

## A 2D numerical model for simulating landslide dam failure and induced morphological changes considering multi-layer and non-uniform sediments

Jiangtao Yang<sup>ic a,b</sup>, Sandra Soares-Frazao<sup>ic a,\*</sup>, Zhenming Shi<sup>b,c</sup> and Danyi Shen<sup>d</sup>

<sup>a</sup> Institute of Mechanics, Materials and Civil Engineering, Université Catholique de Louvain, Louvain-la-Neuve 1348, Belgium

<sup>b</sup> Department of Geotechnical Engineering, College of Civil Engineering, Tongji University, Shanghai 200092, China

<sup>c</sup> Key Laboratory of Geotechnical & Underground Engineering of Ministry of Education, Tongji University, Shanghai 200092, China

<sup>d</sup> Institute of Geotechnical Engineering, College of Civil Engineering and Architecture, Zhejiang University, Hangzhou 310058, China

\*Corresponding author. E-mail: sandra.soares-frazao@uclouvain.be

 JY, 0009-0001-6148-1480; SS-F, 0000-0001-5088-0278

### ABSTRACT

Landslide dams are formed by the blockage of rivers with landslides or any debris flows. Such dams are prone to failure and therefore, simulation of their failure and induced morphological changes is of great importance for hazard mitigation. Multi-layer and non-uniform sediments are usually observed in landslide dam materials, but these are rarely considered in simulations. In this paper, a two-dimensional numerical model considering multi-layered and non-uniform sediments is proposed. The model solves the shallow water equations, the Exner equation considering the different size fractions and Hirano's active layer equation based on a finite volume scheme in a weakly coupled approach. Two experimental tests are used to assess the applicability and accuracy of the model. The results show that it can well simulate the flow and morphological changes, as well as the surface coarsening and fining phenomena related to multi-layer and non-uniform sediments. The numerical results of dam failure process and morphological changes are in overall good agreement with the experiments, but the peak discharge and bed elevation tend to be underestimated when finer material is considered. Finally, the influence of sediments fractions and active layer depth on numerical results, as well as the limitations of the model are discussed.

**Key words:** dam failure, finite volume scheme, Hirano's active layer model, morphological changes, multi-layer and non-uniform sediments, shallow water equations

### HIGHLIGHTS

- A numerical model considering multi-layered and non-uniform sediments is developed.
- The applicability and accuracy of the model in simulating dam failure and dam-break flow with multi-layered and non-uniform sediments are verified.
- The influence of sediments fractions and the active layer depth on numerical results is discussed.

## 1. INTRODUCTION

Landslide dams are natural dams that are formed by the blockage of river channels with landslides, rockfalls, or debris flows (Costa & Schuster 1988). The failure of landslide dams is one of the essential issues related to disasters in mountainous areas. The flood induced by the failure of such a dam may bring catastrophic disasters in the downstream area, and significantly alter the local topography and environment (Korup 2004). For instance, the flood induced by the failure of Yigong landslide has caused hundreds of deaths in the downstream area in India (Delaney & Evans 2015). The failure of Tangjiashan landslide dam, which was formed by the 2008 Wenchuan earthquake in China, has induced several meters of deposits and dramatically altered the local morphology (Cui *et al.* 2010). More recently, the flood induced by the failure of two Baige landslide dams, which occurred in 2018 at the boundary of Tibet and Sichuan Province, China, has inundated quantities of farmland and resident area over 600 km in the downstream area (Yang *et al.* 2022). Therefore, analysis of dam failure-induced flooding and local morphological changes is of great importance for hazard management.

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Research on landslide dam failure and induced morphological change are carried out based on field investigation, experimental and numerical studies (Coleman *et al.* 2002; Casagli *et al.* 2003; Cao *et al.* 2011), the latter being widely used. The numerical study of landslide dam failure can be further divided into the following two categories: simplified models and physics-based models. Simplified models are usually used to predict the discharge hydrograph during dam breaching and the results can be further used in flood routing analysis in combination with hydrodynamic software, such as HEC-RAS. These models use empirical erosion rate equations and the assumption of a broad crested weir to calculate the discharge and breach parameters, such as the DABA model and the DB-IWHR model (Zhong *et al.* 2018). Although these models are efficient in simulating landslide dam failure, several input parameters and also the empirical breach evolution pattern have to be determined before using the model. As the results are highly sensitive to these choices, this may lead to a series of uncertainties during the simulation. In addition, as the flood routing in the downstream area is simulated separately, the morphological changes induced by the dam failure are not considered.

Physics-based models, in which the hydrodynamic and morpho-dynamic equations are considered, have been proposed to simulate landslide dam breaching and induced morphological changes (Balmforth *et al.* 2008; Cao *et al.* 2011). These models can well represent the interaction between flow and sediments and are widely used in dam breaching simulations (Soares-Frazão & Zech 2010; Chen & Zhang 2015; Juez *et al.* 2014; Meurice & Soares-Frazão 2020). However, in these models, the sediments are treated as a uniform material with a unique median diameter ( $d_{50}$ ), thus, leading to inaccurate simulation results. In recent years, several numerical models considering non-uniform sediments have been developed. These models can be divided into the following two categories: the two-phase models or the quasi-single-phase models (Li *et al.* 2018; Xia *et al.* 2018; Martínez-Aranda *et al.* 2022) and the active layer models (Stecca *et al.* 2014; Juez *et al.* 2016; Chavarrías *et al.* 2019, 2022).

In both types of numerical models, the non-uniform sediments are divided into several fractions with different median diameters ( $d_{50}$ ), while different algorithms are considered to calculate the transport of these sediments. In the two-phase model or the quasi-single-phase model, the flow and sediments are assumed to be well mixed in the flow, and the models solve the solid and fluid phases using distinct equations. The mass conservation equation is then considered in each of the sediment fractions associated with the non-uniform sediments (Pudasaini 2012; Johnson *et al.* 2012). These models have been widely used to simulate the propagation of sediment laden flow such as mudflow or debris flow, while ignoring the variation of the bed material composition during flow propagation. In terms of the active layer model, two layers of sediments are considered in the erodible bed, including the upper active layer and the bottom substrate layer. The interactions between the flow and the sediments are mainly considered in the upper active layer. Thus, the sediment transport rate, the evolution of morphology and the change of surface grain size distribution can be calculated within the active layer based on the sediment transport rate of each fraction and its percentage in the total sediment (Hirano 1971).

The active layer model has been widely used to simulate the flow propagation and morphological changes related to a non-uniform erodible bed, but not under severe transient flow conditions. However, when considering dam failure and the induced morphological changes, especially in the case of landslide dams, the material composition is one of the main characteristics that significantly influences the dam failure and the sediment transport (Mei *et al.* 2021). Current numerical models rarely consider the material characteristics of landslide dams, especially their non-uniform material composition and multi-layered characteristics, leading to the inaccurate results associated with the peak discharge, breach evolution duration during dam failure.

In this paper, a two-dimensional numerical model is developed with non-uniform and multi-layer sediments to simulate dam failure and induced morphological changes. The model is solved in a weakly coupled way based on a finite volume scheme by Soares-Frazão & Zech (2010), and two flume tests are used to verify the applicability and accuracy in simulating dam failure and morphological changes. The paper is organized as follows: In Section 2, the three parts of the numerical model, including the hydrodynamic module, the morpho-dynamic module and the active layer module are introduced. Then, the numerical scheme is explained in Section 3. In Section 4, the proposed model is verified by simulating two laboratory tests: (i) dam-break flow test over movable bed and (ii) dam failure test with different non-uniform sediments. In Section 5, the influence of the sediment fraction and active layer depth on numerical results, and also the limitations of the model are discussed. Finally, the conclusions arising from the current work are described in Section 6.

## 2. MODEL CONCEPTION

The numerical model in which the non-uniform and multi-layer sediments are considered is shown in Figure 1. In this model, the sediments in the movable bed are divided into several layers numbered from #1 to #n, with the upper active layer included in layer #1. In each layer, sediments are further divided into fractions with different median diameter and percentage content. The numerical model is composed of three parts, including the hydrodynamic module, the morpho-dynamic module and the module for the variation of surface grain size distribution. The governing equations and characteristics of each module are introduced in the following section.

### 2.1. Governing equations

#### 2.1.1. Hydrodynamic module

The hydrodynamic module is used to calculate the flow field during the simulation and it consists of the following three governing equations (shallow water equations):

$$\frac{\partial h}{\partial t} + \frac{\partial(hu)}{\partial x} + \frac{\partial(hv)}{\partial y} = 0 \quad (1)$$

$$\frac{\partial(hu)}{\partial t} + \frac{\partial(hu^2 + (1/2)gh^2)}{\partial x} + \frac{\partial(huv)}{\partial y} = gh(S_{0,x} - S_{f,x}) \quad (2)$$

$$\frac{\partial(hv)}{\partial t} + \frac{\partial(huv)}{\partial x} + \frac{\partial(hv^2 + (1/2)gh^2)}{\partial y} = gh(S_{0,y} - S_{f,y}) \quad (3)$$

where  $h$  is the water depth,  $u$  and  $v$  are the flow velocity in the  $x$  and  $y$  directions, respectively,  $g$  is the gravitational acceleration,  $S_{0,x} = -(\partial z_b / \partial x)$  and  $S_{0,y} = -(\partial z_b / \partial y)$  are the bed slopes in  $x$  and  $y$  directions, with  $z_b$  being the bed level, and

$$S_{f,x} = \frac{n^2 \sqrt{u^2 + v^2} u}{h^{4/3}} \text{ and } S_{f,y} = \frac{n^2 \sqrt{u^2 + v^2} v}{h^{4/3}}$$

are the friction slopes in the  $x$  and  $y$  directions, with  $n$  being the Manning coefficient.

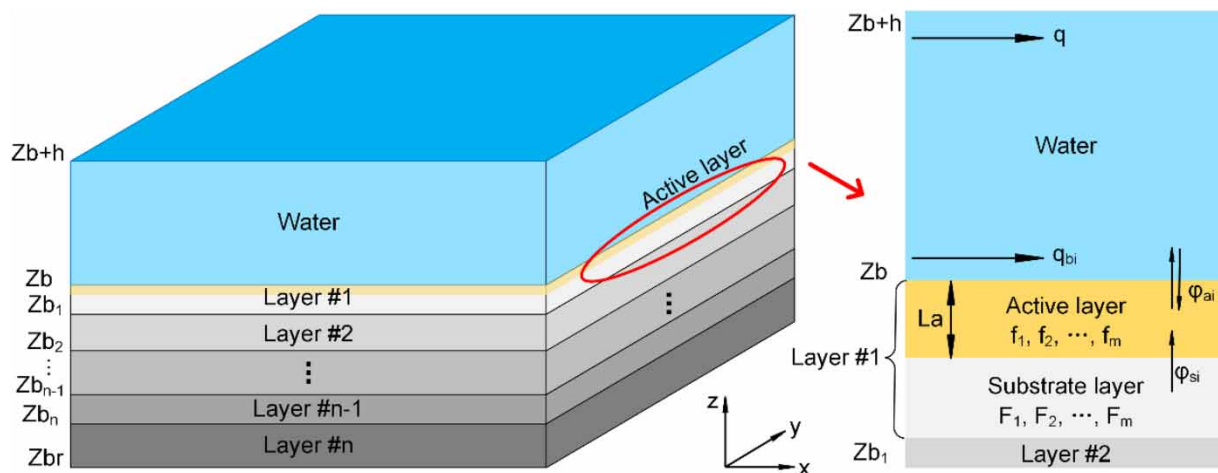


**Figure 1** | Sketch of the numerical model with non-uniform and multi-layer sediments.

### 2.1.2. Morpho-dynamic module

The morpho-dynamic module is used to update the bed level during the simulation. For this purpose, a modified Exner equation considering non-uniform and multi-layer sediments is used, that can be expressed as follow:

$$\frac{\partial z_b}{\partial t} + \frac{1}{1-\varepsilon} \frac{\partial(\sum_{i=1}^N f_i q_{si,x})}{\partial x} + \frac{1}{1-\varepsilon} \frac{\partial(\sum_{i=1}^N f_i q_{si,y})}{\partial y} = 0 \quad (4)$$

where  $\varepsilon$  is the bed porosity, which is considered as constant during simulation and can be calculated as  $\varepsilon = 1 - \rho/\rho_s$ , with  $\rho$  and  $\rho_s$  being the density of the multi-fraction material and the grain density respectively.  $N$  is the total number of sediments fractions,  $f_i$  is the percentage content of the  $i$ th sediment fraction (Figure 1),  $q_{si,x}$  and  $q_{si,y}$  are the unit sediment transport rates of the  $i$ th sediment fraction in the  $x$  and  $y$  directions. These transport rates are calculated using a closure equation as described in the following.

