer 1998 at Wonji gauging station a) compared with modelled discharge hydrographs for different Nikuradse's roughness Ks-values b) compared with modelled discharge hydrographs for different eddy viscosity values (Vis.).

The model is validated against the August 1996 flood event that caused the levee breach and the extensive flooding at Wonji, Ethiopia. The modelled water depth at the office location corresponding to the available breach information (see section 2.3) is given Figure 7. The modelled maximum water depth at the office location is 0.87 m. This is in good agreement with the eyewitness information that the flood level at the office location during the flood event reached window beam levels. The window beam levels are at 0.90 m above the ground level. In addition, the modelled water depth is increasing on 27 August 1996 which is in agreement with the report of Ahrens (1996).

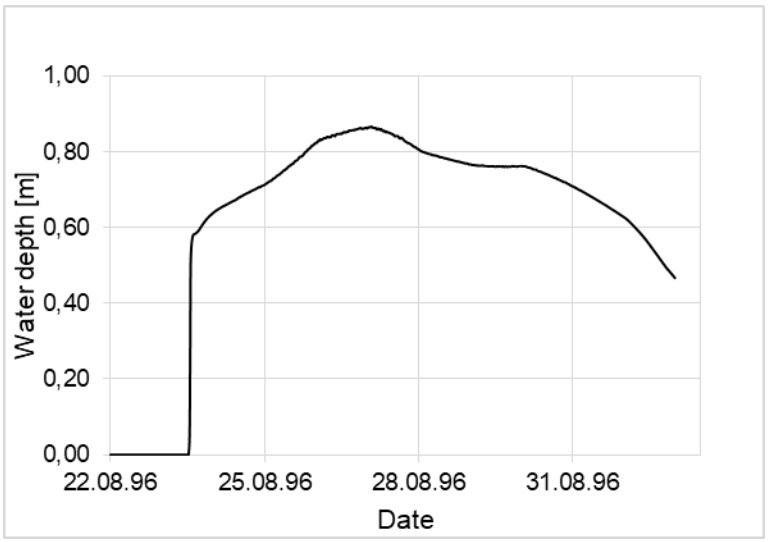

Figure 7. Model validation: Modelled water depth at the office location corresponding to the breach information

4 Sensitivity analysis of the Breach model parameters

As described in section 2.2, the levee breach model used in this study uses breach parameters to describe the levee breaching processes. The major parameters are breach duration, breach start time, final breach width, final level of breach and breach growth mode. For real levee breaching cases, these parameters are associated with high uncertainty as they depend on factors such as characteristic of levee material, flow in the river, maintenance situation of the levee, quality of construction, etc. As a result, it is essential to analyse the sensitivity of the modelled flood inundation and extent to each of these breach parameters. This helps to find out the parameters that one should give due attention when using parametric levee breach models for flood inundation modelling. In the following sections, the sensitivity of the modelled water depth in the hinterland (at office locations, see Figure 3 for the location) and the discharge through the breach (breach discharge) to each of the major breach parameters is analysed.

4.1 Breach duration

To determine the sensitivity of the modelled water depth in the hinterland and the modelled breach discharge to breach duration, the breach duration is varied within the range of the historical breach duration of 1 hour. Numerical simulations for breach duration of 30 min, 1 hour and 2 hours were undertaken. Water depth in the hinterland and breach discharge obtained from the model runs for the respective breach durations (BD) are compared in Figure 8. The results show that the modelled flood inundation is insensitive to breach duration.

