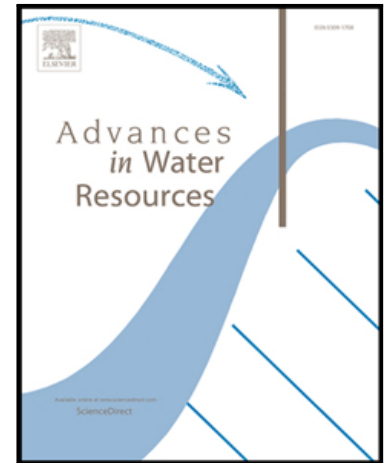


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Highlights

- The flow surface patterns close to the breach differ from those below the surface for all tested cases.
- Strong secondary flows is generated by the flow separation near the breach and breach enlargement alters the vertical flow dynamics in the main channel.
- The lateral velocity at the levee breach cross-section shows a complex, non-uniform distribution

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Corresponding Author: Xuefang Li

Guangxi University

China

ABSTRACT

River dike failure events can have catastrophic consequences for the areas near the dike breach. Overtopping is one of the main causes of dike failures in rivers and understanding the complex flow dynamics in such cases is of vital importance for the modelling of dike failure processes and the prediction of the breach discharge that is required for flood warning systems and evacuation plans. This study investigates the intricate flow dynamics in different instances of an expanding dike breach due to overtopping with the aid of a 3D nonhydrostatic numerical model that solves the Reynolds-averaged Navier-Stokes equations. The dike breach evolving geometry was taken from a previous experimental study, which the present study complements by investigating the associated flow patterns. The numerical model was validated by reproducing fairly well the flow depth at the upstream part of the main channel (difference of $\sim 6\%$ on average) and discharge partition at the breach (difference $\sim 6\% - 15\%$). The flow surface patterns at upstream main channel differ from those below the surface for all tested cases. Specifically, the velocity magnitude close to free surface is approximately 50% to 70% higher than that near the bottom. Breach-induced secondary flow is observed at both horizontal

and vertical cross-sections downstream of the breach while the distribution of velocity in the transverse direction is not uniform. Meanwhile, the level of water surface increases along the channel direction at the levee breach at the earlier stage of breaching. The expansion of the breach increases the turbulent kinetic energy peak locally near and downstream of the breach depending on the breach size. In addition, the simulated wall shear stress at the breach generally surpasses the critical erosion threshold of the dike material, and the zone of high shear stress extends downstream as the breach widens. This uneven distribution of shear stresses explains the asymmetrical lateral expansion of the breach. This study shows the complexity of the flow patterns, turbulent characteristics and the exerted wall shear stress during a dike failure process. From a practical perspective, 3D numerical simulations provide information that is challenging to measure in physical experiments and could also be used to calibrate lumped or 2D dike failure models, which are typically used in real-world applications.

Keywords: Dike Failure; Overtopping Flow; 3D Numerical Model; Flow Patterns; Turbulent Flow; Wall Shear Stress

1 Introduction

Dike failures can induce severe flood events that often result in substantial ecological, societal and economical losses, and even human casualties (Vorogushyn et al. 2010). Recent compilations in international databases have recorded over 1,500 significant levee failure events worldwide, in which over 800 events occurred after 1940, exposing millions of people and critical infrastructure to inundation hazard (Özer et al. 2020). For instance, the 2015 dike failure on the Kinu River in Japan has caused critical property damage and widespread flooding across Joso City (Yasuda et al. 2016). More recently, dike failure along the Dongting Lake in the year 2024 resulted in a 226 m breach, leading to the inundation of approximately 50 km² (Lu et al. 2025). Numerous dike failure incidents have demonstrated the critical importance of thoroughly understanding their causes, characteristics and impacts (Lu et al. 2025, Maranzoni et al. 2022, Thieken et al. 2016), and the necessity to quantify accurately the discharge through a dike breach (Schmitz et al. 2024, Schmitz et al. 2025). Meanwhile, with the increasing frequency of extreme weather events (Gimeno et al. 2022) and the growing urbanization close to fluvial dikes (Ding et al. 2023) the potential risk associated to dike failure events has risen significantly posing considerable threats to public safety and property security.

The causes of dike breaches are multifaceted and include both natural processes (such as heavy rainfall, overtopping, and natural soil erosion) and human-induced factors (such as design flaws and inadequate maintenance) (Asghari Tabrizi et al. 2016, Danka and Zhang 2015, Özer et al. 2020, Schmitz et al. 2021, Zhang et al. 2024). The interplay of these factors may lead to a decrease in the structural integrity and stability of dikes, ultimately resulting in catastrophic consequences. Asghari Tabrizi et al. (2016) investigated the effect of compaction on the failure of a homogenous non-cohesive embankment, and concluded that the peak discharge decreased with compaction level and the time to peak discharge increased. Schmitz et al. (2021) investigated the effect of dike geometry on overtopping-induced dike failure breach discharge and widening, and showed that the dike geometry influences breach expansion mostly during the initial stage when rapid erosion occurs, highlighting the complex, non-linear nature of the breaching process. There is a need for a better understanding of dike failure mechanics to develop sound early warning systems and evacuation plans (Zhou et al. 2019, Zhu et al. 2010). Thus, exploring the flow dynamics during an overtopping induced-dike failure could significantly contribute to this objective.

1.1 Complexity of flow patterns in dike failure

The dike failure is a complex process that involves hydraulic and geotechnical processes (Amaral et al. 2020). Extensive research efforts have been devoted to understanding dike failure, which can be primarily categorized into prototype observations physical model experiments, and numerical simulations (Schmitz et al. 2021, Schmitz et al. 2023). In actual dikes, the unpredictable nature of real failures makes isolating external factors and achieving large-scale, high-precision measurements often infeasible. As a result, data of field observations remain relatively scarce (Schmitz et al. 2021, Sun et al. 2017).

Compared to costly prototype observations, physical modelling experiments in laboratories offer an efficient approach to explore breaching dynamics under controlled conditions, which provides detailed and valuable data for analysis and validation of numerical modelling (Li et al. 2021, Mignot et al. 2019, Zhang et al. 2024). In physical experiments, research usually focuses

on the extent of breaching and the erosion development (Alvarez et al. 2025, Jónatas et al. 2025), the erosion mechanics (Wei et al. 2016), the discharge distribution (Ashraf et al. 2019) and the associated flow patterns (Michelazzo et al. 2015). Despite the advances in measurement technologies, several limitations still hamper the acquisition of comprehensive data during physical experiments. The geometry of the dike and the flow dynamics continuously evolve during a dike failure process, posing challenges beyond the capability of current measuring systems. Al-Riffai (2014), Wu and Wang (2007) highlight that during rapid breach erosion, intense energy dissipation and non-equilibrium flow cause velocity profiles to deviate from the classical logarithmic distribution, complicating shear stress calculations. In addition, the breach geometry can be particularly irregular (Rifai et al. 2017) further complicating its effect on the flow. Therefore, there is a lack of data for a comprehensive understanding of the flow dynamics associated with dike failures. In addition, laboratory experiments can be costly and time-consuming, and as a result they can only represent a limited number of scenarios.

Numerical models can complement the measurement limitations by providing insights into processes that are not easily measured, while they can also extend the parametric space by testing different boundary conditions and schematizations. 2D models that couple flow hydrodynamics with breach evolution have been widely used and have substantially advanced the understanding of dike failure processes and emergency decision-making. These models have helped to clarify the occurring hydraulic and geotechnical mechanisms, as well as the role of sediment transport and bed-level changes (Dazzi et al. 2019, Orlandini et al. 2015, Viero et al. 2013, Wu et al. 2012). However, most 2D models represent dike breaches using simplified prismatic geometries (Wang and Bowles 2006, Wu and Li 2017). Though it is possible to project complex geometries in 2D models, a substantial loss of 3D geometric information and their associated flow patterns would still occur. As a result, 2D models cannot fully capture the flow patterns that develop during dike failure, which highlights the necessity for 3D numerical modelling. 3D numerical simulations can represent realistic geometries and resolve complex 3D flow structures (Jeong et al. 2025, Li et al. 2024), providing more physically meaningful information on the flow patterns across the depth (Kang et al. 2023). For example, Jeong et al.

(2025) conducted a series of 3D numerical simulations of dike breaching with simplified breach geometry under various Froude numbers in order to estimate the corresponding landward slope velocity, which is essential for predicting the occurrence and intensity of erosion during overtopping events. Although 3D models are more computationally demanding than 2D approaches, their use in combination with realistic dike geometries remains essential for advancing the understanding of dike breach processes. While such detailed models have been used to simulate flow through frontal dam breaks with realistic geometry (Maranzoni and Tomirotti 2023), their utilization in flow through lateral dike breaching has been solely to simple and idealized geometries.

1.2 Objective of the study

This study aims to investigate the flow patterns during a dike failure event due to overtopping with 3D numerical modelling. The novelty of the study lies on the realistic morphology of the dike breach at different phases of an overtopping event. For this purpose, a sequence of six selected digital elevation models during a complete dike breach event was used from the experimental study of Rifai et al. (2019), which monitored the three-dimensional evolution of a breaching dike geometry due to overtopping, but did not capture in detail the flow patterns. These six representative instantaneous conditions of the dike breach evolution from Rifai et al. (2019) are used here to couple the dike breach geometry with the associated flow velocities and stresses across the flow domain. By resolving the intricate 3D flow structures triggered by evolving geometries, this research provides a more physically grounded insight into the highly unsteady and complex hydraulic conditions that traditional models often fail to capture.

This paper is organised as follows: Section 2 presents the setup of the numerical model along with a brief explanation of the physical model experiment upon which it is based. The analysis of the results is provided in Section 3, followed by a discussion of the findings in Section 4. The main conclusions are summarized in Section 5.

2 Methods

2.1 Laboratory setup of Rifai et al. (2019) and selected cases for numerical simulation

The present study models the flow in the dike-breaching experiments of Rifai et al. (2019). Only a brief summary of the experimental setup and approach is provided herein, and the interested reader is referred to Rifai et al. (2019) for more detailed information. The experiments were conducted at the Laboratory of Engineering Hydraulics at the University of Liège in Belgium (Figure 1). The model consisted of a straight flat-bottomed and cross-sectionally trapezoidal main channel with an erodible trapezoidal sand dike that runs parallel to the main channel and separates it from the floodplain. The length of the main channel was 10.3 m, and its top width was 1 m. The erodible dike had a length of 3 m, and it was followed by an unconstrained floodplain (4.3 m $\times$ 2.5 m). In every test, the dike's height was 0.3 m, the crest was 0.1 m wide, and the inner and outer face slopes were 1:2 (V:H) (Figure 1b). The bottom of the main channel and the floodplain were at the same level. The dike material corresponds to non-cohesive, uniform sand with $d_{16} = 0.84$ mm, $d_{84} = 1.24$ mm, and a median diameter $d_{50} = 1$ mm. The sand had a density $\rho_s = 2500$ kg/m³, a bulk density $\rho_b = 1600$ kg/m³ and a porosity $p = 0.36$. In addition, the flume and floodplain were covered with impermeable whitewash coatings to ensure roughness continuity between the flume, floodplain and sand dike. A steady inflow was imposed at the inlet of the main channel in every test. A perforated plate was used at the outlet of the main channel as a flow regulating system. A drainage system was placed under the dike to prevent seepage flow from reaching the dike floodplain toe. To trigger the overtopping, an initial notch with a width of 0.1 m and a depth of 0.02 m was cut in the dike crest at 0.85 m from the upstream side of the dike. Such a notch had the minimum necessitated size to trigger the failure with a minimal disturbance.

During the test, ultrasound sensors were used to record water levels continuously at five locations (i.e., G1, G2, G3, G4, G5 in Figure 1a). The evolving dike topography was monitored using a laser profilometry technique (LPT), i.e. a non-intrusive method that involves sweeping a laser plane, emitted by a laser line projector, over the dike while processing images were captured by a video camera positioned above it. The accuracy of the applied LPT technique is

mostly below 10 mm, with more detailed accuracy assessment of the LPT explained in Rifai et al. (2020).