### 2.1.3. Surface grain size variation module

The variation of surface grain size distribution is simulated with the Hirano's active layer equation (Hirano 1971). The equation can be expressed as follow for each sediment fraction  $i$  from the  $N$  fractions:

$$\frac{\partial(f_i L_a)}{\partial t} = -\frac{1}{1-\varepsilon} \left( \frac{\partial(f_i q_{si,x})}{\partial x} + \frac{\partial(f_i q_{si,y})}{\partial y} \right) - f_{esi} \frac{\partial(z_b - L_a)}{\partial t} \quad (5)$$

where  $L_a$  is the active layer depth. The left-hand side of Equation (5) refers to the update of the percentage content of each sediment fraction in the active layer. In the right-hand side of the equation, the first term accounts for the interactions between the bed load and the active layer, while the second term refers to the sediment exchange between the active layer and the substrate layer. The  $f_{esi}$  coefficient in the second term is an empirical parameter that can be determined by considering whether the bed is under degradation or aggradation condition. If the bed degrades, the active layer moves downwards, then the exchange between the active and substrate layers is determined by the substrate layer, while if aggrades, the sediment exchange is controlled by the bed load and active layers. Therefore,  $f_{esi}$  can be expressed as follow:

$$f_{esi} = \begin{cases} F_i & \text{if degradation} \\ af_i + (1-a)f_{bi} & \text{if aggradation} \end{cases} \quad (6)$$

where  $a$  is an empirical parameter ranging from 0 to 1 and  $f_{bi}$  is the bedload transport rate of the  $i$ th sediment fraction among the total bedload sediment transport rate ( $q_s$ ) in the bedload transport layer and can be calculated as follows:

$$f_{bi} = \frac{f_i \sqrt{q_{si,x}^2 + q_{si,y}^2}}{q_s} \quad (7)$$

Based on Equations (5) and (6), the median diameter of sediments in the active layer can then be calculated as follow:

$$d_{50} = f(f_1, d_{50,1} \dots f_i, d_{50,i} \dots f_N, d_{50,N}) \quad (8)$$

where  $d_{50,i}$  is the median diameter of the  $i$ th sediment fraction. The function linking the median diameter of the total mixture and that of each sediment fraction can either be determined by a simplified linear relationship or using a regression analysis.

## 2.2. Closure equations

The numerical model consists of  $N + 4$  equations, i.e. Equations (1)–(4) and the  $N$  Equation (5) for the  $N$  fractions, with  $N + 4$  unknown parameters. Closure equations are required for the sediment transport rate  $q_s$ , the active layer depth  $L_a$  and the Manning coefficient  $n$  of the bed that changes according to bed surface composition.

### 2.2.1. Sediment transport rate

The sediment transport rate in each sediment fraction ( $q_{si}$ ) is calculated using the Meyer-Peter and Müller (MPM) formula:

$$q_{si} = 8\sqrt{g(s-1)d_{50}^3(\theta_i - \theta_{ci})^{3/2}} \quad (9)$$

where  $s = \rho_s/\rho_w$  refers to the ratio of grain density and water density,  $\theta_i$  and  $\theta_{ci}$  refer to the dimensionless bed shear stress (Shields number) and critical bed shear stress of the  $i$ th sediment fraction. The dimensionless bed shear stress in each fraction can be expressed as:

$$\theta_i = \frac{n^2}{(s-1)d_{50,i}h^{1/3}}(u^2 + v^2) \quad (10)$$

where  $n$  is the Manning friction coefficient. Taking the hiding-exposure effect into consideration, the formula proposed by Ashida & Michiue (1972) is used to calculate the dimensionless critical shear stress in each fraction. The formula can be expressed as follow:

$$\theta_{ci} = \begin{cases} c \left[ \frac{\log 19}{\log \left( \frac{19d_{50,i}}{d_{50}} \right)} \right]^2 & \theta \frac{d_{50,i}}{d_{50}} \geq 0.4 \\ \theta_c \frac{d_{50}}{d_{50,i}} & \frac{d_{50,i}}{d_{50}} < 0.4 \end{cases} \quad (11)$$

where  $\theta_c$  is the dimensionless critical shear stress of the whole sediment mixture, which is directly determined by the median diameter of the sediment mixture and can be described as follow (Wu *et al.* 2000):

$$\theta_c = \begin{cases} 0.0173D_*^{0.19}, & 20 \leq D_* < 40 \\ 0.0115D_*^{0.30}, & 40 \leq D_* < 150 \\ 0.052, & D_* \geq 150 \end{cases} \quad (12)$$

with

$$D_* = d_{50} \left[ \frac{((\rho_s/\rho) - 1)g}{v^2} \right]^{1/3}$$

being the non-dimensional particle size of the mixture with median diameter  $d_{50}$ .

### 2.2.2. Active layer depth

The active layer depth mainly controls the total volume of sediment in the active layer, which significantly influences the update of the percentage content in each sediment fraction. In this paper, the active layer depth is determined by the  $d_{90}$  of the sediments and can be expressed as follow:

$$L_a = kd_{90} \quad (13)$$

where  $k$  is an empirical coefficient ranging from 1 to 3 (Juez *et al.*, 2016). In addition, since multiple sediment layers are considered in the numerical model, the variation of the active layer depth is described according to three scenarios associated with the change of layers, as shown in Figure 2. In Scenario (a), the active layer is totally located in the upper layer, then it can be directly calculated by Equation (13). In Scenario (b), the active layer overlaps two layers, then  $L_a$  is the maximum value according to Equation (13) considering the  $d_{90}$  of the two layers. In Scenario (c), the active layer is located directly above the fixed bed, in this way,  $L_a$  equals to the depth of the movable sediments.

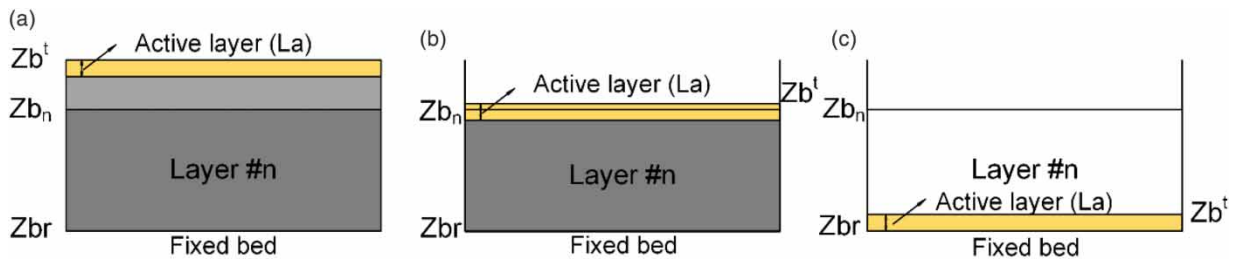


**Figure 2** | Variation of the active layer depth: (a) active layer within one sediment layer; (b) active between two sediment layers; (c) active directly over the fixed bed.

### 2.2.3. Manning coefficient

Wu (2013) proposed a formula that has been widely used to calculate the Manning coefficient based on the median diameter:

$$n = \frac{d_{50}^{1/6}}{A_n} \quad (14)$$

where  $A_n$  is an empirical coefficient taking the value  $A_n = 16$  for laboratory scale tests. However, sediment here are divided into several fractions to represent the widely graded characteristics of landslide dam material. In order to take the different sediment fractions into account, the Einstein assumption of different subsections with equal velocity is used here to calculate the Manning coefficient.

Assuming the flow velocity is the same in each subsection corresponding to each sediment fraction, the following equalities can be written:

$$U = \frac{1}{n} R_b^{2/3} S_f^{1/2} = \frac{1}{n_1} R_{b1}^{2/3} S_f^{1/2} = \dots = \frac{1}{n_N} R_{bN}^{2/3} S_f^{1/2} \quad (15)$$

where  $U$  is the flow velocity,  $S_f$  is the energy slope,  $R_b$ ,  $R_{b1}$ , and  $R_{bN}$  are the hydraulic radii corresponding to the sediment mixture, to the first and the  $N$ th fraction of sediment, respectively. With the Einstein assumption, the hydraulic radius can be further expressed as:

$$R_b = \sum_{i=1}^N f_i R_{bi} \quad (16)$$

Combining Equations (15) and (16), the Manning coefficient of the entire sediment can then be expressed as:

$$n = \left( \sum_{i=1}^N f_i n_i^{3/2} \right)^{2/3} \quad (17)$$

where  $n_i$  is the Manning coefficient in each sediment fraction and can be calculated with Equation (14).

## 3. NUMERICAL SCHEME

The model equations are solved based on a first order finite volume scheme over an unstructured triangular mesh (Soares-Frazão & Zech 2010). As the solution of the active layer model coupled with the shallow water equations may be ill-posed due to the change of mathematical character under specific conditions (Chavarrías *et al.* 2019), a weakly coupled approach proposed by Juez *et al.* (2014, 2016) is used here to solve the governing equations. In this approach, the shallow water equations, Exner equation and Hirano's active layer equation are solved separately. The details of the numerical scheme of each module are given in the following.

### 3.1. Hydrodynamic numerical solution

Within a local co-ordinate system attached to each interface with  $(n_x, n_y)^T$  the unit vector normal to the interface, Equations (1)–(3) then can be written in a matrix form as (Soares-Frazão & Zech 2010):

$$\frac{\partial U_n}{\partial t} + \frac{\partial F(U_n)}{\partial n_x} + \frac{\partial G(U_n)}{\partial n_y} = S \quad (18)$$

where  $U_n$  is conservation term in the local co-ordinate system,  $F(U_n)$  and  $G(U_n)$  are the fluxes term perpendicular and along the interface, respectively, and  $S$  is the source term. These four vectors are expressed as:

$$U_n = T_t(h, hu, hv)^T = (h, hu_n, hv_n)^T = (h, q_{nx}, q_{ny})^T \quad (19)$$

$$F(U_n) = \begin{bmatrix} q_{nx} \\ \frac{q_{nx}^2}{h} + \frac{1}{2}gh^2 \\ q_{nx}q_{ny}/h \end{bmatrix} = \begin{bmatrix} q_{nx} \\ \sigma_{nx} \\ \mu \end{bmatrix}, \quad G(U_n) = \begin{bmatrix} q_{ny} \\ q_{nx}q_{ny}/h \\ \frac{q_{ny}^2}{h} + \frac{1}{2}gh^2 \end{bmatrix} = \begin{bmatrix} q_{ny} \\ \sigma_{ny} \\ \mu \end{bmatrix} \quad (20)$$

$$S = (0, -ghS_{f,x}, -ghS_{f,y})^T \quad (21)$$

with  $T_t$  being the matrix related to the co-ordinate system transformation.