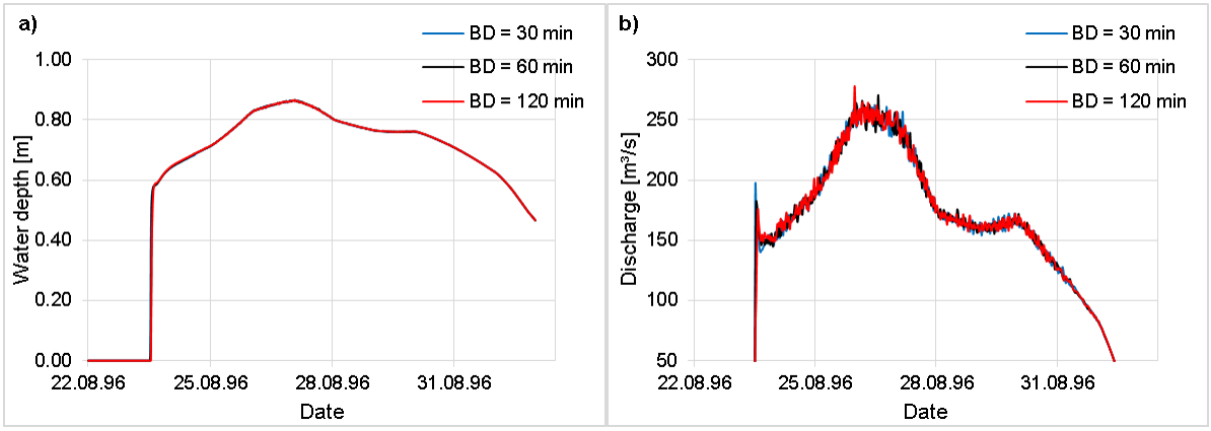

Figure 8. For the indicated levee breach durations (BD) and the historical levee breach location of the August 1996 levee breach flood event at Wonji a) modelled water depth at an office location b) modelled breach discharge. The office location and the historical breach location are shown in Figure 3.

4.2 Breach start time (breach initiation)

Breach initiation options of the levee breach model are highlighted in section 2.2.2. For the analysis here, breach initiation by specifying breach start time is used. This is to have control on the breach start time which helps to make the comparison of the sensitivity of the modelled water depth in the hinterland and the modelled breach discharge against breach start time definite.

To determine the sensitivity of the modelled water depth in the hinterland and the modelled breach discharge to breach start time, hydrodynamic numerical simulations for three breach start times (BT) at 00:00 am, 03:00 am and 06:00 am on 24.08.1996 are carried out. Water depth in the hinterland and breach discharge obtained from the model runs for the respective breach start times are compared in Figure 9. As can be observed from Figure 9, for the breach start times analysed here, the breach start time affects only the start of inundation and the peak inundation depth and breach discharge are insensitive to the breach start time.

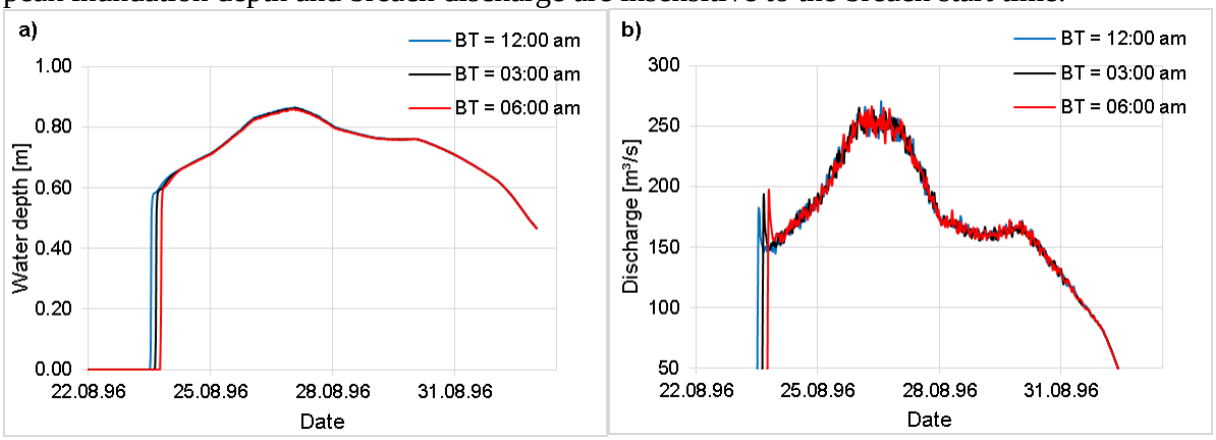

Figure 9. For the indicated levee breach start times (BT) on 24.08.1996 and the historical levee breach location of the August 1996 levee breach flood event at Wonji a) modelled water depth at

an office location b) modelled breach discharge. The office location and the historical breach location are shown in Figure 3.

4.3 Final breach width

The sensitivity of the modelled water depth in the hinterland and the modelled breach discharge to final breach width is analysed by running hydrodynamic numerical model simulations for final breach width (BW) of 50 m, 100 m and 150 m. The final levee breach width of the August 1996 levee breach is about 100 m. Water depth in the hinterland and breach discharge obtained from the model runs for the respective final breach widths are compared in Figure 10. As can be observed from Figure 10, the modelled water depth at the office locations and the modelled breach discharge increase with increasing final breach width.