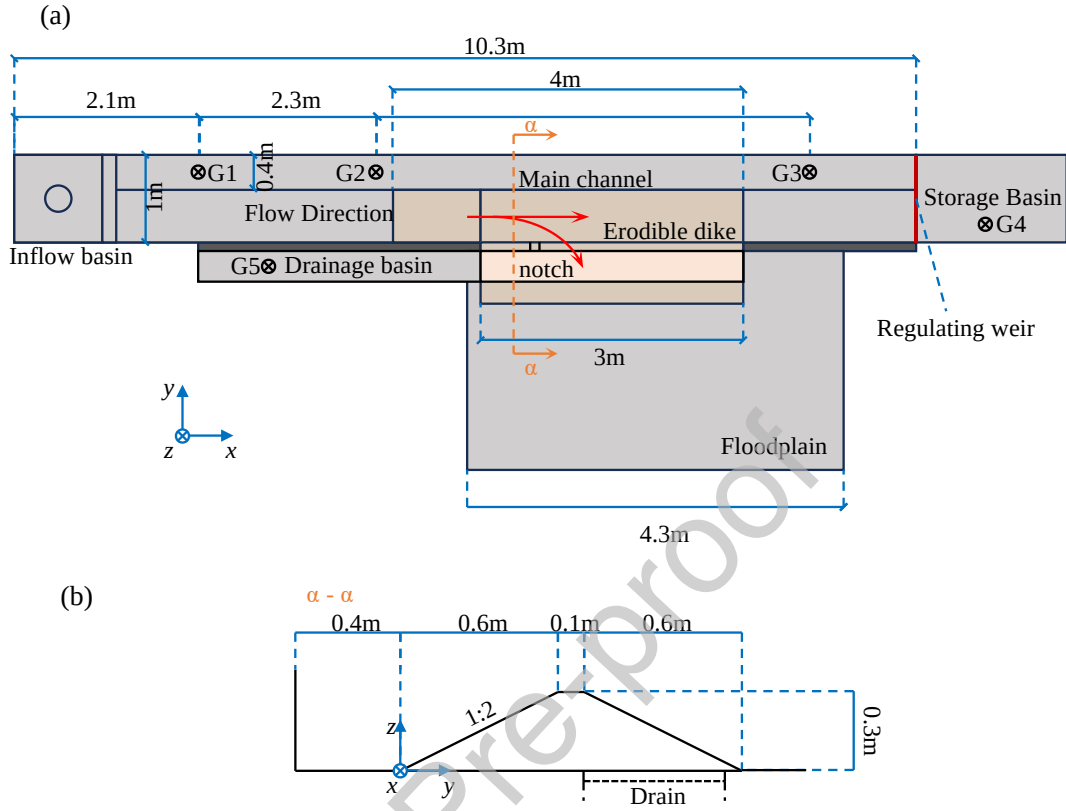


Figure 1. (a) Top view of experimental flume setup used by Rifai et al. (2019) and modelled in this study. (b) Initial dike cross section, before overtopping occurs.

From the experiments of Rifai et al. (2019), test 8 is modelled in the present work. In the physical experiment, a constant inflow discharge of $0.04 \text{ m}^3/\text{s}$ was imposed, with no backwater effect from the floodplain. Six representative instances of this evolving dike breach have been selected to model the flow field, namely the time instances $t = 19 \text{ s}$, 27 s , 36 s , 46 s , 80 s , 100 s (each case is marked with C8-1, C8-2, C8-3, C8-4, C8-5, C8-6, respectively). In each of these cases, the dike geometry recorded by Rifai et al. (2019) was used as a topographical boundary in a steady flow simulation, during which the dike was not allowed to change its geometry. Figure 2 shows the morphologies of the six selected cases.

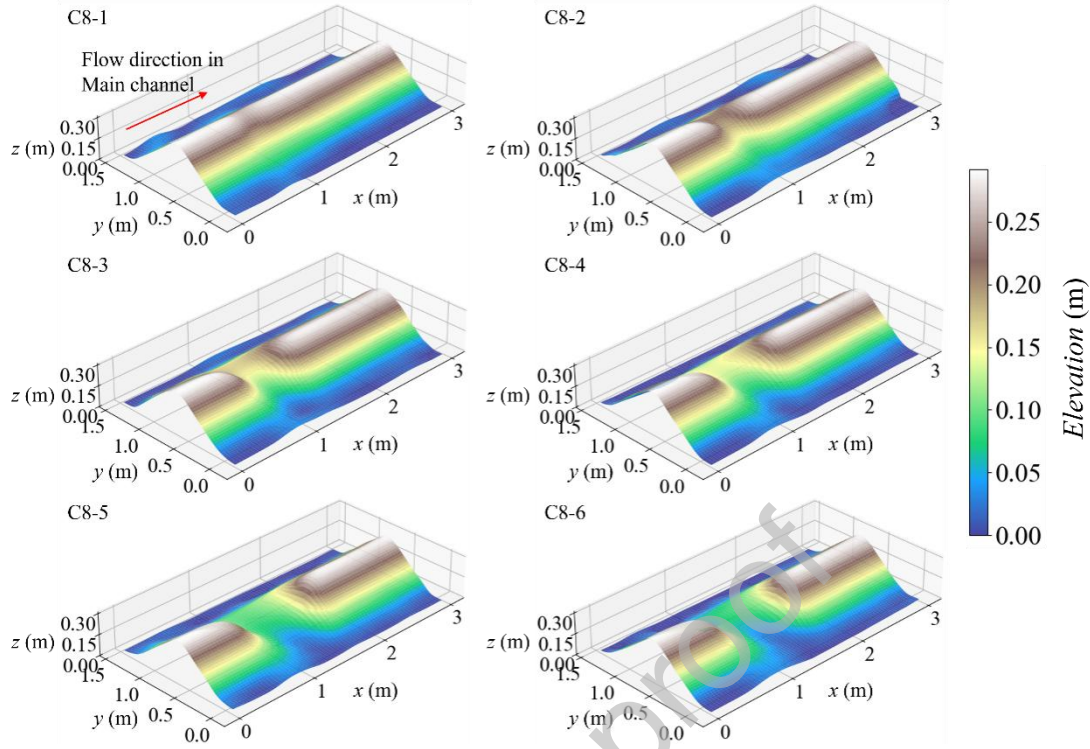


Figure 2. Dike morphology of selected time points of test 8 from the experiments of Rifai et al. (2019).

2.2 3D Numerical model

General description of 3D Numerical model settings

The 3D numerical model solves the incompressible Reynolds-Averaged Navier-Stokes (RANS) equations, together with a turbulent closure model based on a Boussinesq-type eddy viscosity assumption:

$$\nabla \cdot U = 0 \quad (1)$$

$$\frac{\partial U}{\partial t} + \nabla \cdot (UU) = -\nabla P + \nabla((\nu + \nu_t) \nabla U) \quad (2)$$

with U the 3D velocity vector (m/s), P the pressure (m^2/s^2), ν the fluid kinematic viscosity (m^2/s) and ν_t the eddy viscosity (m^2/s). The open-source software *OpenFOAM* (v9) is used with *interFoam* as solver, which is a two-phase solver able to capture the water surface. To estimate the eddy viscosity, the $k-\omega$ SST turbulence closure model was used. This turbulence model was chosen for its extensive usage and validation in engineering applications (Versteeg 2007), including in hydraulic engineering and free surface turbulent flow simulations. Compare with higher-fidelity approaches such as large-eddy simulation (Wang and Bowles 2006), this RANS

turbulence model provides a practical balance between computational cost and predictive capability (Yu et al. 2026), meanwhile, it also combines the advantages of the standard $k-\omega$ model near solid boundaries with the robustness of the $k-\varepsilon$ model in the free-stream region (Menter 1994).

The free surface was captured by the volume of fluid (VOF) method introduced by Hirt and Nichols (1981). Detailed information about this method can be found in Versteeg (2007). The VOF is computationally efficient and has been extensively used and validated. The interface between water and air is detected at the locations where the cells have a water content of 50%, while the rest of the cell is occupied by air. A Courant-Friedrichs-Lewy (CFL) restriction of 1 was imposed, leading to an adjustable time step of the order of 0.002 s. This choice helps ensuring the numerical stability, particularly in regions with high velocity gradients, and also provides a balance between adequately resolving steady-state condition and maintaining computational efficiency. The characteristics of the model are summarized in Table 1.

The near-wall flow behaviour was modelled with wall functions. The *nutkRoughWallFunction* was applied at all walls in the models. The C_s value of *nutkRoughWallFunction* is set to 0.5, meaning that the sand grain is assumed as uniform. The roughness height k_s is set to 1 mm, corresponding to the d_{50} value mentioned in Rifai et al. (2019).

Table 1. Numerical implementation of the 3D model in *OpenFOAM* (v9) and prescribed boundary conditions.

Implementations	Solver	interFoam
	Turbulence closure	RANS $k-\omega$ SST
	Wall treatment	nutkRoughWallFunction
	CFL number	< 1
	Multiphase	VOF
	Interface capturing scheme	MULES scheme
	Pressure-velocity coupling	PISO scheme
Boundary conditions	Inlet	Flow Rate Inlet Velocity
	Outlet/Atmosphere	Pressure Inlet Outlet Velocity

Boundary conditions

In each configuration, a constant inflow discharge is normally imposed at the inlet. Its value is equal to the total measured outflow discharge data of the physical experiment (i.e. the outflow discharge at the downstream end of the main channel, the breach discharge and the drainage discharge; drainage discharge was also considered as breach discharge even though it was only approximately 2% of the total discharge). The inflow discharge varies for the different tests, as listed in Table 2, to correspond to the simulated time instances of the experiment. It should be noted that in the physical experiment, the inflow discharge was constant during the process of dike failure, however, the sum outflow measured during the process (including outflow at dike breach and main channel) varied with time. Therefore, this approach utilizes a series of steady flow simulations to approximate transient conditions during the dike failure process, and the inflow discharge for each simulation was set the same with the corresponding measured transient sum outflow discharge in order to better approximate the transient features. The potential uncertainty of this approach to replicate the experiment is discussed in Discussion section.

The bed is horizontal, and its level is set to zero. A regulating weir was placed at the downstream end of the flume. Its height was adjusted such that the modelled downstream water level at location G3 (Figure 3) matched the water level data from the physical experiment at the considered time point (Figure 1). Given the uncertainties associated with the experimental measurements (i.e. the measurement accuracy of the ultrasonic sensor in the laboratory experiment is $\pm 1\%$ (Rifai et al. 2019), and the rapid variation in flow depth during the breaching process introduces potential additional uncertainty), a difference in the water level at G3 between the numerical model and the experimental data smaller than 2% is considered as acceptable here (Marcelo 2008). This is also consistent with , who reported that 2% tolerance aligns with the cumulative precision limits of hydraulic sensors and model setup under dynamic conditions.

Although the boundary conditions of each model are steady, the computed flow variables (i.e. flow depths, discharges, velocities, etc.) exhibit minor temporal fluctuations. This behaviour is expected as *interFoam* is an unsteady solver. To smooth out these fluctuations, the total simulation time for each case was set to 150 s, and the results reported in this study were time-averaged over the final 50 s of the simulation with an interval of 1s, with the first 100 s excluded to allow the flow to reach a quasi-steady state (Figure S1).

Table 2. Fixed inflow discharge and aiming downstream water level (G3) for the different modelled cases.