Based on the fluxes calculated across the interface between two adjacent cells, the first order finite volume scheme for solving the shallow water equations over a computational cell can be expressed as:

$$U_m^{n+1} = U_m^n - \frac{\Delta t}{\Omega_m} \sum_{j=1}^k T_j^{-1} F_j^*(U_j) L_j + S_{f,m} \Delta t \quad (22)$$

where  $U_m^{n+1}$  presents the state of the conservation term at time  $n + 1$  in cell  $m$ ,  $\Delta t$  refers to the time step,  $\Omega_m$  is the cell area,  $k$  is the interface number ( $k = 3$  in a triangular mesh),  $L_j$  is the  $j$ th-interface length,  $F_j^*(U_j)$  is the relevant flux term through the  $j$ th interface and can be calculated using the lateralized HLLC scheme as follow:

$$F_j^*(U_j) = \begin{bmatrix} q_{nx}^* \\ \sigma_{nx}^* \\ \mu \end{bmatrix} = \begin{bmatrix} \frac{\lambda^+ q_{nx,L} - \lambda^- q_{nx,R} + \lambda^+ \lambda^- (z_{w,R} - z_{w,L})}{\lambda^+ - \lambda^-} \\ \frac{\lambda^+ \sigma_{nx,L} - \lambda^- \sigma_{nx,R} + \lambda^+ \lambda^- (q_{nx,R} - q_{nx,L})}{\lambda^+ - \lambda^-} \\ \begin{cases} v_{n,L} q_{nx}^* & \text{if } \lambda_2 \geq 0 \\ v_{n,R} q_{nx}^* & \text{if } \lambda_2 < 0 \end{cases} \end{bmatrix} \quad (23)$$

where

$$\lambda^+ = \max(\lambda_{3,L}, \lambda_{3,R}, 0) \quad (24)$$

$$\lambda^- = \max(\lambda_{1,L}, \lambda_{1,R}, 0) \quad (25)$$

with  $\lambda_1 = u_n - c$ ,  $\lambda_2 = u_n$  and  $\lambda_3 = u_n + c$  being the hydrodynamic characteristics related to the Jacobian matrix of shallow water equations (Soares-Frazão & Zech 2010).

### 3.2. Morpho-dynamic numerical solution

In the weakly coupled method proposed by Juez *et al.* (2014, 2016) and Meurice & Soares-Frazão (2020), a morpho-dynamic numerical characteristic  $\lambda_s$  is proposed to solve the Exner equation using a finite volume with HLLC-type flux approach. Equation (4) can be written in the local co-ordinate system attached to the considered interface with  $\lambda_s$  as:

$$\frac{\partial z_b}{\partial t} + \lambda_s \frac{\partial z_b}{\partial n_x} = 0 \quad (26)$$

where

$$\lambda_s = \frac{1}{1 - \varepsilon} \frac{\Delta(\sum_{i=1}^N f_i q_{si,nx})}{\Delta z_b} \quad (27)$$

with  $q_{si,nx}$  being the unit sediment transport rate of the  $i$ th sediment fraction in the direction perpendicular to the interface.

The finite volume numerical scheme for the modified Exner equation can then be expressed as:

$$z_{bm}^{n+1} = z_{bm}^n - \frac{1}{1 - \varepsilon} \frac{\Delta t}{\Omega_m} \sum_{j=1}^k \sum_{i=1}^N f_i q_{si,nx,j} L_j \quad (28)$$

where

$$q_{si,nx,j} = \begin{cases} q_{si,nx,L} & \text{if } \lambda_s \geq 0 \\ q_{si,nx,R} & \text{if } \lambda_s < 0 \end{cases} \quad (29)$$

with  $q_{si,nx,L}$  and  $q_{si,nx,R}$  being the unit sediment transport rates of  $i$ th sediment fraction perpendicular to the interface of left and right cells, respectively.

Since a multi-layered sediment bed is considered in the numerical model, the variation of sediment parameters (bed composition) occurring due to the update of the erosion surface elevation has to be accounted for during the simulation. As illustrated in Figure 3, the variation of sediment parameters can be described considering four scenarios according to the degradation or aggradation of the bed and the sediment layer composition change. Starting from the initial situation illustrated in Figure 3(a) with the sediment level  $z_b^t$ , Scenario (b) corresponds to a bed degradation to level  $z_b^{t+1}$  without change in the lower level of the sediment layer. However, it must be noted that the composition of the active layer, and hence its median diameter, can be changed. In Scenario (c), the bed also degrades and the sediment level moves to the lower layer; therefore, the sediment parameters of the lower layer are considered in the simulation. In Scenarios (d) and (e), the bed aggrades and the variation of sediment parameters in the active layer is considered.

### 3.3. Active layer model solution

The numerical scheme of the active layer model can be divided into two parts related to the two terms at the right-hand side of Equation (5). A similar approach as the morpho-dynamic solution is used for the first term and a numerical characteristic  $\lambda_{sg,i}$  is proposed for each sediment fraction. The second term on the right-hand side of Equation (5) can be treated as a source term, in the same way as in Equation (18). Therefore, Equation (5) can then be written as:

$$\frac{\partial(f_i L_a)}{\partial t} = -\lambda_{sg,i} \frac{\partial(f_i L_a)}{\partial n_x} - f_{esi} \frac{\partial(z_b - L_a)}{\partial t} \quad (30)$$

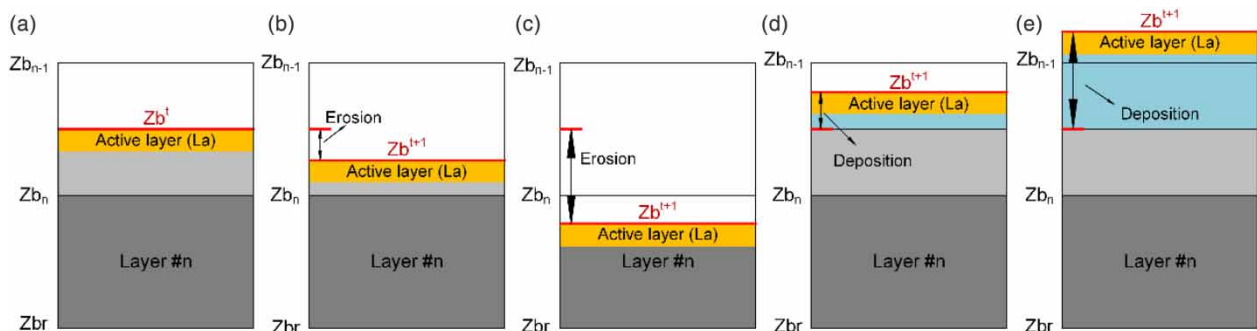


**Figure 3** | Parameters change related to the multi-layer sediments: (a) bed condition at time  $t$ ; (b) bed degradation without layer change at time  $t + 1$ ; (c) bed degradation with layer change at time  $t + 1$ ; (d) bed aggradation without layer change at time  $t + 1$ ; (e) bed aggradation with layer change at time  $t + 1$ .

where

$$\lambda_{sg,i} = \frac{1}{1 - \varepsilon} \frac{\Delta q_{si,nx}}{\Delta(f_i L_a)} \quad (31)$$

The finite volume numerical scheme for the Hirano's active layer equation can then be expressed as:

$$(f_i L'_v)_m^{n+1} = (f_i L_a)_m^n - \frac{\Delta t}{\Omega_m} \sum_{j=1}^k \varepsilon q_{si,nx,j}^* L_j \quad (32)$$

$$(f_i L_a)_m^{n+1} = (f_i L'_a)_m^{n+1} - f_{esi} \Delta(z_b - L'_a) \quad (33)$$

where  $L'_a$  is a temporary value for active layer thickness  $L_a$ . It should be noted that the active layer thickness  $L_a$  should be larger than 0, so as to update the percentage content of each sediment fraction. If  $L_a \leq 0$ , then the percentage content of each sediment fraction is assumed to be 0. In addition, the term  $q_{si,nx,j}$  in Equation (32) can be calculated as follows:

$$q_{si,nx,j} = \begin{cases} q_{si,nx,L} & \text{if } \lambda_{sg,i} \geq 0 \\ q_{si,nx,R} & \text{if } \lambda_{sg,i} < 0 \end{cases} \quad (34)$$

### 3.4. Numerical stability

The Courant–Friedrichs–Lewy (CFL) condition is applied to ensure the stability of the numerical scheme while solving the equations. Therefore, the time step with consideration of the CFL condition is calculated as:

$$\Delta t = \frac{C \Delta x}{\lambda_{\max}} \quad (35)$$

where  $C$  is the CFL number usually taken as  $C = 0.9$ ,  $\Delta x$  is a reference length in the computational cell (e.g., the radius of the incircle for a triangular cell),  $\lambda_{\max}$  is the maximum value of numerical characteristics related to the shallow water equations, the modified Exner equation and the Hirano's active layer equation, and can be calculated as follow:

$$\lambda_{\max} = \max(\lambda_1, \lambda_2, \lambda_3, \lambda_s, \lambda_{sg,i}) \quad i = 1, \dots, N - 1 \quad (36)$$

## 4. MODEL VALIDATION

In this section, a synthetic test and two laboratory tests are considered to assess the accuracy and suitability of the proposed numerical model, and also to clarify the active layer assumption. The synthetic test is a sediment feed test and is used to clarify if the model can adjust to the equilibrium state under initial aggradation and degradation conditions considering non-uniform sediments. The first experimental test is used to investigate a dam-break flow and its induced morphological changes over movable bed with uniform sediments (Soares-Frazão *et al.* 2012). Based on this test, three numerical scenarios with both uniform and non-uniform sediments are analyzed. The second experimental test aims to investigate landslide dam failure and the induced morphological changes, in which three scenarios with different non-uniform sediments are considered. Furthermore, the numerical results related to the variation of water level and discharge during dam failure, the dam breaching process and the downstream morphological change are compared with the experimental data.