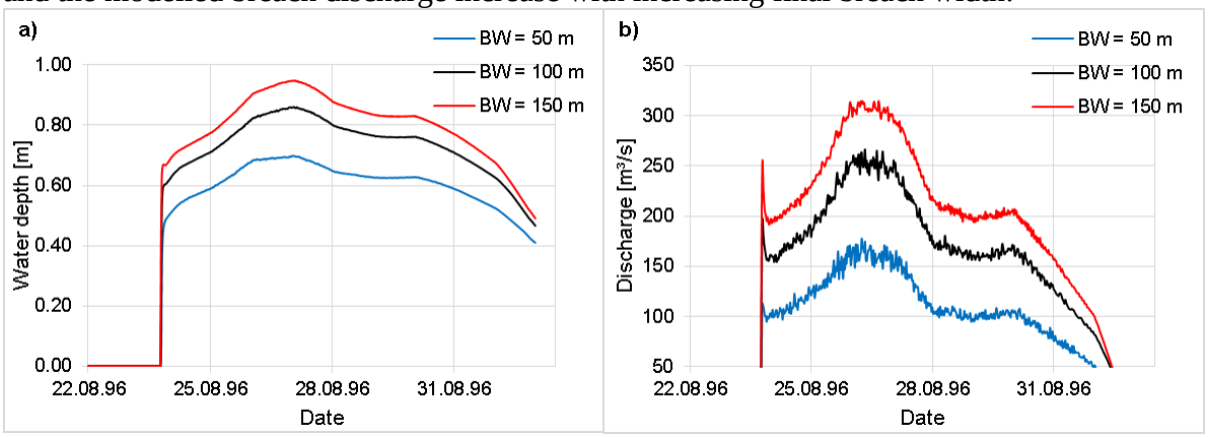

Figure 10. For the indicated final levee breach widths (BW) and the historical levee breach location of the August 1996 levee breach flood event at Wonji a) modelled water depth at an office location b) modelled breach discharge. The office location and the historical breach location are shown in Figure 3.

4.4 Final breach level

In an event of a levee breach, the levee may erode only to some level and not to the ground level depending on the prevailing hydrodynamic and morphodynamic conditions. The sensitivity of modelled water depth in the hinterland and breach discharge to final breach level is analysed by running hydrodynamic numerical simulations for final breach levels BL = 0 m (levee erodes to the ground level) and BL = 1.5 m (levee erodes to half its height, which is 3 m). The simulation results are shown in Figure 11. As can be observed from Figure 11, the modelled water depth at the office location as well as the breach discharge increase as function of the final breach level. Simulations for further final breach levels are avoided as the results with the two cases show the sensitivity clearly.

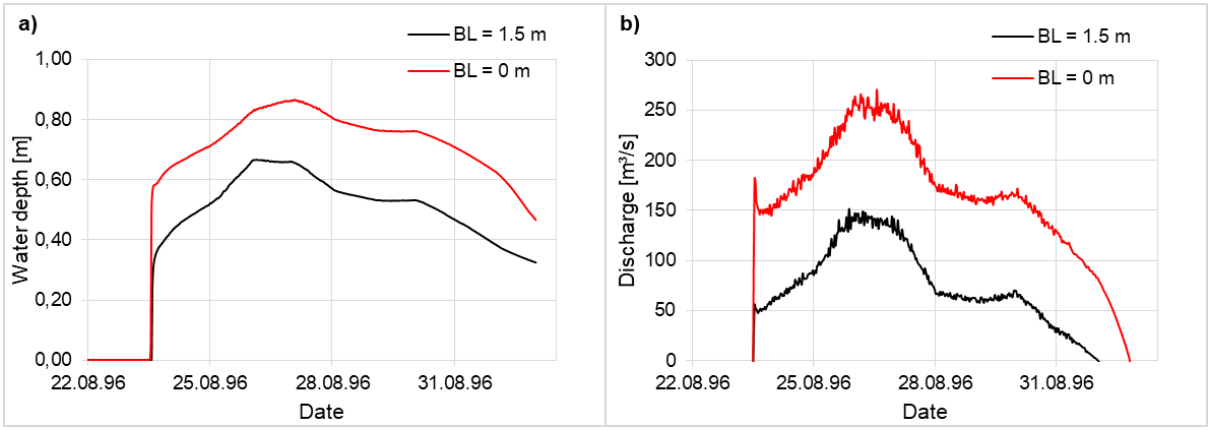

Figure 11. For the indicated final breach levels (BL) (BL = 0 m (levee erodes to the ground level) and BL = 1.5 m (levee erodes to half its height, which is 3 m)) and the historical levee breach location of the August 1996 levee breach flood event at Wonji a) modelled water depth at an office location b) modelled breach discharge. The office location and the historical breach location are shown in Figure 3.