Cases	Boundary Conditions	
	Inflow discharge (m ³ /s)	Downstream water level (m)
C8-1	0.0405	0.281
C8-2	0.0461	0.279
C8-3	0.0555	0.270
C8-4	0.0603	0.252
C8-5	0.0498	0.203
C8-6	0.0475	0.187

Mesh strategy

A structured mesh was chosen to build the computational domain. In the current study, all cells are hexahedra with same dimensions in the x, y directions, and refined in the z direction to better capture 3D flow features and the air-water interface. To select a suitable mesh for simulating the intricate flow dynamics during dike failure, a mesh size analysis was carried out for model C8-1. Three cell sizes of 50×50×50 mm (in the x, y, z directions, respectively), 25×25×25 mm, and 20×20×10 mm with a locally refined region of 10×10×5 mm (which includes the whole dike area, as illustrated in Figure 3b) were tested.

The modelled flow depth at G1 with the coarser grid (50×50×50 mm) differs from the observation value by 13.8%. The 25 mm grid produced results comparable to those obtained with the 20×20×10 mm mesh (with a locally refined region of 10×10×5 mm) for flow depth. However, higher spatial resolution is required to accurately resolve detailed flow dynamics. Consequently, the 25 mm grid was not adopted because its vertical resolution was considered

insufficient. The grid convergence index (GCI, a quantitative measure used in CFD to determine whether the solution has reached grid independence., proposed by Roache (1998)) calculated with the water depth at G1 for the fine mesh of $20 \times 20 \times 10$ is 0.861% which is could be regard as sufficient according to Coleman and Members (2009), indicating that the fine grid provides a grid-independent solution. Detailed results of the mesh sensitivity analysis are provided in the Supplementary Material (Figure S2 and Table S1). Based on this analysis, the mesh size of $20 \times 20 \times 10$ mm with a locally refined zone of $10 \times 10 \times 5$ mm was selected for all modelled cases.

The geometry of the dike was constructed by firstly extracting the digital elevation model data from the experimental dataset (Figure 2). The function “*snappyHexMesh*” was applied to construct the dike geometry, and for a mesh refinement near the breach (Figure 3a and b).

The simulations were run in parallel using the public domain OpenMPI implementation of the standard MPI on Guangxi University's high-performance computing platform, which is powered by the Sugon 7000A high-performance super-parallel computer system. The aggregate cell number for each case is around 7.5 million, and the computational domain was divided into 40 subdomains. Approximately 85 h were needed to simulate 150 s of flow for each case.

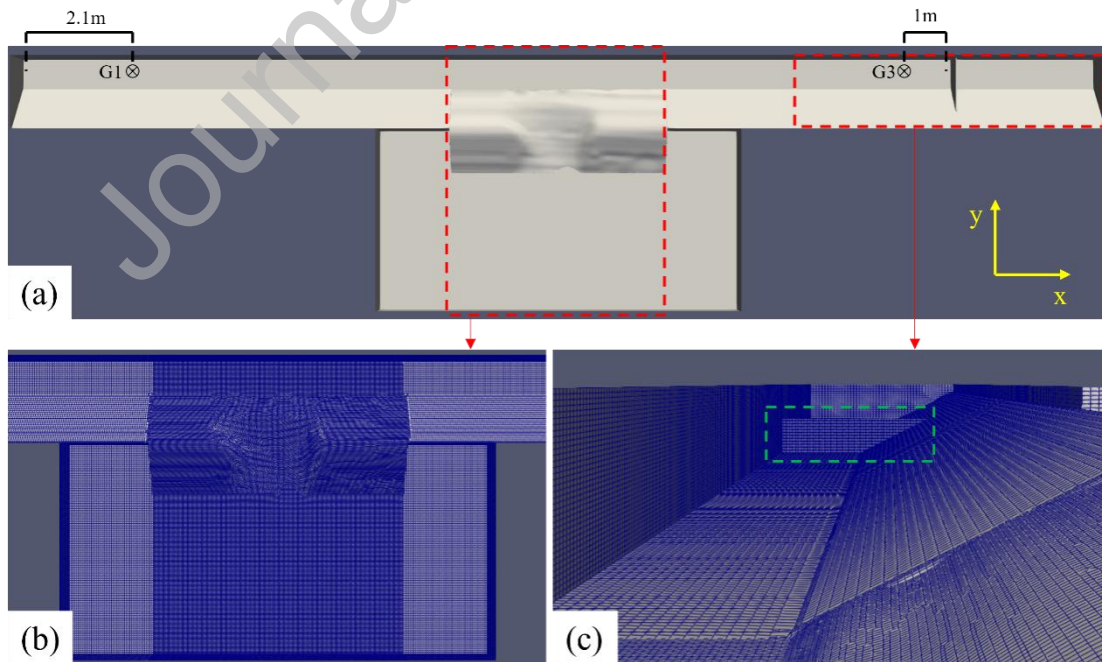


Figure 3. Model configuration of case C8-5. The mesh consists of hexahedra cells with size of $20 \times 20 \times 10$ mm (with a more refined area of $10 \times 10 \times 5$ mm including the whole dike area). (a)

Top view of the study area illustrating the position of two key probes monitoring the water level in the corresponding experimental setup, (b) zoomed top view of the refined mesh area, and (c) view of the outlet with the regulating weir.

3 Results

Section 3.1 compares the computed flow depths and the discharge distribution at the dike breach against the physical experiment measurements. In Sections 3.2 and 3.3, the modelled horizontal and vertical flow velocity patterns, respectively, are presented and analysed. Section 3.4 focuses on the turbulence characteristics.

3.1 Flow depth and discharge distribution

Figure 4 shows a comparison between the modelled flow depths and the corresponding measurements at location G1. The standard deviation of the numerical results over the 50 s monitoring period is on the order of 10^{-4} – 10^{-5} m, which is negligible compared to the corresponding mean values. Overall, the modelled flow depths at G1 differ from the experimental measurements by 6% on average (0.012 m to 0.018 m).

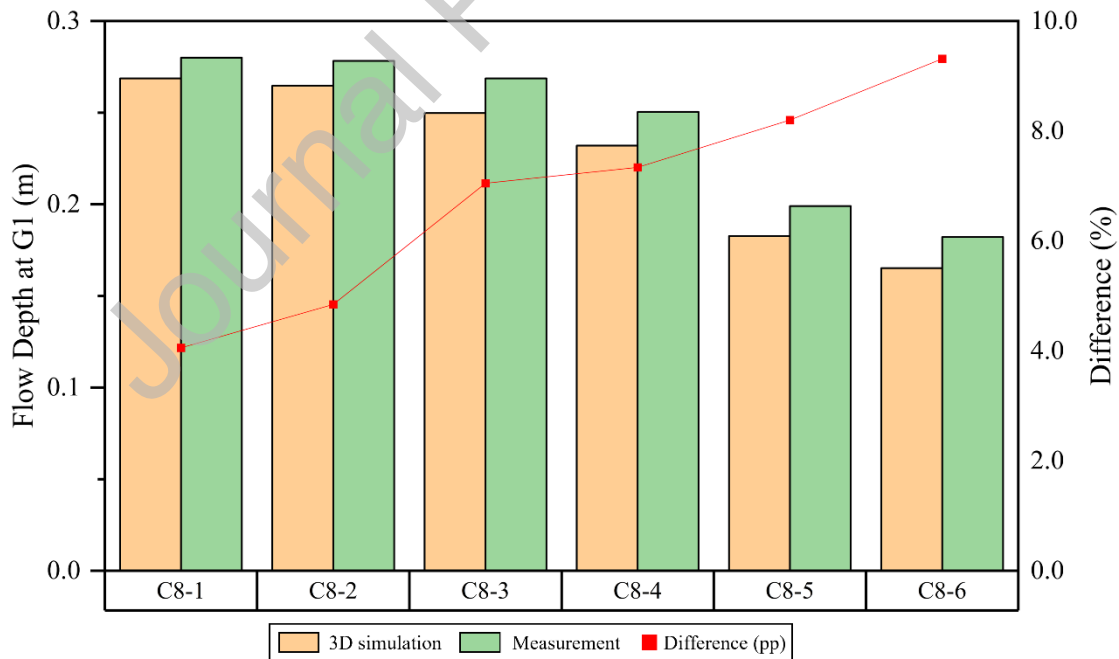


Figure 4. Comparison between modelled and measured flow depths at G1 (indicated in Figure 3) for the tested cases. The Difference is an absolute error normalized by the lab measurement results.

Figure 5 compares the simulated and measured outflow discharge distributions at the breach and the main channel for all cases. In the physical experiments, the fraction of breach outflow increases with the expansion of the breach. The numerical model successfully captured this trend. In addition, the calculated standard deviation of outflow discharge at both the breach and the main channel is very low and on the order of 10^{-4} – 10^{-5} m³/s, which shows that the model provides a steady solution for the discharge partitioning. The relative differences between the numerical simulations and the experimental measurements range from 6.6% to 15.2%, with larger differences at the last stages of the breach expansion.

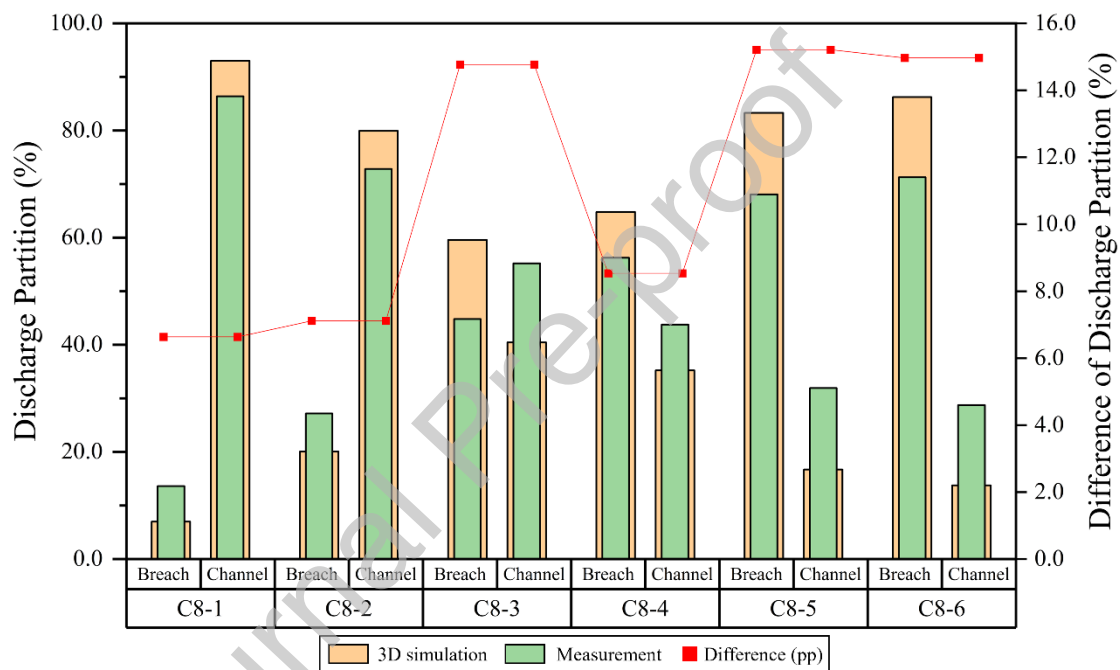


Figure 5. Comparison between modelled and measured flow discharge partitioning into the Breach and the main Channel downstream outlet (Figure 3). The Difference is an absolute error normalized by the lab measurement results.

3.2 Horizontal flow features

Main channel

Figure 6 illustrates the horizontal flow velocity patterns at the surface of the main channel and at the location $z = 0.05$ m for all tested cases. First, the focus is set on the surface flow patterns. In cases C8-1 and C8-2, the surface velocity along the channel is rather constant. The enlargement of the breach (starting from C8-3) leads to a significant velocity reduction

downstream of the breach. In all cases, the maximum surface velocity in the main channel is located near the breach.