### 4.1. Synthetic test: sediment feed test under aggradation and degradation conditions

The synthetic test was carried out in a 4 m long and 10 m wide rectangular channel with a triangular mesh of 0.5 m (average edge length). The bedrock level at the bottom of the channel remained at 0, and the channel was filled with sediments with a specific bed slope. The following two numerical tests were considered: first a test considering uniform sediments and then a test considering non-uniform sediments with two fractions. The sediments used to fill the channel in both tests had a median diameter of 1.7 mm, a Manning coefficient of  $0.00167 \text{ s/m}^{1/3}$ , a porosity of 0.44 and a relative specific gravity of 2.65. A

constant unit water inflow rate and unit sediment feed rate of 0.05 and  $0.00098 \text{ m}^3/\text{s}/\text{m}$ , respectively, were imposed at the upstream channel boundary, while a transmissive boundary was considered at the downstream boundary of the channel. Based on the transport equation for calculating the sediment transport rate (Equation (9)) in the present model, the equilibrium bed slope was estimated to be 5%, with an initial water depth of 0.035 m. Furthermore, two additional bed slopes of 6 and 4%, which represent the aggradation and degradation conditions, were considered as initial conditions to test the accuracy and stability of the present model. It should also be noted that in the numerical test considering non-uniform sediments, the median diameter of each fraction was 1 and 2.4 mm, respectively, and the percentage content of each fraction was equal to 50%. Therefore, the corresponding unit sediment feed rate of each fraction was estimated to be 0.0005 and  $0.00048 \text{ m}^3/\text{s}/\text{m}$ , respectively.

Figure 4 shows the results of the synthetic tests. Both the numerical tests considering uniform sediments and non-uniform sediments with two fractions can adjust well to the expected equilibrium state. When the bed aggrades, the bed slope decreases until the equilibrium slope is observed, while it increases to the equilibrium slope when the bed degrades. The difference associated with the bed slope between tests considering uniform and non-uniform sediments is negligible.

#### 4.2. Test 1: Dam-break flow over movable bed

The first experimental test was conducted at Université catholique de Louvain in a 36 m long and 3.6 m wide flume. A sketch of the flume is shown in Figure 5 and the details of the experimental set-up was illustrated in Soares-Frazão *et al.* (2012). In this test, the upstream and downstream water levels were 0.470 and 0.085 m, respectively, and a movable bed extending over 1.5 m upstream and 9.0 m downstream of the gate was considered. The movable bed had a depth of 8.5 cm, which consisted

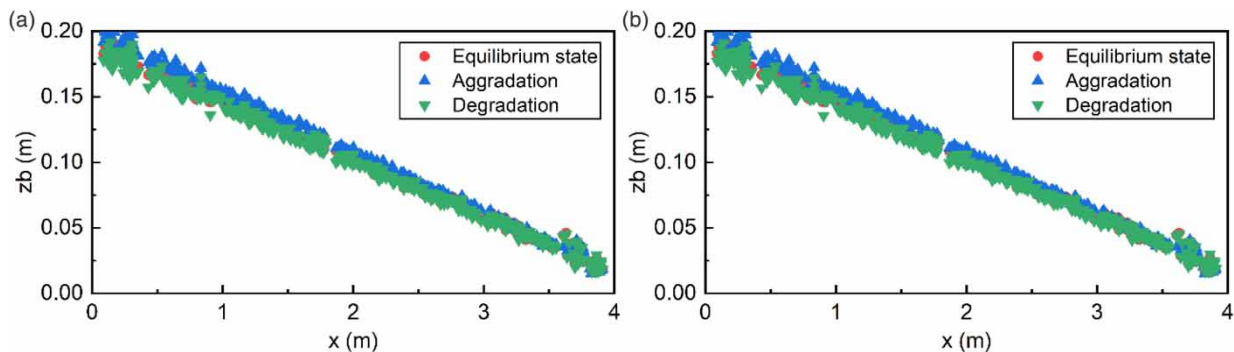


**Figure 4** | Final bed slope under equilibrium state, aggradation and degradation conditions: (a) uniform sediments and (b) non-uniform sediments with two fractions.

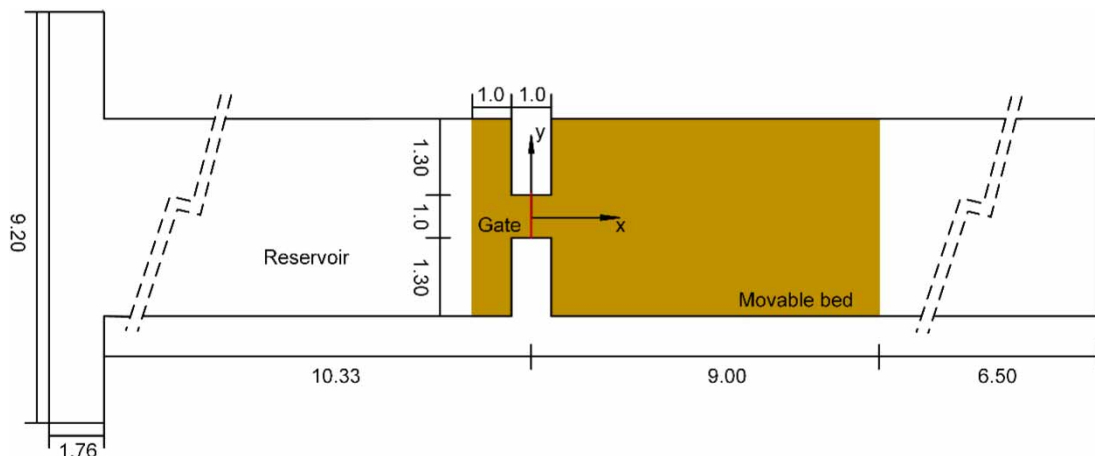


**Figure 5** | Plane view of the flume and its set-up.

of uniform sand with a median diameter of 1.61 mm, a relative specific gravity of 2.63 and a porosity of 0.42. In the present simulations, this case is referred to as Scenario 1. Furthermore, another two scenarios considering three layers of uniform and non-uniform sediments (Scenarios 2 and 3) were designed based on Scenario 1 to verify the ability of the model to simulate the flow and the entrainment of non-uniform and multi-layer sediment induced by a dam-break flow. The details of the three numerical scenarios are shown in Table 1. In Scenario 2, three layers of uniform sediments are considered, and the sediments in each layer is as same as in Scenario 1. In Scenario 3, three layers of non-uniform sediments with depth of 0.028, 0.028 and 0.029 m are considered. The sediments in each layer consist of three fractions (with a median diameter of 0.5, 2 and 9 mm, respectively). The median diameter of the three-layer sediment ranges from 1.85 to 5.84 mm by varying the percentage content of each fraction. The Manning coefficient of Scenarios 1 and 2 is the same as the experimental value ( $0.0165 \text{ s/m}^{1/3}$ ), while for Scenario 3, the coefficient is calculated with Equation (16).

Numerical simulations for the three scenarios were conducted on an unstructured triangular mesh with a spatial resolution of 0.1 m. Two monitor points located at (0.64, -0.5) for Point 1 and (1.94, -0.33) for Point 2 are selected to compare the water level variation that is obtained from Scenario 1, Scenario 2 and the experimental results. The final topography and the bed elevation along four profiles ( $y = 0$ ,  $y = 0.2 \text{ m}$ ,  $y = 1 \text{ m}$  and  $y = 1.45 \text{ m}$ ) are compared between the numerical and experimental results. Furthermore, the surface grain size distribution and sediment transport rate related to the non-uniform and multi-layer sediments are analyzed based on the results of Scenario 3.

#### 4.2.1. Numerical results of Scenarios 1 and 2

Water level variation at two monitor points is shown in Figure 6. The numerical results of Scenarios 1 and 2 are in good agreement with each other, and both can well replicate the experimental data. In addition, compared with the water level variation at Point 1, which is located closer to the gate, the numerical results are in better agreement with the experimental data at Point 2. This discrepancy can be attributed to the fact that the shallow water equations are depth-averaged and thus

**Table 1** | Numerical scenarios and the parameters

| Scenario | L | Parameters          |      |                      |      | F | P    |      |      |     |   |   |
|----------|---|---------------------|------|----------------------|------|---|------|------|------|-----|---|---|
|          |   | d <sub>50</sub> /mm | s    | n/sm <sup>-1/3</sup> | ε    |   |      |      |      |     |   |   |
| S1       | 1 | 1.61                | 2.63 | 0.0165               | 0.42 | 1 | 1    |      |      |     |   |   |
| S2       | 3 | 1.61                | 2.63 | 0.0165               | 0.42 | 1 | 1    |      |      |     |   |   |
|          |   |                     |      |                      |      | 1 | 1    |      |      |     |   |   |
|          |   |                     |      |                      |      | 1 | 1    |      |      |     |   |   |
| S3       | 3 | 1.85                | 2.63 | 0.0174               | 0.4  | 3 | 0.57 | 0.33 | 0.10 | 0.5 | 2 | 9 |
|          |   | 3.89                | 2.63 | 0.0185               | 0.3  | 3 | 0.33 | 0.33 | 0.34 |     |   |   |
|          |   | 5.84                | 2.63 | 0.023                | 0.25 | 3 | 0.10 | 0.33 | 0.57 |     |   |   |

Note: L is the number of layers and F is the number of sediments fractions in each layer, P is the percentage content of each sediments fraction,  $d_{50}$  is the median diameter of each layer, s is the relative specific gravity,  $\varepsilon$  is the porosity and n is the Manning coefficient,  $d_{50,i}$  is the median diameter of each sediment fraction.

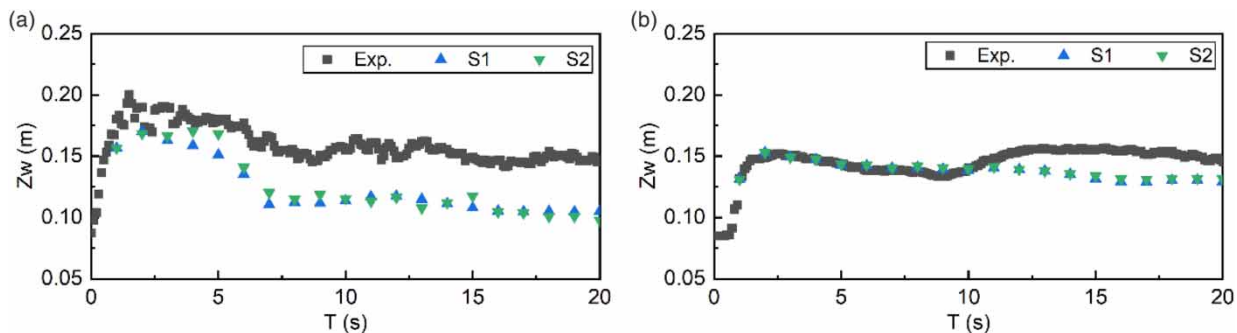


**Figure 6** | Water level variation at (a) point 1 (0.64, -0.5) and (b) point 2 (1.94, -0.33).

unable to consider the vertical water velocity that is induced by instantaneous removal of the gate. However, the numerical model can well simulate the peak water level and the trend of water level variation in general.

When it comes to the final topography, in the same way as for the water level variation, the numerical results obtained from Scenarios 1 and 2 are identical (Figure 7). The final bed topography shows two zones, including the erosion zone and the deposition zone. The erosion zone is located closer to the gate. Sediments that are eroded from this area are transported in all downstream directions due to the abrupt widening of the flume, leading to the formation of the deposition zone where sediments deposit around the edge of the erosion zone. Due to the abrupt widening of the flume, the maximum deposition depth is observed at the two sides of the flume, and it gradually decreases from the boundary to the center of the flume, as shown in Figure 7. Furthermore, the numerical results are compared with the experimental data based on the bed elevation along four profiles, as shown in Figure 8. The numerical results of bed elevation along different profiles are all in good agreement with the experimental data. Specifically, the numerical model can well simulate the range and the location of both the erosion and deposition zones. In addition, both the maximum erosion and deposition depth obtained from numerical simulations are in good agreement with the measured data.