4.5 Breach location

The sensitivity of modelled water depth in the flood plain and breach discharge to breach location is analysed by running hydrodynamic numerical simulations for two breach locations (BP 1 and BP 2) shown in Figure 4. Water depth at the office location in the flood plain and breach discharge from the simulations results are extracted and shown in Figure 12. The results show that water depth at the office location is sensitive to the beach location, but breach discharge is only marginally sensitive to breach location. Simulations for further breach locations are avoided as the results with the two cases show the sensitivity clearly.

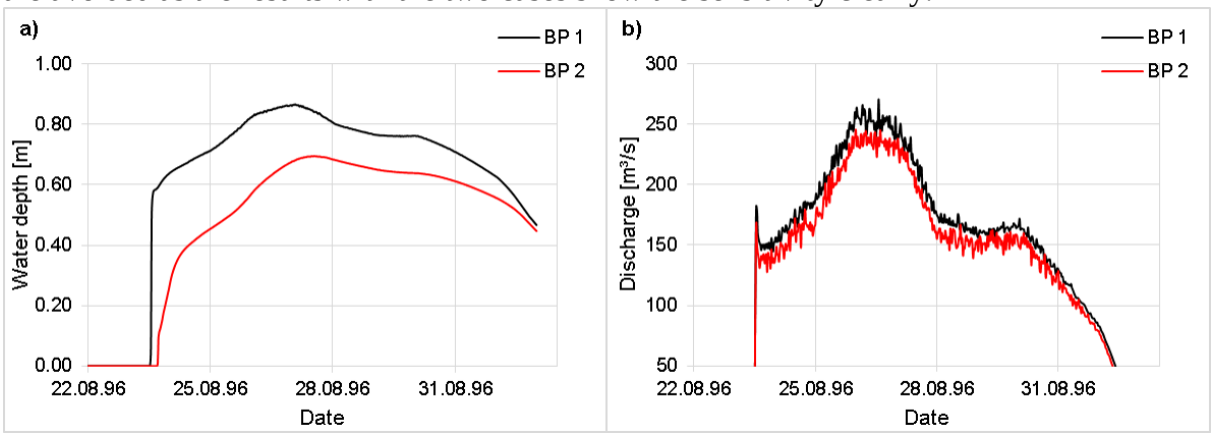

Figure 12. For the two breach locations (BP 1 and BP 2) shown in Figure 4 and the August 1996 Awash River flood event a) modelled water depth at an office location b) modelled breach discharge. The office location is shown in Figure 3.

4.6 Erosion type

The levee breach model has two levee lowering options (erosion options) (see section 2.2). The sensitivity of the modelled water depth at the office location and breach discharge to

the levee lowering options are tested by running hydrodynamic numerical simulations. The water depth at office location and the breach discharge are extracted from the simulations results and shown in Figure 13. The results of the simulations show that the levee lowering method has influence neither on the water depth at office location nor the breach discharge.