Recirculation zones are formed downstream of the dike breach in cases C8-3 to C8-6. Their shape and number depend on the breach expansion, which is particularly visible in cases C8-5 and C8-6. In C8-3 and C8-4, a single centre of recirculation is observed at the opposite side from the breach, while in C8-5 and C8-6, two centres of recirculation are detected, one at each side. In addition, the observed recirculation zones of all simulated cases have a superimposed location to the corresponding low-velocity zones. Low-velocity zones in C8-3 and C8-4 appear near the side opposite of the breach. In C8-5 and C8-6, these zones are larger than those in C8-3 and C8-4 and are accompanied by lateral shifting and contraction of the high-velocity region downstream.

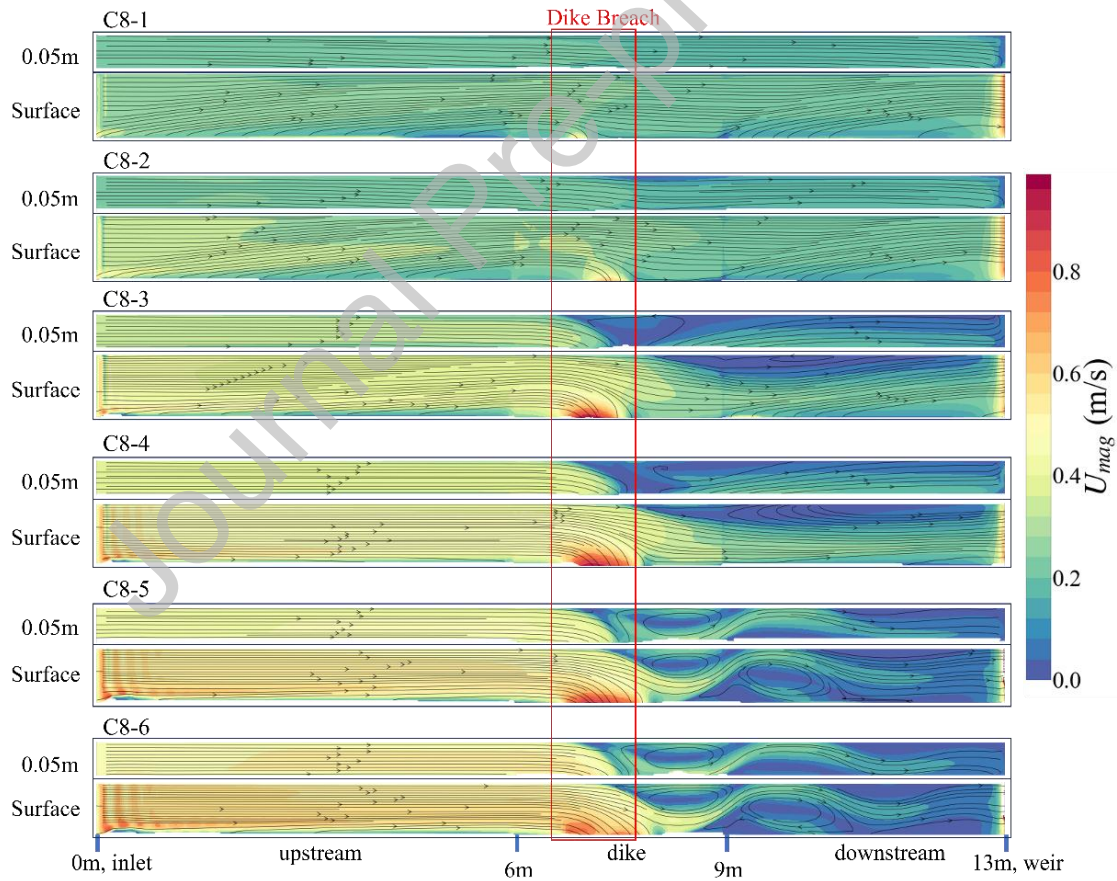


Figure 6. Horizontal flow velocity patterns in the main channel at the free surface and at $z = 0.05$ m. Colours and streamlines refer to the norm and the direction of the sum of $\vec{U}_x$ and $\vec{U}_y$, respectively.

Figure 7 presents spatial distribution of the vertical vorticity along the z direction ω_z across multiple horizontal cross-sections of the main channel under six different flow conditions (C8-1 to C8-6). In the recirculation zone downstream of the breach, the vorticity component ω_z exhibits elevated values that are spatially consistent with the secondary flow region, this correspondence indicates that rotational motion contributes to the formation and maintenance of the recirculation zone. Notably, no recirculation zones are observed in C8-1 and C8-2, suggesting that a minimum level of discharge diversion toward the dike breach is required for their formation.

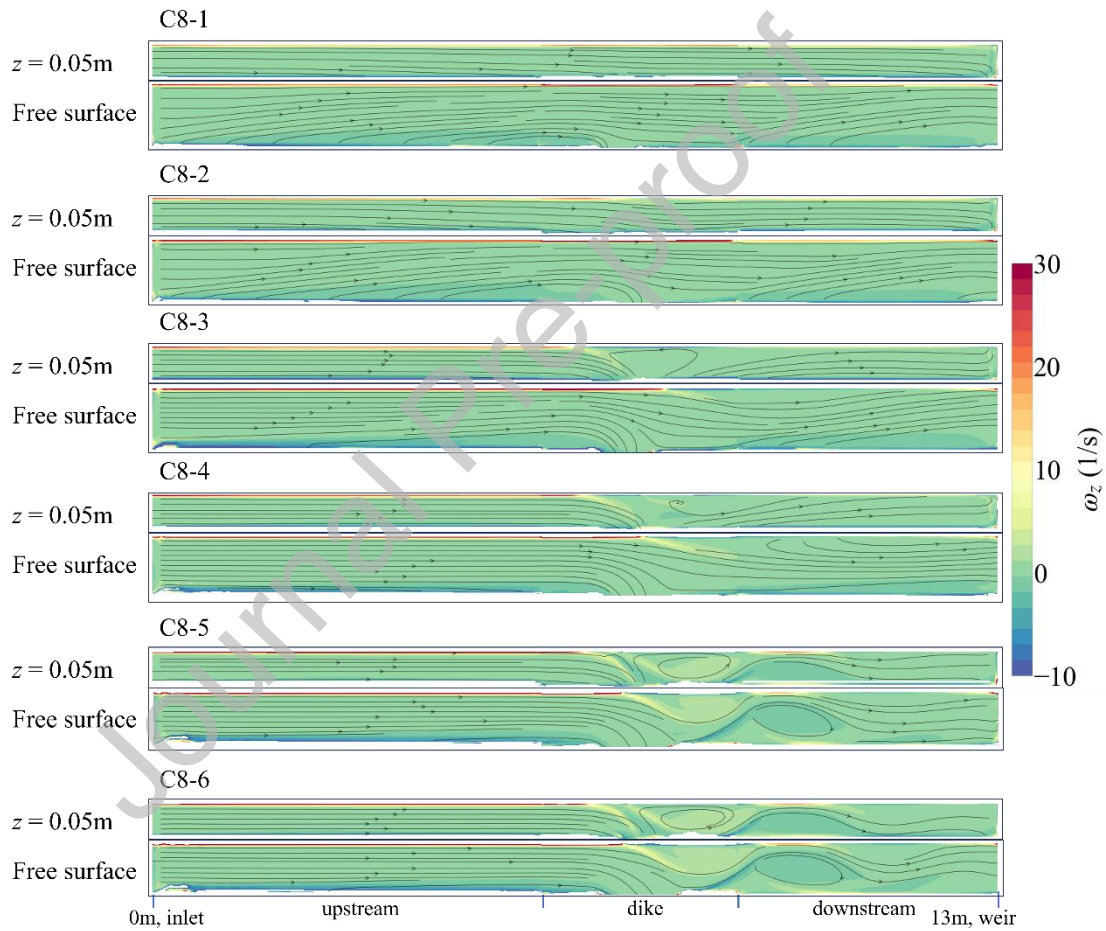


Figure 7. Vertical vorticity of z direction at the free surface and at $z = 0.05$ m of all simulated cases. Colours refer to the vorticity and streamlines refer to the norm and the direction of the sum of $\vec{U}_x$ and $\vec{U}_y$.

The flow patterns at $z = 0.05$ m show similarities to the surface velocity field in terms of the overall downstream variation of flow velocity and the presence of complex recirculation zones downstream of the breach. However, significant differences are observed between the surface

and subsurface flows. First, surface velocities upstream are systematically higher than those at $z = 0.05$ m, and the flow acceleration near the breach is not observed at this subsurface level. Taking the case C8-3 as example, Figure 8 demonstrates the vertical profile of the U_{mag} at several location for case C8-3. Specifically, the U_{mag} near surface could be 50% to 70% higher than that near the bottom. Besides, both the location and number of recirculation zones differ between $z = 0.05$ m and the free surface. These features shed light on the three-dimensional nature of the flow structure from the bed to the free surface. Consequently, Section 3.3 further examines the characteristics of secondary flow zones downstream of the dike breach by analysing the vertical flow patterns.

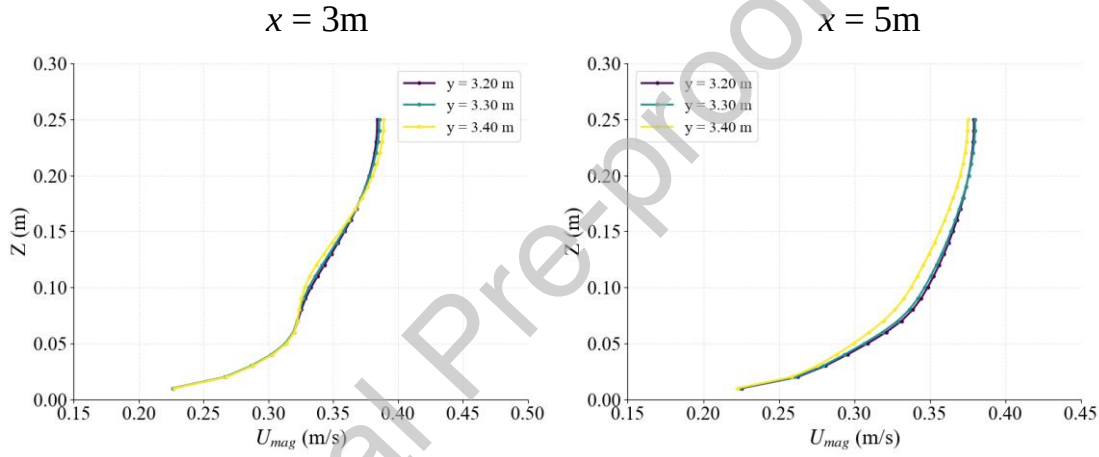


Figure 8. Vertical velocity profiles of $\vec{U}_x + \vec{U}_y$ of velocity at $x = 3\text{m}$ and 5m while $y = 3.2\text{m}$, 3.3m , 3.4m in the main channel of C8-3.

Dike breach

Figure 9 illustrates the surface flow patterns through the breach of the erodible dike. In C8-1 and C8-2, the breach is extremely narrow. Consequently, the flows are predominantly controlled by the breach geometry, which strictly confines the direction of water release into the floodplain, thus the cases C8-1 and C8-2 are not illustrated here, but can be found in the Supplementary Material (Figure S11). Figure 9 shows that for the other cases (C8-3 to C8-6), the breach outflow is rather symmetrical at the earlier stages of the dike failure process (C8-3 and C8-4), whilst the outflow direction deviates to the downstream side when the breach grows. For each case, the flow velocity is maximum immediately after exiting the dike breach. The maximum surface velocity decreases when the breach grows, as expected.