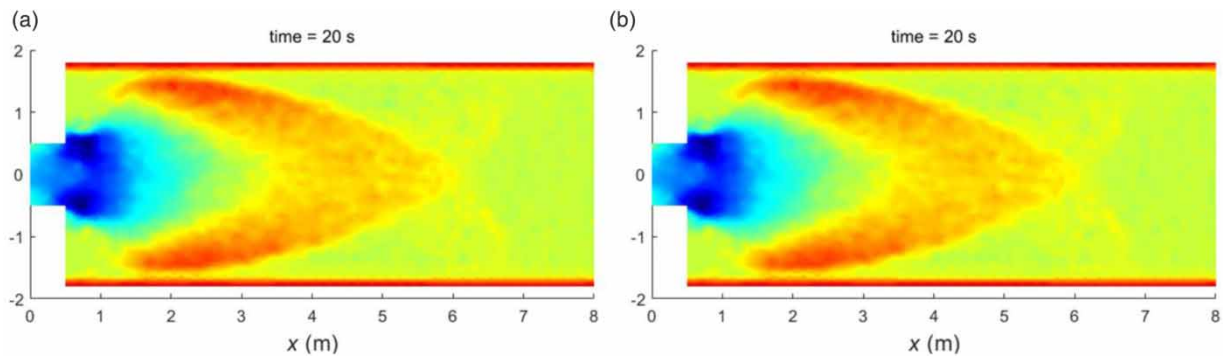


**Figure 7** | Final topography of the movable bed: (a) S1 and (b) S2.

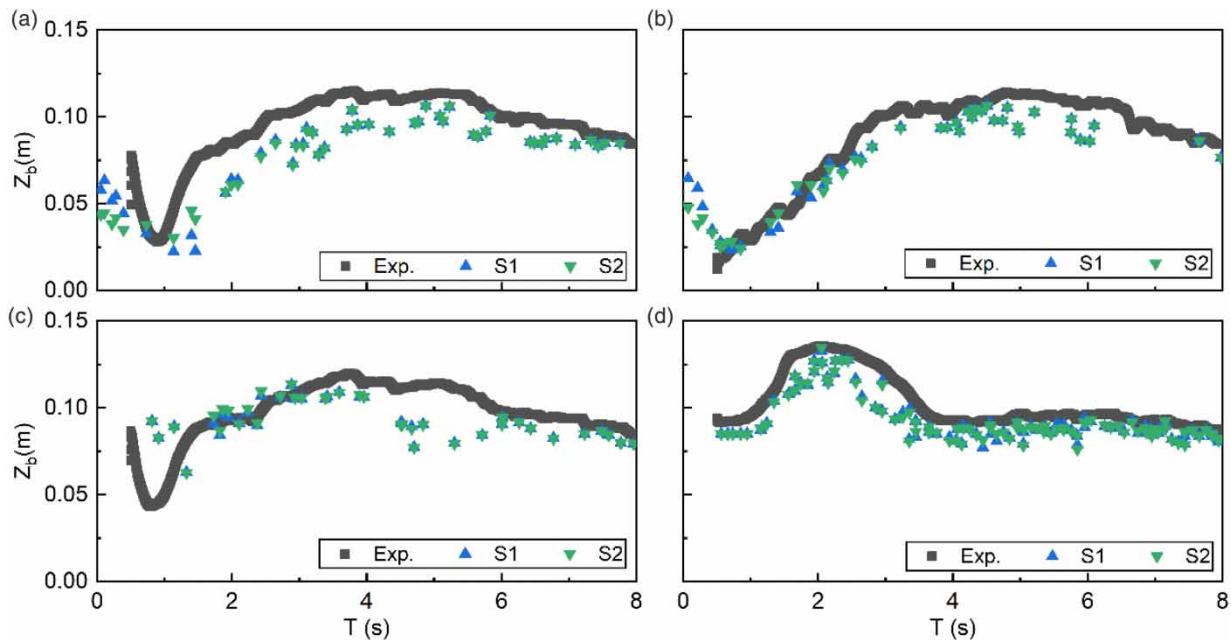


**Figure 8** | Bed elevation along different profiles: (a)  $y = 0$ ; (b)  $y = 0.2$  m; (c)  $y = 1$  m; (d)  $y = 1.45$  m.

Based on the analysis of Scenarios 1 and 2, it can be concluded that the proposed numerical model can well simulate the dam-break flow and the induced morphological change. In addition, since the results of Scenario 1 are identical to those of Scenario 2, the accuracy of the multi-layer numerical scheme applied to uniform sediments in the proposed numerical model can be confirmed.

#### 4.2.2. Numerical results of Scenario 3

Surface grain size distribution is analyzed by considering a multi-layer non-uniform sediment distribution in the movable bed (Scenario 3). Since the surface grain size distribution is determined by the erosion and deposition of the sediments and also by the distribution of each fraction of sediments within the layer, here the surface grain size distribution is analyzed along with the variation of the bed topography and also the percentage content distribution of the finest sediments ( $d_{50,i} = 0.5 \text{ mm}$ ). As shown in the first column of Figure 9, the movable bed sediments in the first layer are mainly eroded at  $t = 5 \text{ s}$ . The erosion and deposition of finer sediments in the first layer lead to a slight decrease of the percentage content of the finest sediments on the main channel, further leading to the increase of median diameter on the surface of the movable bed. Meanwhile, at the corner where the flume widens abruptly, the erosion depth is larger than the depth of the first layer, therefore the erosion of the following sediment layers leads to the increase of the surface median diameter at this area. With the increase of erosion depth and area ( $t = 10 \text{ s}$ ), the percentage content of finest sediments at the central downstream of the gate decreases from 0.57 to 0.33, indicating that the second layer is eroded in this area. In the meantime, the surface median diameter at the corner where flume suddenly widens decreases to zero due to the total erosion of the movable bed (i.e., the fixed bed has been reached), while around the corner, it greatly increase due to the deposition of the sediments in the third layer. At the final state of the simulation ( $t = 20 \text{ s}$ ), sediments in different layers are eroded at different locations on the movable bed. The surface median diameter mainly increases on the movable bed, and the increase rate decreases along the downstream of the flume. During the whole simulation, the increase of surface median diameters induced by the change of sediment layer is much larger than that induced by the variation of percentage content of each sediment fraction in each layer.

Furthermore, the sediment transport rates of each sediment fraction along the central profile of the flume ( $y = 0$ ) are analyzed, as shown in Figure 10. The sediment transport rate in each fraction first increases and then decreases along the flow direction. In addition, the transport rate in the sediment fraction with finer material is much larger, and it decreases with the increase of the median diameter in each sediment fraction. The difference of sediment transport rate in each fraction leads to

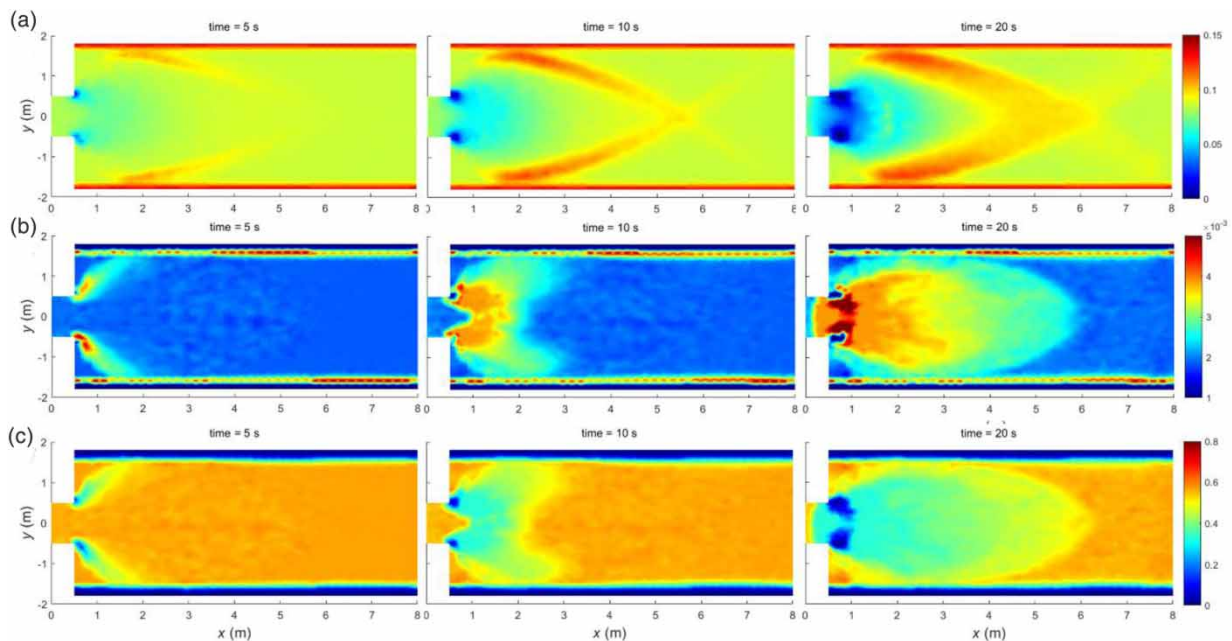


**Figure 9** | Variation of bed topography, surface median diameter and percentage content of coarsest sediments ( $d_{50,i} = 9 \text{ mm}$ ) at different times: (a) bed topography; (b) surface median diameter and (c) percentage content of fine sediments.

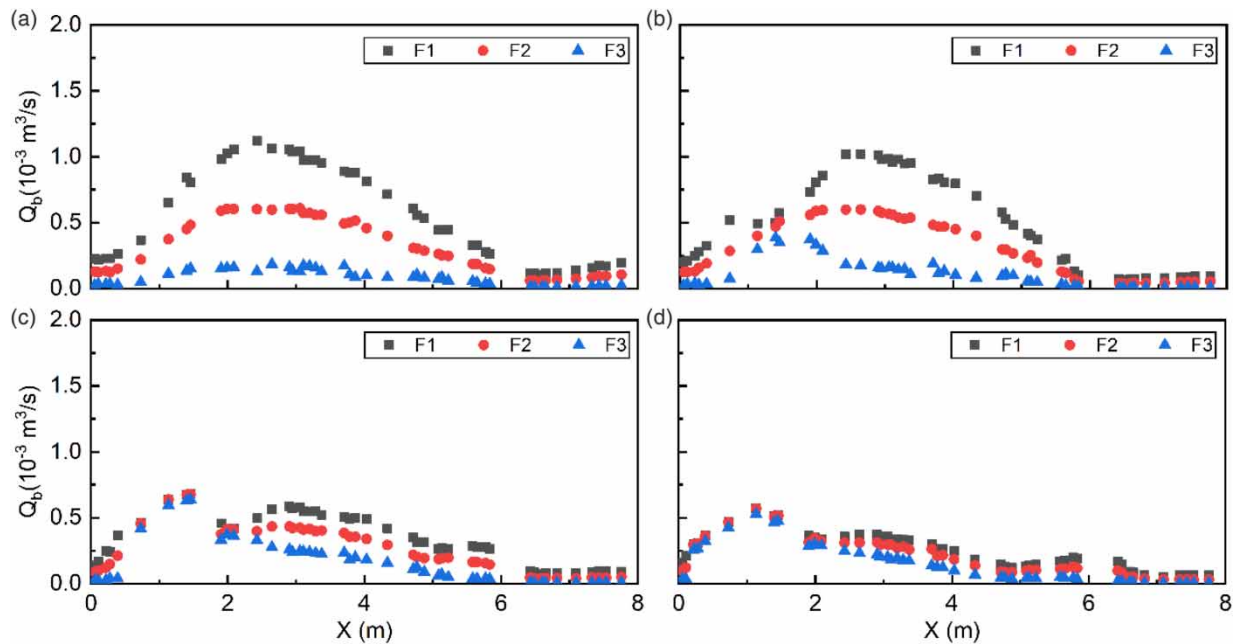


**Figure 10** | Sediment transport rate for each fraction  $F$  along profile  $y = 0$  at different time: (a)  $t = 5$  s; (b)  $t = 10$  s; (c)  $t = 15$  s; and (d)  $t = 20$  s.

the variation of surface median diameter. As the larger transport rate is observed in the finer sediment fraction, the percentage content of coarsest sediment fraction increases gradually, and finally leading to the increase of the median diameter on the surface of the movable bed. In addition, in Figures 10(b)–(d), a sudden decrease of sediment transport rate occurs at  $x = 2.0$  m, where the change of sediment layers and also the mixing of sediments with different diameters are observed, as shown in Figure 9. With these two phenomena, the percentage content of finest sediment decreases around the boundary of sediment layers, while it increases downstream of the boundary, further leading to the variation of sediment transport rate in each sediment fraction.