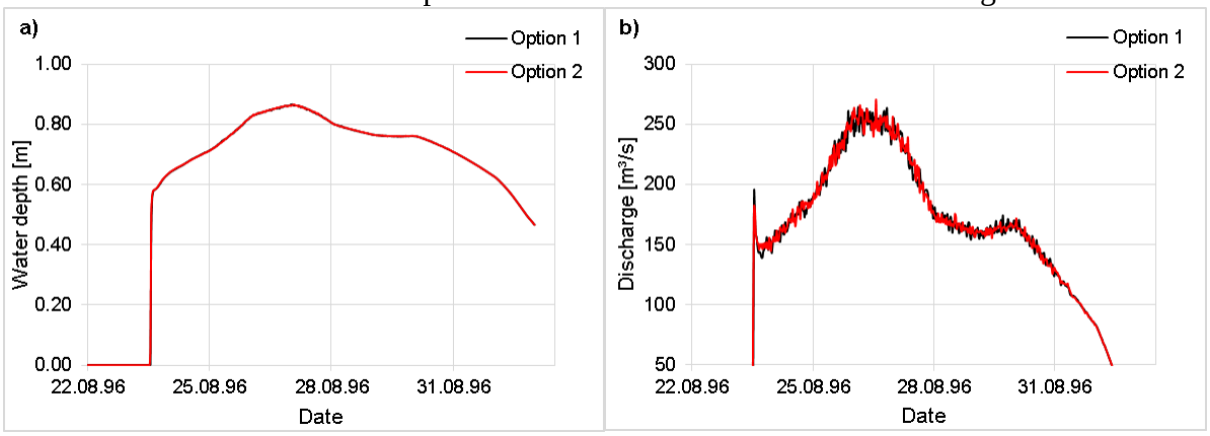

Figure 13. For levee lowering options (Option 1: only vertical breach growth, Option 2: vertical and lateral breach growth) and the historical levee breach location of the August 1996 levee breach flood event at Wonji a) modelled water depth at an office location b) modelled breach discharge. The office location and the historical breach location are shown in Figure 3.

5 Discussion

Parametric breach models are frequently used for analysis of flood inundation due to levee breaching (ASCE/EWRI 2011). Levee breaching is simulated using parameters that define the breach location, the breach development and the initial and final breach dimensions. Parametric levee breach models coupled with 2D-hydrodynamic models are used to reconstruct historical levee breach flood and to map flood hazard based on scenario levee breach events. The number of parameters vary from one breach model to another and can be up to dozens. Nevertheless, the importance of some of the parameters is high with regard to the flood inundation in the hinterland.

Two important aspects can be observed from the results presented in chapter 4. First, the model results (Figure 8 - Figure 13) show that flood inundation of the flood plain due to levee breach modelled with a parametric levee breach model integrated into a hydrodynamic numerical model depends on the breach parameters final breach width, final breach level and breach location. This means that flood inundation due to levee breach is rather the function of the final breach dimensions (final breach width and final breach level) and the breach location. The final breach dimensions strongly influence the flow through the levee breach (breach discharge), in other words, the volume of water getting into the flood plain. Thus, breach model parameters that influence the breach discharge also affect the depth and extent of inundation of the flood plain proportionally.

Breach location affects the spatial variation of the inundation depth without affecting the breach discharge significantly. Thus, breach location affects the resulting consequence of levee breach flooding. To determine the maximum consequence resulting from levee breach, many levee breach locations should be systematically analysed. This also helps for prioritising levee

maintenance and strengthening purposes. The levee segment with maximum consequence in case of breach is to be maintained and strengthened first.

Two, the levee breach parameters describing the breaching processes (breach development), these are breach start time, breach duration and levee lowering method (erosion type), have no or only marginal influence on the resulting modelled flood inundation. Hence, the breach development has no significant influence on the resulting flood inundation of the hinterland.

Thus, the determination of final levee breach dimensions and location are critical for flood hazard mapping of the hinterland. Reliable and accurate methods and models for forecasting breach location and final breach dimensions are unavailable. It is emperative that it such methods and modells are developed. Physically based breach model can play significant role in this regard. But for this, the focus of physically based methods should not be only on exact modelling of the levee breaching process using sediment transport and erosion rate equations, as it is often encountered in the literature, but also on accurate estimation of the final breach dimensions. In addition, methods for reliable estimation of breach location should be developed. This aspect is often neglected in the literature.

The presented in this paper apply to river floods lasting days long. For flood events lasting for short period such as flash floods, the levee breaching processes can have significant influence on volume of water flowing into the hinterland and thus affect the depth and extent of inundation.

6 Conclusion

In this paper, the sensitivity of flood inundation due to levee breach against breach parameters of a parametric levee breach model integrated into the 2D hydrodynamic numerical model Telemac-2D is systematically analysed using the August 1996 Awash Levee breach flood event at Wonji, Ethiopia. From the analysis of the model results, the following conclusions are drawn.

- Flood inundation due to levee breach depends on the final dimensions of the levee breach and the breach location. The final breach dimensions affect the consequence of the levee breach as the amount of water that can flow into the hinterland is the function of the breach dimensions.
- The breach location affects the consequence of the levee breach even if the discharge through the breach is not significantly influenced.
- The process of the levee breaching (breach development) has marginal to no influence on the resulting flood inundation of the hinterland. This applies however to floods lasting over days. Consequences of levee breaches due to flash floods lasting for short period could be sensitive to the levee breaching processes.
- Accurate and reliable methods for determining the final breach dimensions and breach location are important to augment parametric breach models.

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