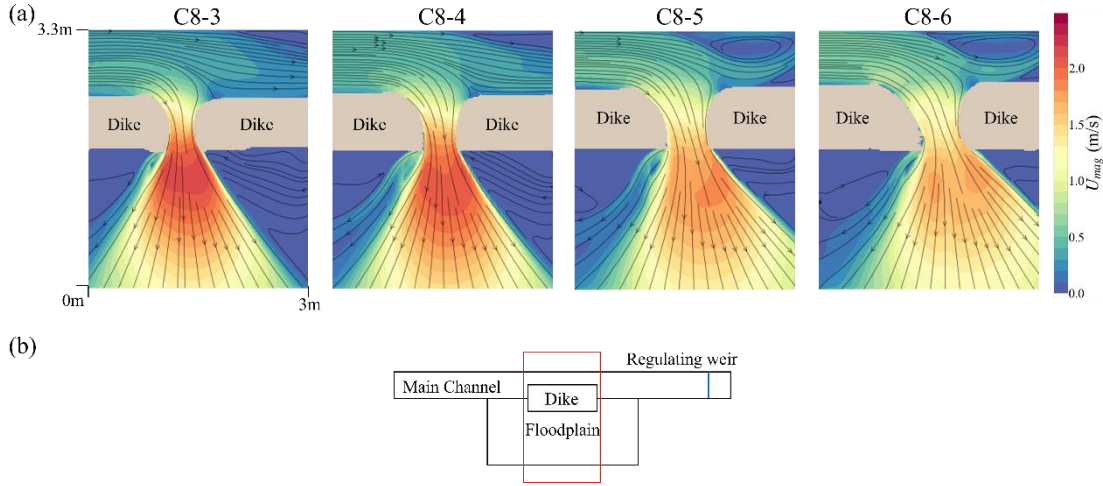


Figure 9. (a) Surface flow velocity patterns through the dike breach. Colours and streamlines refer to the norm and the direction of the sum of $\vec{U}_x$ and $\vec{U}_y$, respectively. (b) Sketch of the presented area.

3.3 Vertical flow features

Main channel

Figure 10 illustrates the flow velocities at selected cross-sections from upstream to downstream for the case C8-3. The other cases exhibit similar flow patterns away from the breach and are therefore presented only in the Supplementary Material (Figure S3 to Figure S7).

First, the streamwise component of the velocity, U_x , is nearly uniform upstream of the breach. As the flow approaches the breach, pronounced velocity stratification develops in both the horizontal and vertical directions, reaching a maximum at $x = 7$ m. Zones of negative velocity appear across the channel from the dike breach near the bed between $x = 6.6$ m and $x = 8$ m, indicating near-bed flow recirculation and resulting in complex flow dynamics around the breach. Downstream of the breach, the flow gradually mixes, and U_x returns to a uniform distribution.

Besides, the streamlines consisting of $\vec{U}_y$ and $\vec{U}_z$ also bring insight into the flow structure. Upstream of the dike breach, the flow exhibits a counterclockwise rotation. Along the main flow direction x , this rotation gradually weakens, and the water is directed toward the breach until $x = 7.5$ m. Downstream, as the flow mixes, a secondary rotation forms again but in the clockwise direction. These observations indicate that the enlargement of the breach not only

impacts secondary vertical flow patterns downstream of the breach but also affects vertical flow dynamics upstream.

Figure 11 presents a systematic comparison of the flow velocity patterns at two cross-sections located upstream ($x = 5$ m) and downstream ($x = 11$ m) of the breach for all six cases. Figure 11 clearly illustrates that upstream of the breach at $x = 5$ m, U_x increases as the breach expands due to the decreasing flow depth, which is consistent with the horizontal flow fields presented in Figure 6. For all cases, the velocity profile across the channel section exhibits a typical pattern influenced by wall friction: U_x reaches its maximum near the mid-channel and decreases towards both side walls, with the peak shifted towards the breach side. A figure illustrating the boundary layers more clearly is provided in the Supplementary Material (Figure S8). Downstream of the dike breach in the main channel, at $x = 11$ m, the U_x pattern is significantly impacted by the reduction in the water level and the reduction of the flow discharge due to the increase of the breach discharge, as well as by the secondary flow. Specifically, the maximum velocity first increases with the expansion of the breach but then decreases (i.e. the maximum value of U_x is larger in C8-3 and C8-4 than in C8-1 and C8-2, whilst it is lower in C8-5 and C8-6 than in the former four cases). Besides, the downstream distribution of U_x in configurations C8-1 and C8-2 is relatively even, showing little lateral variation from the left to the right side., whilst a significant gradient is presented from the left side to the right side (i.e. breach side) in the other cases.

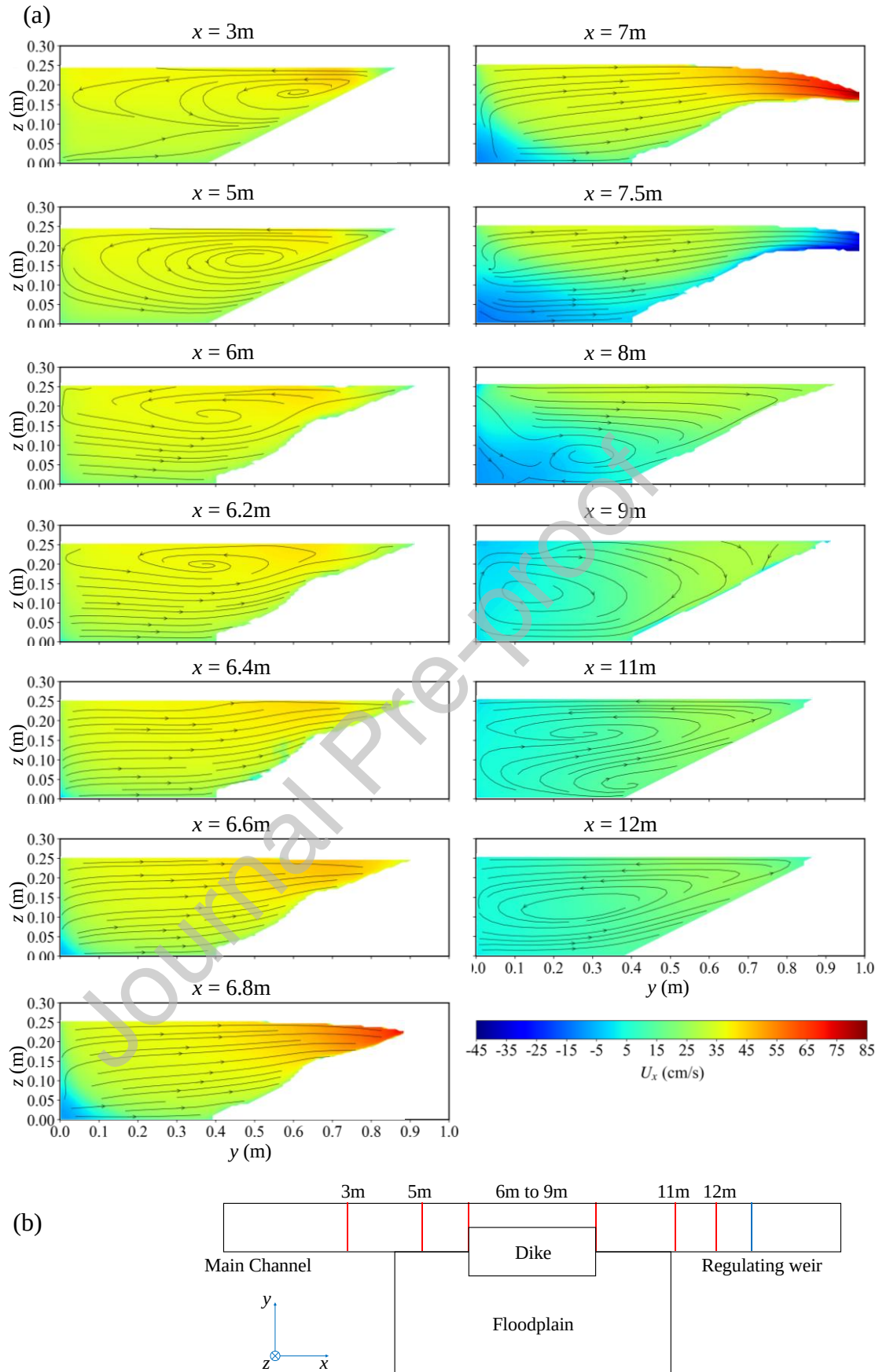


Figure 10. (a) Flow velocities at cross-sections located at $x = 3$ m, 5 m, 6 m, 6.2 m, 6.4 m, 6.6 m, 6.8 m, 7 m, 7.5 m, 8 m, 9 m, 11 m, and 12 m along the main channel for case C8-3. Colours

refer to the streamwise component of the velocity, U_x , and streamlines represent the direction of the sum of $\vec{U}_y$ and $\vec{U}_z$. (b) Sketch of the selected cross-sections.

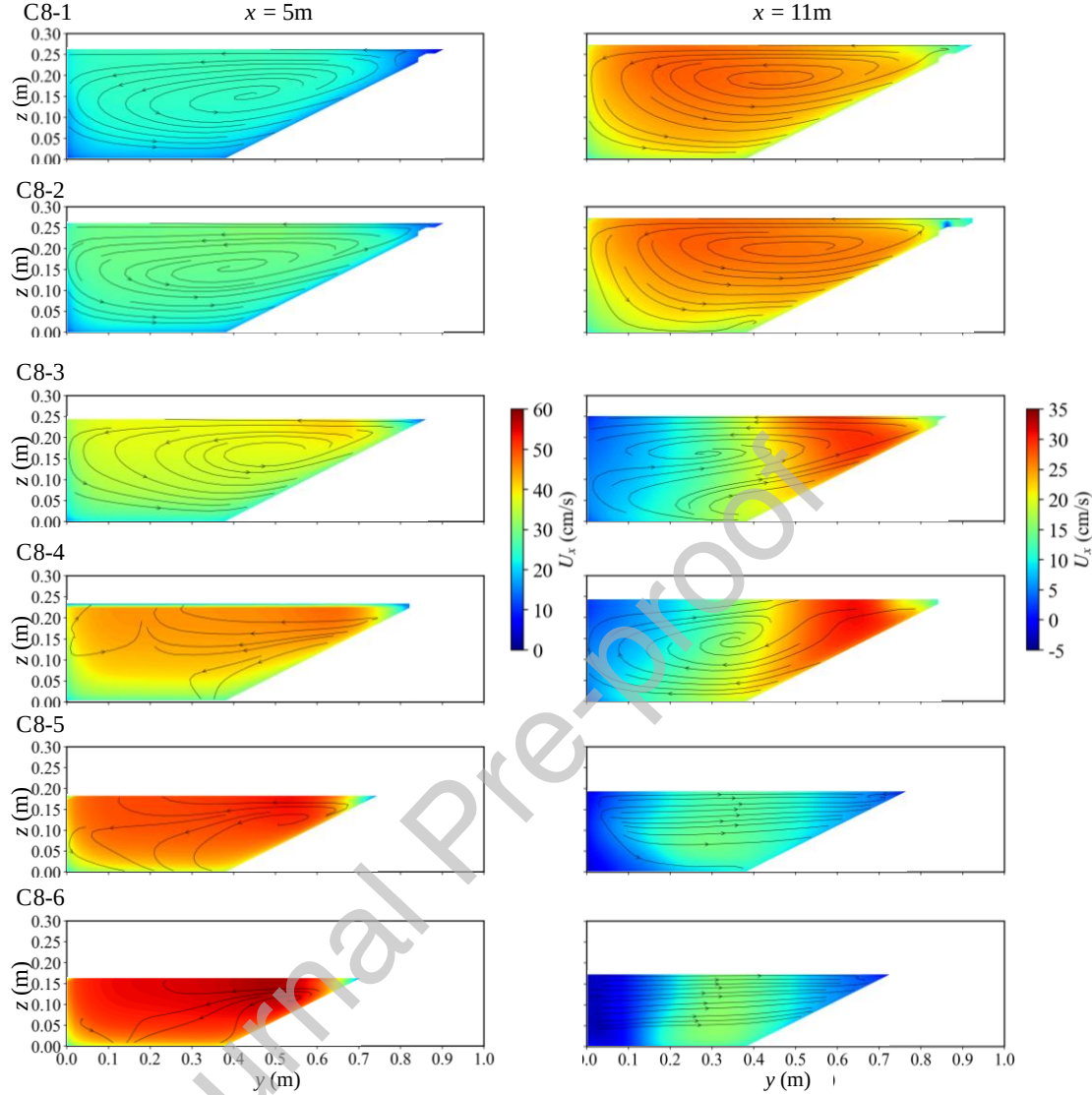


Figure 11. Comparison of flow velocities at cross-sections upstream ($x = 5$ m) and downstream ($x = 11$ m) of the dike breach for all tested cases. Colours refer to the norm of the streamwise component of the velocity, U_x , and streamlines represent the direction of the sum of $\vec{U}_y$ and $\vec{U}_z$.