Based on the analysis of Scenario 3, it can be concluded that the proposed numerical model can well simulate the variation of surface median diameter relate to multi-layered and non-uniform sediment.

#### 4.3. Test 2: Dam failure with different non-uniform sediments

The second experimental test concerns a dam failure and the induced morphological changes that was conducted in Tongji University in a flume being 6 m long, 0.4 m wide (Yang *et al.* 2024). The set-up of the test is shown in Figure 11. The bottom of the flume consisted of 1.5 m fixed bed and 3.5 m movable bed with a depth of 0.08 m, and the modeled landslide dam was located at 2 m downstream of the flume. The dam was in a trapezoidal shape, with a height of 0.25 m, a top width of 0.12 m and a bottom width of 1.08 m. In addition, a diversion channel with a depth of 0.05 m and a width of 0.06 m was considered at the dam crest. The slope gradient of the flume was 0.016 and the inflow rate remained constant with  $0.001 \text{ m}^3/\text{s}$  during the test. For the material composition of dam and movable bed, three materials with sand content ( $d < 2 \text{ mm}$ ) ranging from 30% - 70% were considered, as shown in Figure 12. Based on these three materials, three numerical scenarios are designed as described in Table 2. The material in each scenario is divided into two fractions with same percentage content. A relationship between the median diameter of the sediment mixture and the median diameter and percentage content of each sediment fraction is determined based on a polynomial regression from the grain size distribution, as shown in Figure 12(b). The Manning coefficient and the active layer depth are calculated based on Equations (16) and (12), respectively.

Numerical simulations of this dam failure test were conducted on an unstructured triangular mesh with a spatial resolution of 0.02 m. In addition, in order to simulate the lateral slope collapse during the test, a bank failure module is applied in our simulation. The details of the bank failure module are given by Swartenbroekx *et al.* (2010) and the relevant parameters used in our simulation are shown in Table 3. In the simulation, the critical and residual angles are the same, the values below the water level is determined by the repose angle of the material, while the values above the water level are derived from

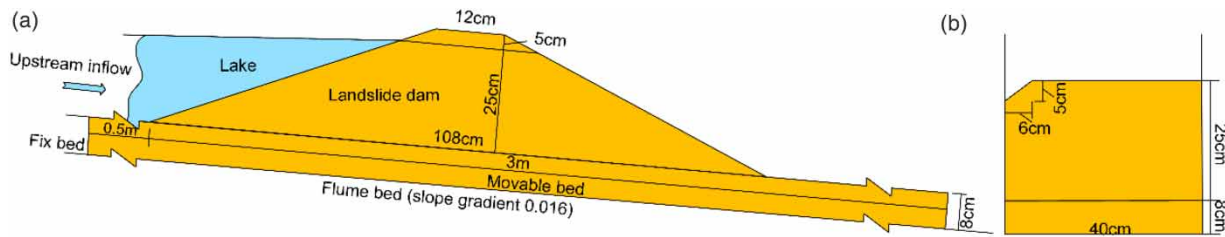


**Figure 11** | Set-up of the dam failure test: (a) Direction along the flume and (b) perpendicular to the flume.

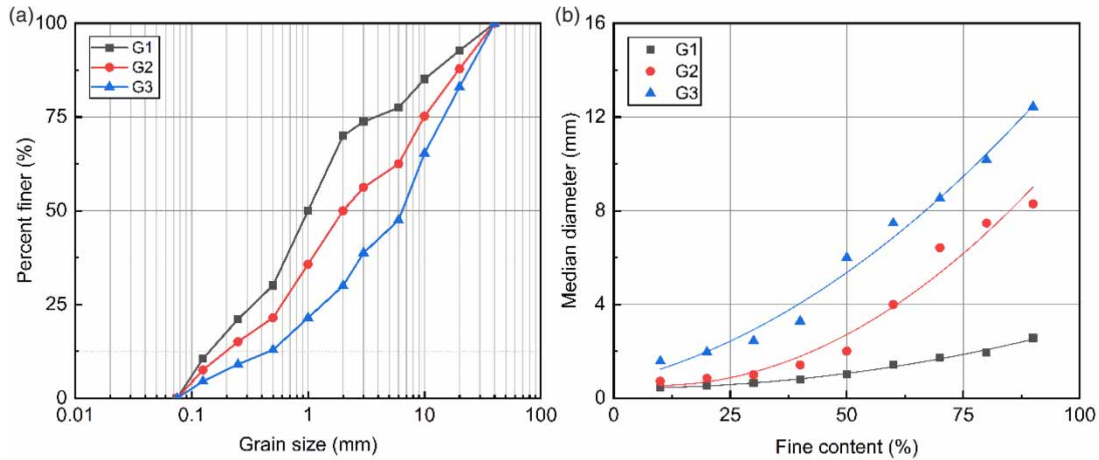


**Figure 12** | Experimental materials: (a) Grain size distribution and (b) relationship between fine content and median diameter.

**Table 2** | Numerical simulation scenarios and the parameters

| Scenario | Parameters |       |               |       |               |               | $n/sm^{-1/3}$ | $\varepsilon$ | $L_a/m$ |
|----------|------------|-------|---------------|-------|---------------|---------------|---------------|---------------|---------|
|          | $N$        | $f_i$ | $d_{50,i}/mm$ | $s$   | $n/sm^{-1/3}$ | $\varepsilon$ |               |               |         |
| S2-1     | 2          | 0.5   | 0.5           | 14.74 | 01.27         | 2.65          | 0.026         | 0.18          | 0.03    |
| S2-2     |            |       |               | 9.00  | 0.62          |               | 0.024         | 0.25          |         |
| S2-3     |            |       |               | 4.02  | 0.36          |               | 0.022         | 0.31          |         |

Note:  $N$  is the number of sediment fractions,  $f_i$  is the percentage content of each fraction,  $d_{50,i}$  is the percentage content of each fraction,  $s$  is the relative specific gravity,  $n$  is the Manning coefficient,  $\varepsilon$  is porosity and  $L_a$  is the active layer depth.

**Table 3** | Parameters of bank failure module

| Scenario | Critical angle(°) |             | Residual angle(°) |             |
|----------|-------------------|-------------|-------------------|-------------|
|          | Above water       | Under water | Above water       | Under water |
| S2-1     | 87                | 37.8        | 87                | 37.8        |
| S2-2     | 75                | 36.6        | 75                | 36.6        |
| S2-3     | 50                | 33.1        | 50                | 33.1        |

Note: The critical and residual angle are same and are determined by experimental data and the rest angle of the sediments.

experimental observation as steeper slopes can be achieved because of apparent cohesion due to moisture of the material. Based on the three scenarios in Table 2, the parameters change, dam failure process and also the downstream morphological changes are analyzed and compared with the experimental data.

#### 4.3.1. Parameters change and dam failure process

For each set of parameters, the variation of upstream water level and discharge during dam failure are analyzed, as shown in Figure 13. The variation of both the water level and discharge during dam failure are in good agreement with the experimental

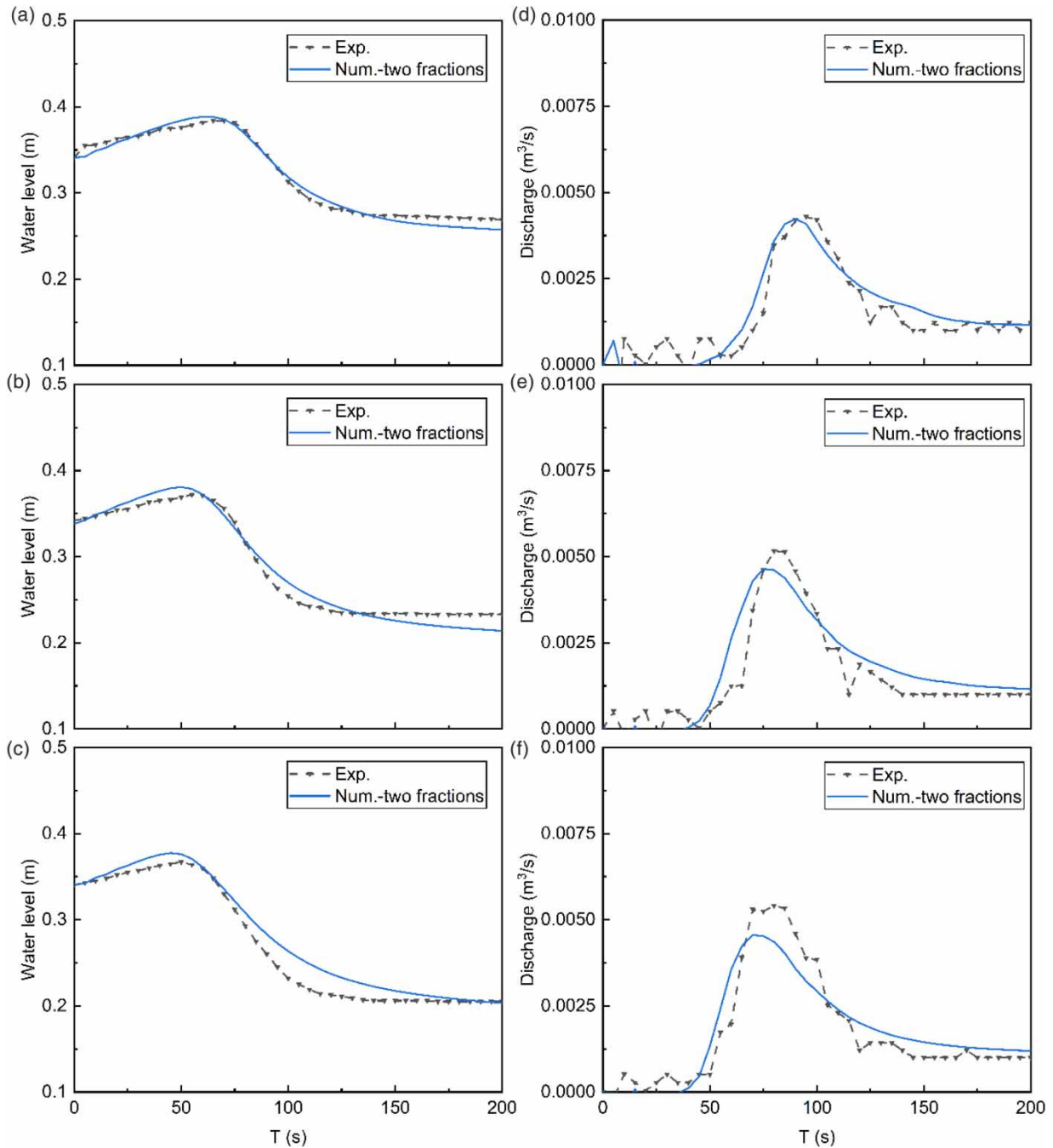


**Figure 13** | Upstream water level and discharge variation. (a), (b), and (c) are the upstream water level variation of S2-1, S2-2, and S2-3, respectively; (d), (e), and (f) are the discharge variation of S2-1, S2-2, and S2-3, respectively.

data, and the accuracy of the numerical results increases with the decrease of sand content in the materials. For instance, when the sand content is 30% (S2-1), the numerical model can well replicate both the decrease rate of upstream water level and the peak discharge of the experimental data (Figure 13(a) and (b)). However, with the sand content increasing to 70% (S2-3), the decrease rate of the upstream water level is much slower than that of the experimental data (Figure 13(e)), further leading to a smaller peak discharge as observed in the numerical results. As shown in Figure 13(f), the peak discharge of S2-3 obtained from numerical simulation is  $4.56 \times 10^{-3} \text{ m}^3/\text{s}$ , while the related value obtained from experimental data is  $5.41 \times 10^{-3} \text{ m}^3/\text{s}$ .

The final breach and the dam failure process are analyzed in Figure 14. The final breach size and shape for the different scenarios are in good agreement with the experimental data, especially the breach depth (Figures 14(a)–(c)). Note that the breach shape of S2-2 shows some difference with the experimental data. This is because the material above the water level is mainly in an unsaturated state, which cannot be properly reproduced in the numerical model. However, the influence of this difference is relatively small and can be ignored as the profile below the water level is well reproduced. For the dam failure process, in the same way as for the variation of upstream water level and discharge, the accuracy of the numerical results decreases with the increase of sand content in the material. As shown in Figure 14(f), at  $t = 80 \text{ s}$ , the dam profile obtained from the numerical results is much higher than that of the experimental results, and the slope of the erosion surface is also much smaller. However, this discrepancy has little influence on the final profile after dam failure.

The difference between the numerical and experimental results when the finer material is considered can be attributed to the dam failure characteristics. The erosion of the dam mainly occurs as surface erosion when the coarser material is considered. However, for the dam with the finer material, the erosion of the dam may be transformed from surface erosion to backward erosion with mass failures of blocks of material as it happens for cohesive material, further leading to a steeper erosion surface during dam failure. This steeper erosion surface cannot be properly simulated with the present model, thus, leading to the difference between the numerical and experimental results.

#### 4.3.2. Morphological changes

The final topographies of the movable bed related to the different scenarios are shown in Figure 15. The change of the movable bed is mainly induced by the deposition of dam material, and the deposition areas in the different scenarios are well reproduced. However, in the same way as the parameters change and dam failure process illustrated before, the numerical final topography of S2-3 shows some difference with the experimental data, where the erosion channel observed in the middle of the movable bed is not clearly reproduced in the numerical results. This is because the erosion and deposition of dam material is determined by the dam failure process, that is less well reproduced for finer material (i.e., higher sand content).

Furthermore, two profiles located at  $y = 0.1 \text{ m}$  and  $x = 3.5 \text{ m}$  (Figure 15) are selected to analyze the bed elevation. The comparison between numerical and experimental results of different scenarios is shown in Figure 16. In the different scenarios, the numerical bed elevation along the profiles is always little bit lower than that of the experimental results, this may be attributed to the fact that only two sediment fractions are considered in the simulation. Therefore, the influence of the largest sediments (20–40 mm) may be ignored, leading to a longer transport distance and thus less deposition. However, the difference between numerical and experimental bed elevation is relatively small, and the present model is considered able to simulate the bed variation induced by dam failure. Still, the numerical and experimental bed elevations on Profile 2 (Figures 16(b), (d) and (f)) show larger differences when the finer material is considered, as was already observed in the bed topography simulation: the difference in dam failure simulation leads to a difference in the creation of the erosion channel in the middle part of the movable bed, further leading to the observed difference in bed elevation along Profile 2.

## 5. DISCUSSION

### 5.1. Influence of sediment fractions and active layer depth on numerical results

In this section, we consider S2-2 as the reference case, and four additional numerical scenarios are considered to analyze the influence of sediment fractions and active layer depth on the numerical results. The four additional numerical scenarios based on S2-2 are described in Table 4. In SA-1 and SA-2, one and four sediment fractions are considered respectively and the others parameters remain as those in S2-2. In addition, the four sediment fractions in SA-2 are obtained by sub-dividing the two fractions considered in S2-2 around the median diameter of each fraction. In SA-3 and SA-4, only the active layer

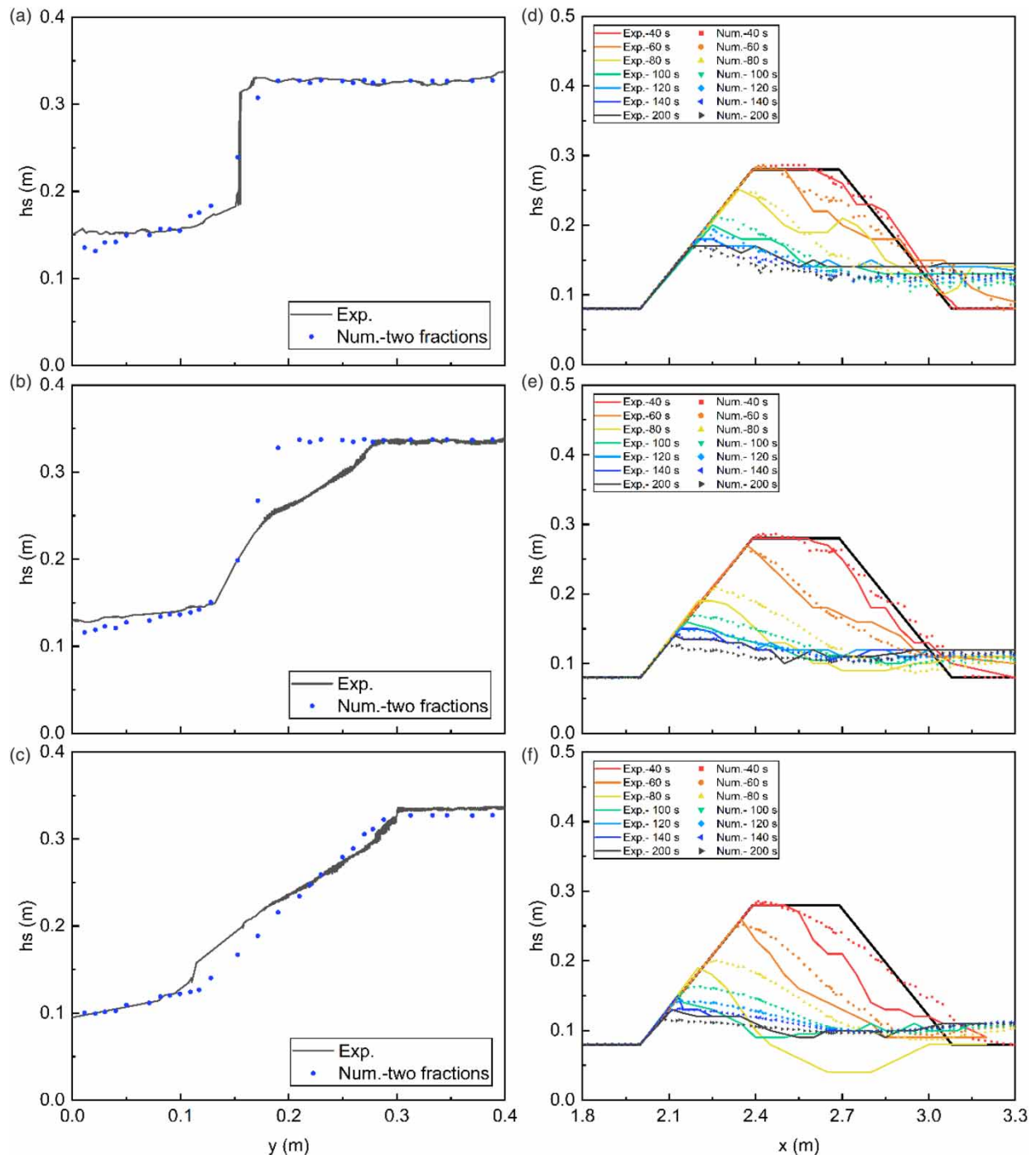


**Figure 14** | The final breach and the longitudinal dam profile evolution during dam failure. (a), (b) and (c) are the final breach of S2-1, S2-2 and S2-3, respectively; (d), (e) and (f) are the longitudinal dam profile evolution of S2-1, S2-2 and S2-3, respectively.

depth is changed from 0.03 m to 0.02 and 0.04 m. The relationship between the median diameter and the fine content in SA-2 is the same as that in Figure 12(b), but the fine content here is the sum of the last two fractions shown in Table 4.

Figures 17(a) and (b) show the influence of sediment fractions and active layer depth on the variation of the upstream water level during dam failure. For the influence of sediment fractions, the same water levels are observed in Scenarios S2-2, SA-2 with two and four sediment fractions, however, the numerical results of Scenario SA-1 with one sediment fraction shows obvious differences. In this case, the maximum water level is smaller and it decreases earlier than that of the others two

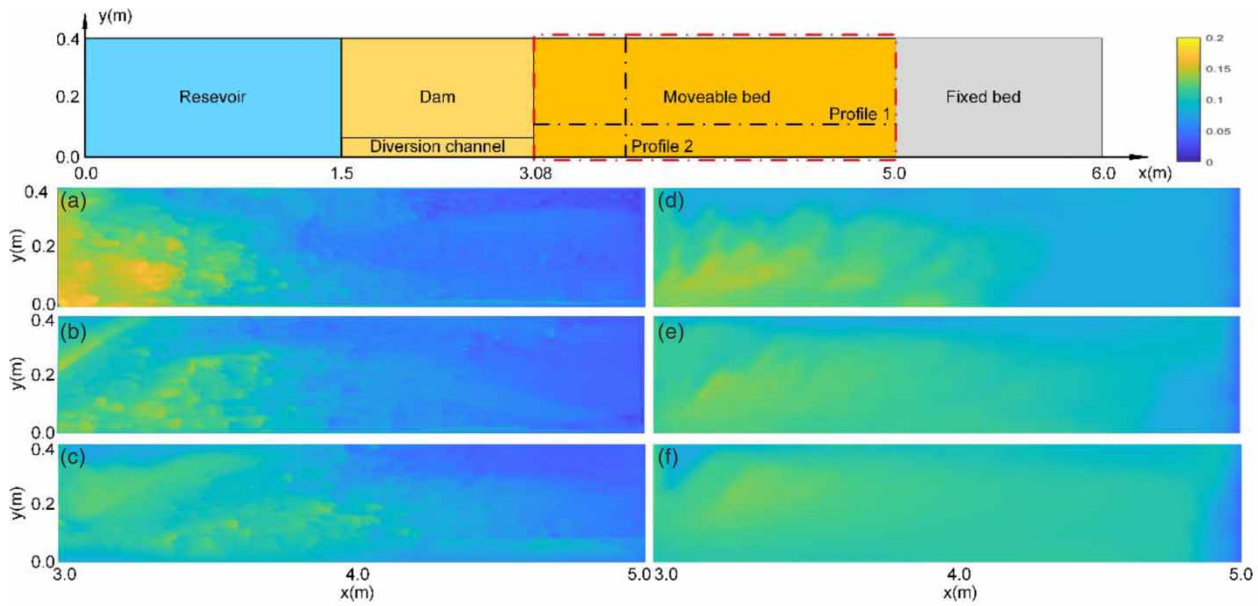


**Figure 15** | Comparison between numerical and experimental results related to the final topography of the movable bed: (a), (b) and (c) are experimental results of S2-1, S2-2, and S2-3, respectively; (d), (e) and (f) are numerical results of S2-1, S2-2, and S2-3, respectively.