Figure 10 shows the helicity in the y - z cross-section along the main channel for the case C8-3 of which distributions indicate the presence of three-dimensional rotational structures within the flow. For cross-sections at upstream and downstream main channel (i.e. $x = 3$ m, 5 m, 6 m, 6.2 m, 9 m, 11 m), at the left side of the section across the main channel from the dike breach, helicity remains almost zero (i.e. grey colour), implying weak three-dimensional rotation movement. Toward the right side (i.e., the breaching side), localized regions of higher negative

helicity develop near the surface and the slope bottom, corresponding to the formation of streamwise rotationality in counterclockwise direction. This may result from the interaction between the primary flow and the secondary circulation induced by the sloping boundary and the breaching. Specially, the helicity keeps increasing for the sections located in the breaching zone ($x = 6.4\text{m}, 6.6\text{m}, 6.8\text{m}, 7\text{m}, 7.5\text{m}$), suggesting a counterclockwise rotation, while secondary flow is not observed by the streamlines, this can be related to the relatively high velocity U_y toward the breaching. Then, moving downstream to $x = 8\text{m}$, regions of positive helicity show up, indicating the formation of right-handed spiral flow as rotational structures begin to align with the main flow. This progression from negative to positive helicity reflects the evolution of rotational flow induced by the dike breaching, highlighting how localized recirculation near the breach transforms into downstream-propagating spiral vortices.

The spatial variation of helicity shows the coupling between velocity and vorticity, with stronger values occurring in regions of enhanced shear rather than at vortex centres. This confirms that the flow exhibits pronounced secondary circulation and helical motion, deviating from a purely two-dimensional structure.

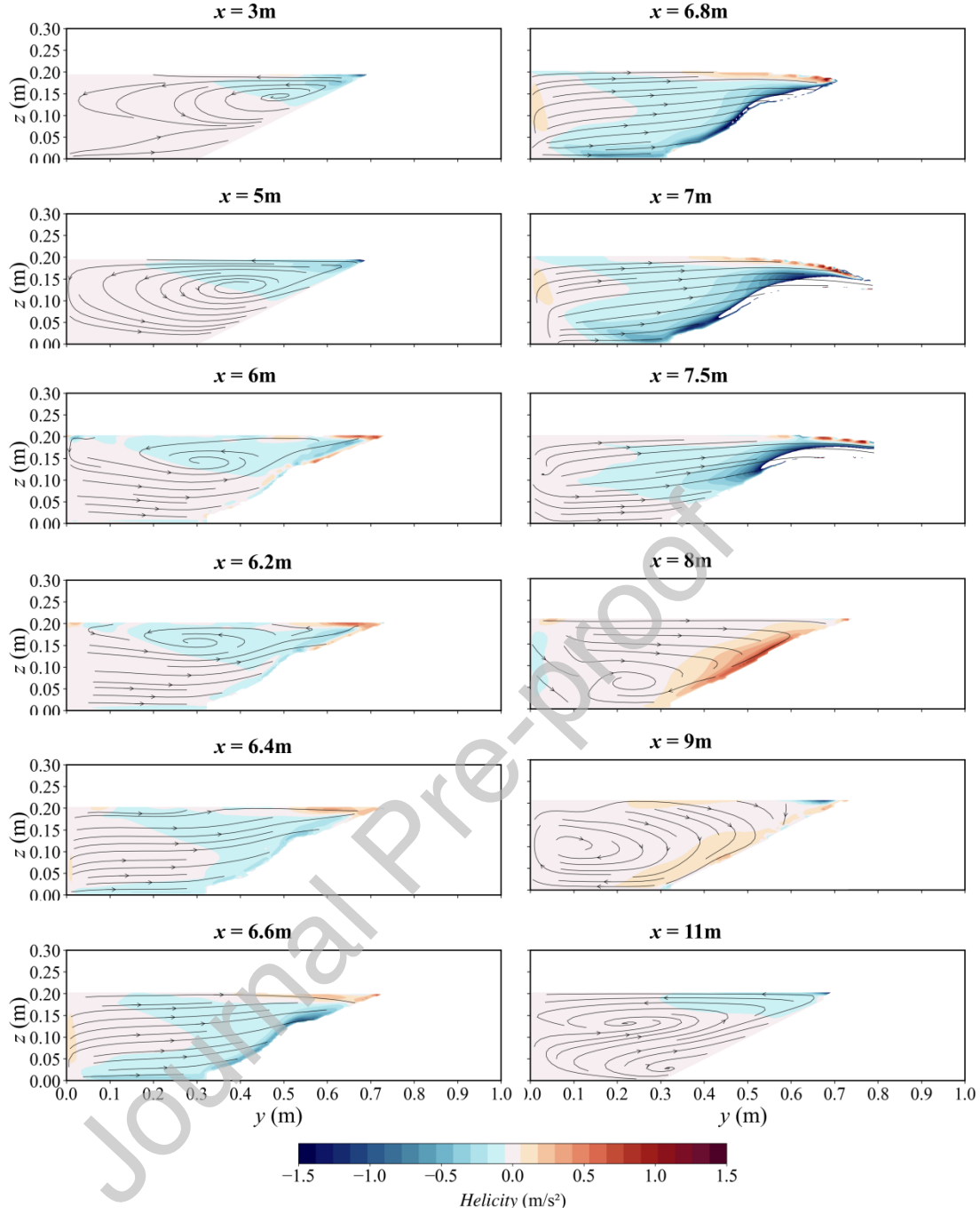


Figure 12. Helicity and streamlines at cross-sections alongside main channel of case C8-3. Colours refer to the helicity, and streamlines represent the direction of the sum of $\vec{U}_y$ and $\vec{U}_z$.

Dike breach

Figure 13a illustrates the distribution of flow velocity U_y at the cross sections located at the position of the levee crest and the corresponding velocity field for all simulated cases. At earlier stages when the dike breach is relatively small, obvious slope of the flow surface is observed at the breach section. The free surface elevation at the downstream side is significantly higher

than at the upstream side, especially in case C8-3. When the breach expands, this phenomenon fades and the free surface elevation tends to be horizontal. Besides, it could be observed that $\overline{U_z}$ is not negligible at all the presented sections, particularly at the downstream side of the sections, with secondary flow forming for the case C8-6. The water flowing through the breach comes also from the downstream side of the breach at the early stages (i.e. C8-1 - C8-4), highlighted by negative values of $\overline{U_x}$ as demonstrated in Figure 10 (i.e. $x = 7.5\text{m}$).

Furthermore, the distribution of U_y exhibits an irregular, non-uniform distribution across the breach cross-section in all cases. More specifically, cases C8-1 to C8-4 share a characteristic where the maximum U_y is concentrated in a distinct high-velocity core area. From this core, velocity decays towards both the free surface and the side walls. In contrast, case C8-5 shows a significantly more widespread region of high U_y , indicating a more uniform distribution, while Case C8-6 presents a unique pattern with multiple, discrete high-velocity cores. These detailed observations are contradictive to the common assumption of uniform velocity across the dike breach which is typically adopted by simplified models. 3D flow simulations in realistic dike breaches can provide critical insights for improving discharge predictions in dike breach models, and contribute to the development of new parameterisations of transverse velocity variability in simplified breach models, which would be essential for practical usage in large-scale simulations.

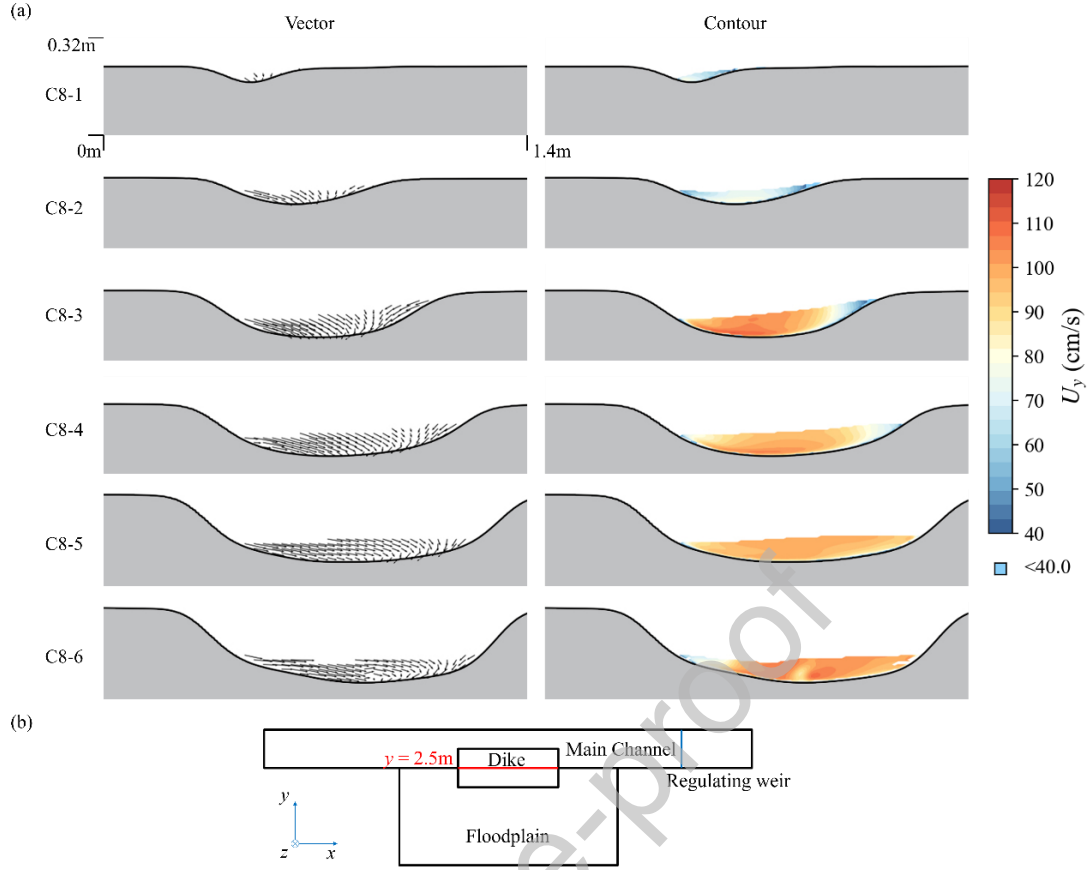


Figure 13. (a) Flow patterns across the breach in each configuration. Colours refer to the norm of the velocity component along y-axis, U_y , and streamlines represent the direction of the sum of $\vec{U}_x$ and $\vec{U}_z$. The gray shaded areas in Figure 10a show the dike longitudinal cross section at this location. (b) Sketch of the location of the considered vertical cross-sections at the breach.

3.4 Turbulence characteristics

Figure 14 illustrates the turbulent kinetic energy (i.e. k from the *RANS* solver) at various horizontal cross-sections for case C8-1 to C8-6. Horizontal layers are selected every 0.05 m from the bottom of the main channel. In general, regions of elevated *TKE* (depicted in yellow and red) correspond to zones of strong flow disturbance, such as recirculation and vortex formation, while low *TKE* areas (blue) indicate relatively stable flow. First, by visualizing the *TKE* distribution at horizontal layers, this term is rather small for the cases C8-1 and C8-2, then both the magnitude and the spatial extent of high *TKE* regions increase progressively from C8-1 to C8-6, reflecting the intensification of turbulence as the dike failure progresses. Notably, areas with higher *TKE* for the other cases are noted near the breach or downstream of it for the cases C8-3 and C8-4, and these zones are rather on the opposite side of the breach. For the C8-

5 and C8-6, high *TKE* zones are predominantly located near the centre of the channel and around the breach (i.e. flow constrictions). The distribution of high *TKE* area is consistent with the horizontal velocity gradients shown in Figure 6. These results reveal that the dike breach expansion significantly amplifies flow turbulence intensity and energy dissipation in the main channel.

Figure 15 presents the *TKE* at selected vertical cross-sections in the main channel for case C8-3. Figure 15 shows that the location of relatively large *TKE* overlaps with the area where high-velocity core flow and slower-moving regions interact (i.e. see in Figure 10, $x = 6.8$ m, 7 m, 7.5 m, 8 m, also highlighted Figure S10 in the in the Supplementary Material). This interaction creates shear layers and intense mixing with large production of turbulence.

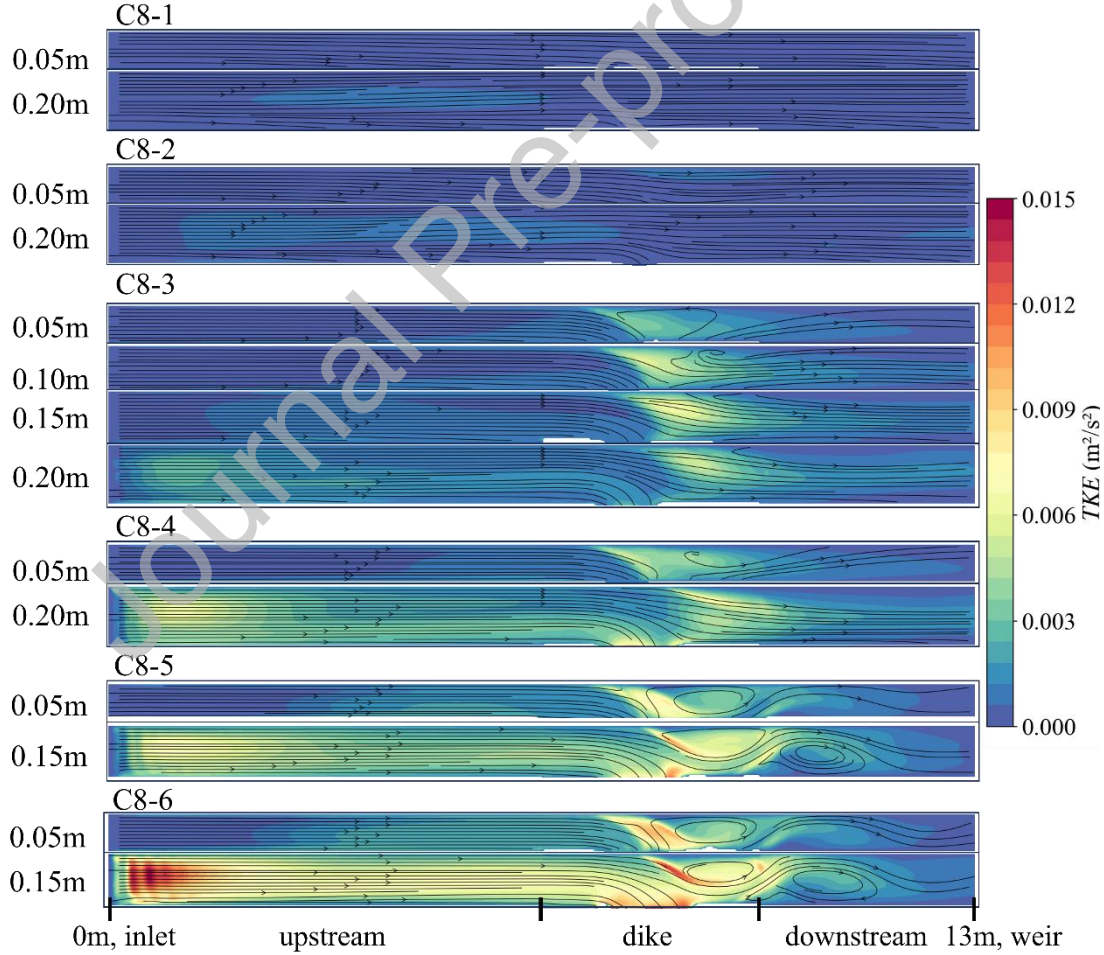


Figure 14 *TKE* at selected horizontal cross-sections with reference to the bed ($z = 0$ m) for all cases (C8-1 to C8-6). Streamlines refer to the direction of the sum of $\vec{U}_x$ and $\vec{U}_y$.

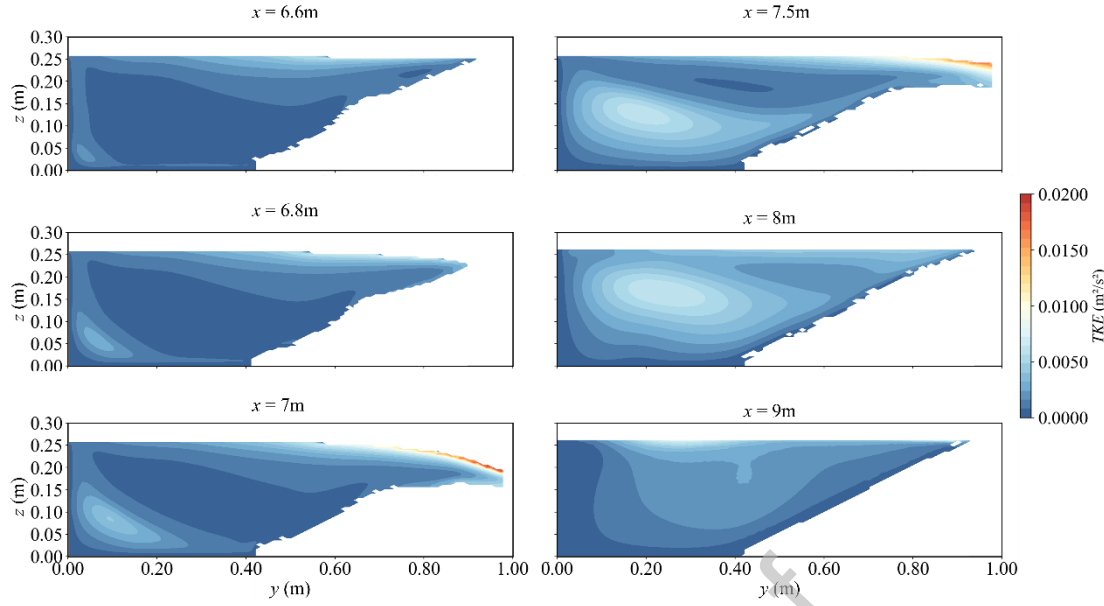


Figure 15. Time-averaged TKE at selected cross-sections in the main channel near the breach of case C8-3.

4 Discussion

4.1 Influence of breach shape on flow patterns

Current models use simplified shapes (including rectangular and trapezoid) to reproduce the breach during a dike failure (Wang and Bowles 2006, Wu and Li 2017). We are particularly interested in the similarity and difference of breach flow between a realistic breach topography and a simplified shape. Michelazzo et al. (2015) conducted a laboratory flume experiment with a zero-height rectangular side weir, then conducted detailed measurements of key parameters including free surface flow depth, discharge and velocity fields at various locations to gain information of the three-dimensional flow pattern. Firstly, similar to their research, the current study has also shown reverse flow in the main channel downstream of the breach area. Besides, a long flow separation zone is also observed at the opposite side of the breach at the downstream of main channel, similar phenomenon was also revealed by (de Ruijscher et al. 2020) and Ibrahim et al., (2022) , this is clearly reproduced in the stages C8-3 and C8-4, but with the evolution of breach geometry toward C8-5 and C8-6, this flow recirculation is relatively small and downstream flow pattern in much more complex with the presence of jet flow (Figure 6). Next, Michelazzo et al. (2015) showed that the pattern of streamlines towards the breach varies

across different water layers, with the flow at the lower water layers being affected further away from the upstream side of the weir compared to the flow at the upper water layers. On the contrary, the findings of the current study with an asymmetric breach that is shaped by the flow dynamics indicate that the streamlines begin to curve at similar locations upstream of the breach at the different stages of the dike breach. Moreover, both the experiments of Michelazzo et al. (2015) and the numerical simulations in this study revealed a helicoidal flow pattern near the breach, but the rotation centres were at different locations. Michelazzo et al. (2015) observed the rotation centre below mid-depth, while the simulated rotation centres in this study approach the flow surface (i.e. see Section 4.2). This difference may be attributed to the combined influence of the bottom geometry of the breach and the slope of the dike, which highlights the need for exploring the flow dynamics with realistic dike geometries where the side opening does not have an idealized rectangular shape.

4.2 *Correlation between wall shear stress and breach expansion*

The breach expansion is closely related to the wall shear stresses during the failure process. Therefore, a better understanding of wall shear stress distribution is essential for better characterizing the lateral breach expansion process. C8-1 is at the very beginning of the breaching process and the breach is extremely small, thus, it is not considered here.

Figure 16 (a) illustrates the distribution of the shear stress exerted on the dike breach surface, and Figure 16 (b) demonstrates the shear stress at a certain terrain profile to facilitate comparison and analysis. In every simulated case, the wall shear stress increases with increasing flow velocity from the dike crest to the floodplain. As the breach expands, both the maximum wall shear stress and the area with intense shear stress decrease, which explains the reduced rate of breach expansion. Additionally, the distribution area of elevated wall shear stress at the downstream side is generally larger than the area at the upstream side (Figure 16b). To be more specific, at the upstream side, larger wall shear stress appears on the relative flat “bottom” of the breach, while at the downstream side, the larger wall shear stress appears on the slope, causing erosion and breach expansion. This observation aligns with the results of the breach

expansion process presented in Rifai et al. (2017) that the initial hourglass-shaped breach eventually lost its symmetry with the expansion of the breach. This phenomenon reasonably explains and validates the non-symmetrical expansion of the breach in failures in fluvial dikes compared to failures in earthen dams.

According to the empirical formulation of van Rijn (1984) and the sand characteristics detailed in Section 2.1, the critical Shields number is evaluated as approximately 0.033, giving a critical bed shear stress of $\tau_{cr} \approx 0.485$ Pa. Meanwhile, the area with relatively high wall shear stress from the simulation results presents values (up to 7.3 Pa) that significantly exceed the critical threshold, this exceedance confirms that the numerical model accurately captures the initiation and intensification of the scouring process.

Due to the inherent challenge in measuring transient data during dike failure process, critical data for validation on the wall shear stress remain scarce. Studies on curved channel bends have demonstrated that RANS models tend to overestimate bed shear stress by approximately 20% on average, especially near the inner bank at the bend apex and in the central region of the channel (Blanckaert and De Vriend 2004, Song et al. 2022, Stoesser et al. 2010). Given the importance of wall shear stress in controlling breach erosion and expansion, advances in experimental measurement techniques and more comprehensive data collection are essential for improving the robustness of wall shear stress predictions.

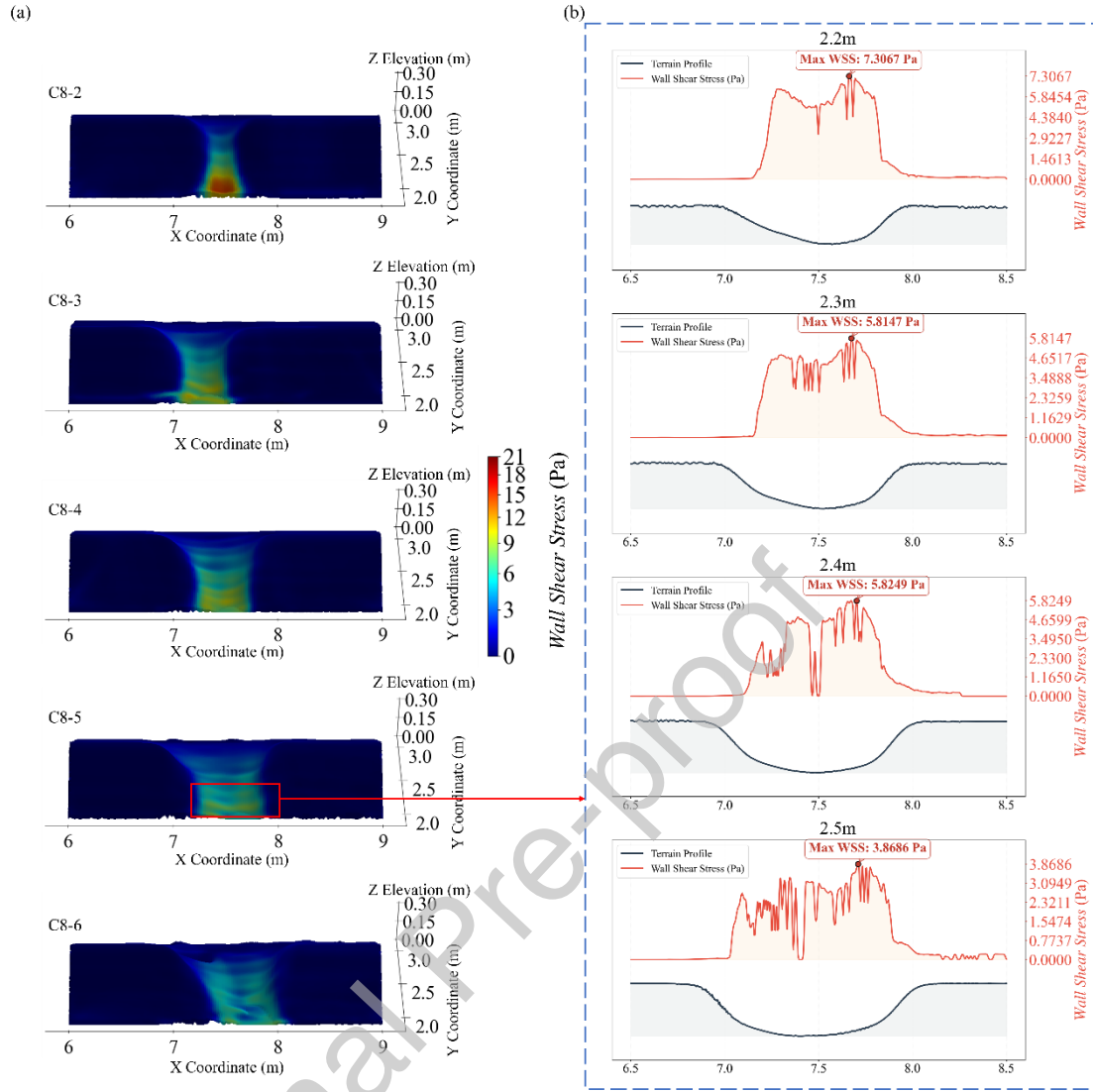


Figure 16. (a) Distribution of the wall shear stress exerted on the dike breach surface. The view direction is from the floodplain to the main channel. (b) Distribution of the wall shear stress along the x direction at $y = 2.2\text{m}$, 2.3m , 2.4m , 2.5m for the case C8-5 (highlighted with red rectangular; note that the rectangular is a sketch and does not represent precise axis).

4.3 Main limitations and research needs

The findings of this study provide valuable insights into fluvial dike failure processes using high-resolution numerical simulations to approximate transient conditions and analyse the associated fluid dynamics, which are challenging to capture through prototype observations or physical experiments with existing measurement techniques.

Steady-flow simulations were adopted in the present study to approximate the instantaneous flow conditions during dike failure processes. However, this approach cannot fully capture

actual transient flow features, and further investigation towards it would be necessary. Meanwhile, The methodology employed in the current study builds upon a state-of-the-art approach previously applied in an experimental study on earthen dam breach by Alvarez (2024), in which fixed breach geometries were constructed to represent different instances in time of a collapsing earthen dam and particle image velocimetry (PIV) was employed to capture the flow field at the free surface. The present study not only applies this method to fluvial dikes for the first time, but also makes an initial attempt to reveal the complex three-dimensional flow structures beneath the free surface. Furthermore, this study to some extent bridges the gap between the work of Alvarez (2024) and the study by (Michelazzo et al. 2015) that investigated the three-dimensional flow patterns along and across a prismatic side weir.

Errors on water depth and discharge distribution between numerical models and measurement results could be mainly attributed to the presumption of steady flow in the numerical simulations while the flow is unsteady in the experiments. In the physical experiment, the water level in the main channel drops with time (the water level for the considered cases can be found in Table 2, and the hydrograph of the physical experiment can be found in Figure S9 in the Supplementary Material), indicating that the total outflow discharge is larger than the constant inflow. As the downstream regulating weir (Figure 3) was calibrated to obtain the instantaneous water levels measured at G3 (i.e., downstream part of the main channel), it can be assumed that the weir height was systematically overestimated to obtain this constant water level in steady flow conditions. For a given inflow, this results in more flow diverted through the breach, especially for later time points when the water level in the experimental tests varies the most. To tackle this limitation, future studies could consider using time varying topographic data obtained from physical experiments at multiple discrete time points to fully capture the inherent transient nature of dike breaching processes. Nevertheless, this method may require considerably greater resources for model development and more detailed spatial and temporal measurements of topography in physical model experiments.

Available datasets on levee breaching are often limited in scope and parameter coverage (Flynn et al. 2025), and such situation is further amplified by the inherent challenge of capturing instantaneous water dynamics. The critical experimental data for comparison with the model remain scarce, such as turbulence characteristics and shear distribution etc. These quantities are subject to uncertainty associated with the turbulence model assumptions, particularly the isotropic eddy viscosity hypothesis in RANS models, as well as limitations in resolving complex three-dimensional flow structures such as separation and secondary circulation. Due to the inherent challenge in measuring instantaneous velocities, the validation of the model was done solely based on the water levels and the discharge partitioning. As a result, there is some underlying uncertainty about the accuracy of the velocity and shear stress modelling results, since it was not possible to test different turbulence closures and to ascertain whether RANS modelling is able to capture adequately the flow patterns. However, in studies in an urban district with several bifurcations (e.g., Yu et al. (2026)), which share similarities with the splitting of the flow at a dike breach, a similar RANS modelling approach with k- ω SST has replicated well not only the flow depths and the discharge partitioning, but also the surface flow velocities. This brings confidence that the modelling approach in this study is suitable for this analysis. Nonetheless, eddy-resolving modelling that resolves large-scale turbulence, such as LES, should be considered in a future dike-breach study and be compared to RANS modelling to determine if eddy-resolving modelling can overcome potential RANS limitations to an extent that justifies the very large additional computational cost.

Furthermore, most physical experiments, though aiming to represent a variety of dike failure events, might not be representative of more complex real-world incidents. Several components are neglected or overly simplified in physical experiments, and thus in the numerical models in this study, including bed friction, upstream inflow variability, the realistic condition of a relatively stable main channel water level, channel curvature, etc. Such factors may potentially significantly influence the hydrodynamics and the related erosion dynamics at the breach. Therefore, future physical experiment research should focus on such cases and include

measurements of turbulence-related quantities (Gebremariam et al. 2025). With sufficiently reliable data for validation, numerical models can be more rigorously validated and bring even more confidence, enabling researchers to systematically vary influencing factors to gain deeper insights into the hydrodynamics during dike failure processes.

Lastly, the model is applied at laboratory scale in this study to resolve detailed three-dimensional hydrodynamic processes associated with breach flow, rather than to directly simulate real-world flooding scenarios. As reported in this study, the computational cost (i.e. approximately 85 hours for a 150 s simulation) highlights the current limitations of fully three-dimensional CFD models for large-scale dike failure simulation. The relatively long simulation time reflects the high computational demand for resolving complex turbulent flow patterns with secondary circulations. Nevertheless, such high-resolution simulations provide valuable process-based insights that can inform the development of simplified or reduced-order models. Future improvements in computational hardware, parallelization techniques, and numerical algorithms are expected to enhance the scalability and practical applicability of 3D CFD approaches.

5 Conclusion

Understanding the complex flow dynamics during a dike failure event is essential for enhancing the predictive accuracy of numerical tools like river models and supporting decision-making. Focusing on revealing the flow patterns during a dike failure due to overtopping, a sequence of six cases representing different instances in the dike failure process, from the experiments of Rifai et al. (2019), was simulated with RANS modelling in *OpenFOAM*. Detailed analyses of horizontal and vertical flow velocity patterns, turbulence characteristics and wall shear stresses were conducted, revealing the intricate flow dynamics during dike failure.

The outflow distribution of the numerical models presents a similar trend to the physical experiment: with the expansion of the dike breach, the water level in the main channel declines, increasing the discharge fraction to the breach and reducing the fraction that continues to the

main channel. The numerical models tend to slightly underestimate the outflow fraction at the breach at early stages and later on overestimate it a little as the breach expands.

As the breach expands, the flow structure transitions through distinct stages. Initially, the outflow into the floodplain is relatively uniform. This symmetry breaks down as the breach widens, with the outflow concentrating toward the downstream side. This shift correlates with the downstream migration and intensification of horizontal recirculation zones in the main channel. The maximum surface velocity at the breach declines with the breach expansion. The evolution of these horizontal patterns exhibits a clear time dependency (e.g. C8-3 and C8-4 show similar developed structures, while C8-5 and C8-6 share characteristics of a later, more expanded phase).

The breach-induced disturbance is also apparent in vertical flow structures within the main channel. Upstream of the breach, the streamwise velocity distribution is discernibly influenced by wall roughness. However, downstream of the breach this frictional influence is overwhelmed by strong secondary flows generated by the flow separation near the breach. The development and upstream propagation of a vertical counterclockwise vortex, coupled with distinct changes in the streamwise velocity (U_x) distribution, indicate that breach enlargement alters the vertical flow dynamics in the main channel not only downstream but also upstream of the breach. In addition, the level of water surface increases along the channel direction at the dike breach for some cases (i.e. C8-3, C8-4). Furthermore, the lateral velocity at the levee breach cross-section shows a complex, non-uniform distribution, revealing the complex flow structure at the breach. The turbulence characteristics and wall shear stresses are also linked to the breach size. For small breaches, areas of significantly elevated TKE are absent. As the breach expands, zones of high TKE were generated locally near the dike and downstream of it in the main channel. The magnitude and extent of wall shear stress (around $\sim 2-7$ pa) which far exceeds the critical threshold of the dike material) at the erodible dike in each case also correlate with the stage of breach development. In addition, the wall shear stress at the downstream side of the breach is generally larger than that at the upstream side, which is in line with the asymmetrical breach expansion toward the downstream side during dike failures.

To further improve the 3D model's robustness, additional physical experimental data are essential. However, the heavy computational cost of sophisticated 3D simulations is prohibitive for real-world scenarios, which explains why 2D dike breach models are still predominant in decision-making for real dike failure events. Nevertheless, reliable 3D simulation results can serve as valuable references for calibrating and improving parameterizations in 2D dike breach models.

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Supplemental data

Supplemental material is provided in a separated file.

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Declaration of interests

☒The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

☐The authors declare the following financial interests/personal relationships which may be considered as potential competing interests:

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