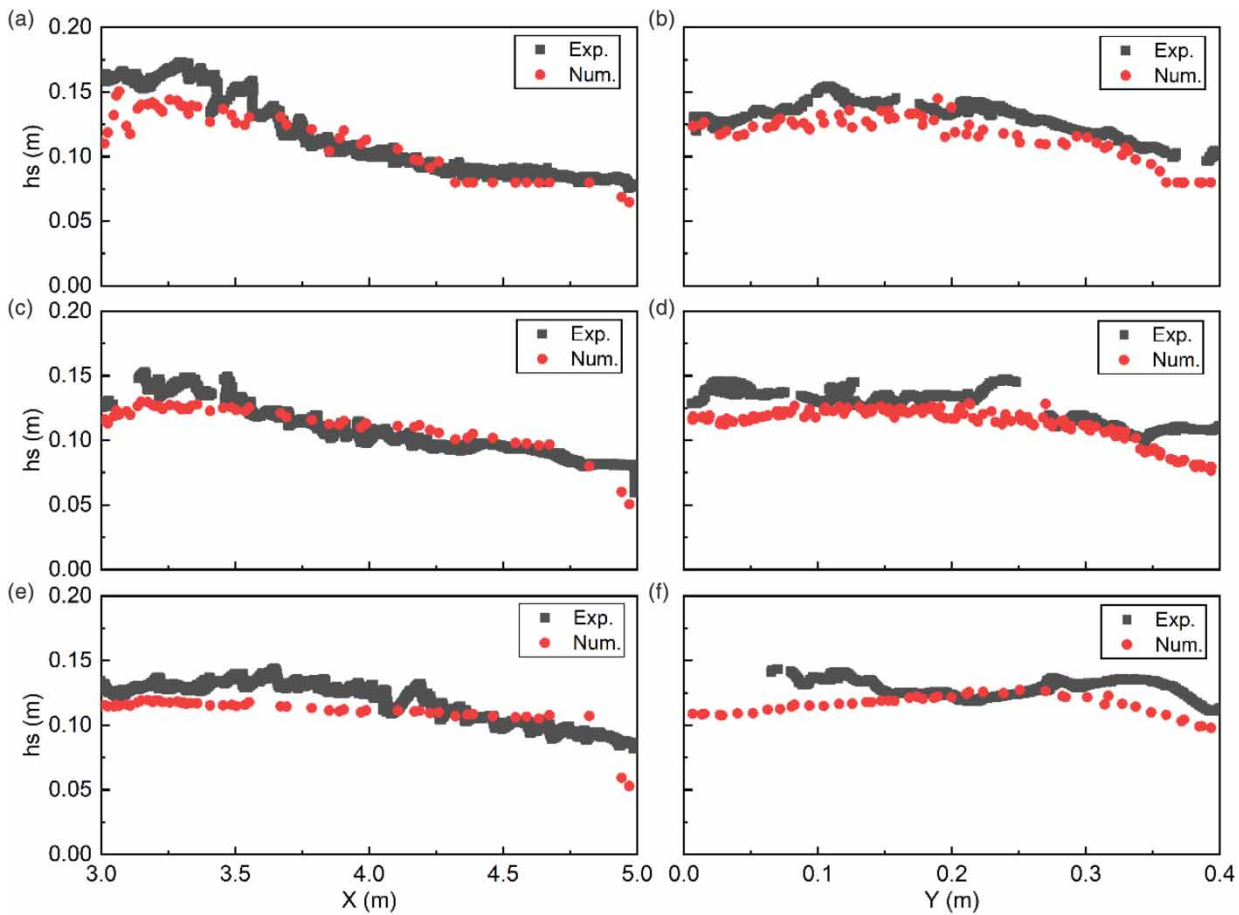
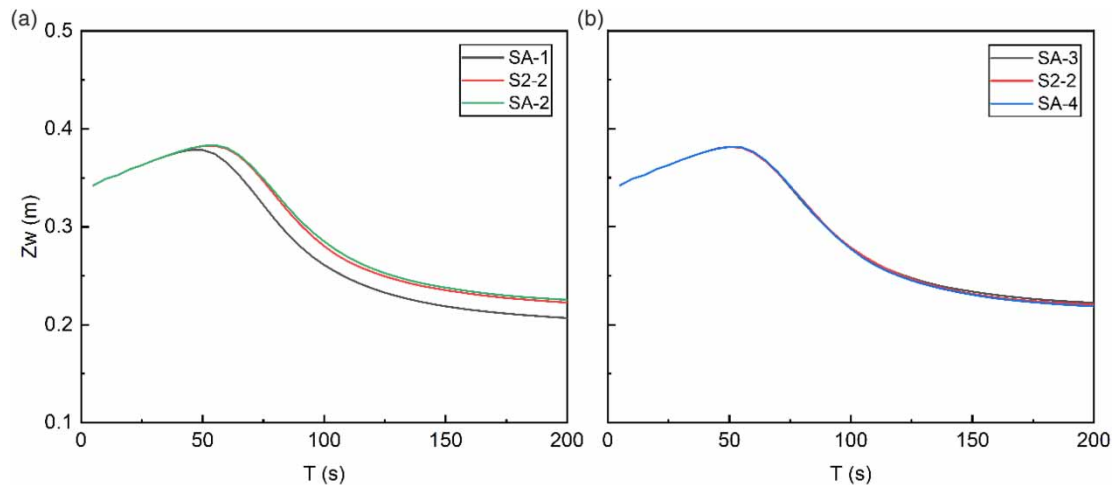


**Figure 16** | Comparison between numerical and experimental results related to the bed elevation along Profiles 1 and 2: (a) and (b) S2-1; (c) and (d) S2-2; (e) and (f) S2-3.

**Table 4** | Numerical scenarios with different sediment fractions and active layer depth

| scenario | $N$ | $f_i$ | $d_{50,i}/\text{mm}$ |      |      |       |      |      |      |
|----------|-----|-------|----------------------|------|------|-------|------|------|------|
| SA-1     | 1   | 1     | 2.00                 |      |      |       |      |      |      |
| SA-2     | 4   | 0.25  | 0.25                 | 0.28 | 0.22 | 19.80 | 6.03 | 1.00 | 0.18 |
| SA-3     | 2   | 0.5   | 0.5                  |      | 9.00 |       | 0.62 |      |      |
| SA-4     | 2   |       |                      |      |      |       |      |      |      |

Note:  $N$  is the number of sediment fractions,  $f_i$  and  $d_{50,i}$  are the percentage content and median diameter of each sediment fraction, respectively.

**Figure 17** | Variation of upstream water level with different sediment fractions and active layer depth: (a) sediment fractions and (b) active layer depth.

scenarios, while the decrease rate of the water level remains unchanged (Figure 17(a)). This phenomenon indicates that the number of sediment fractions mainly influence the initiation of erosion of the dam when the discharge is relatively small. Indeed, in this case, the difference in grain size has a significant influence on the dam erosion process, as smaller fraction can be eroded already for smaller discharges. However, when the discharge is quite large, the difference in erosion rate for sediments with different size is small and can be ignored, therefore, the similar decrease rate of water level is observed. When it comes to the influence of active layer depth, as shown in Figure 17(b), the variation of upstream water level in the different scenarios is the same, indicating that the active layer depth has no obvious influence on the dam failure process.

Similar observations can be made for the numerical results of bed elevation. Here we take the bed elevation along Profile 1 (the location is shown in Figure 15) as an example, and the influence of sediment fractions and active layer depth on the bed elevation is shown in Figure 18. Compared to the results of Scenarios S2-2 and SA-2 with two and four sediment fractions respectively, the bed elevation of Scenario SA-1 with one sediment fraction is lower than that of the others two scenarios. As in the previous analysis, the numerical results of bed elevation are slightly lower than the experimental results; therefore, better simulation results can be observed when considering multiple sediment fractions. However, the numerical results of Scenarios S2-2 and SA-2 with two and four sediment fractions show no difference with each other. For the influence of active layer depth, in the same way as for the water level change, no difference is observed between the different scenarios.

Since the non-uniform sediments and active layer models are considered, the model is able to describe the surface grain size distribution of the movable bed after dam failure. As shown in Figures 19(a) and (b), the distribution of sediment on the bed surface is well reproduced. Specifically, the zones where the median diameter increases and decreases obtained from numerical results are in good agreement with the experimental results. Furthermore, two points which are located at the zones where the median diameter increases and decreases respectively are selected to analyze the variation of median diameter and the influence of sediment fractions and active layer depth. In the zone where the median diameter increases (around P1 in Figure 19), the median diameter first increases and then slightly decreases until the end of the simulation. In addition, the increase rate of the median diameter increases with the number of sediment fractions while it decreases with the increase

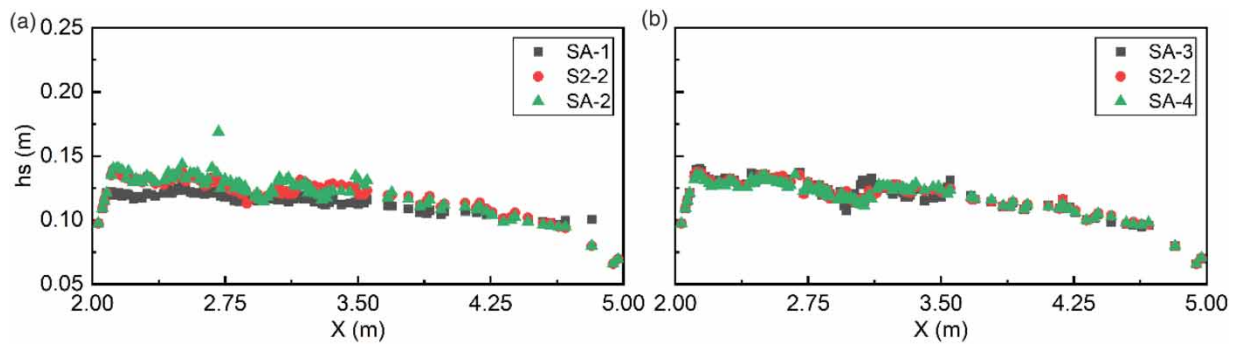


**Figure 18** | Variation of bed elevation along Profile 1 with different sediment fractions and active layer depth: (a) sediment fractions and (b) active layer depth.

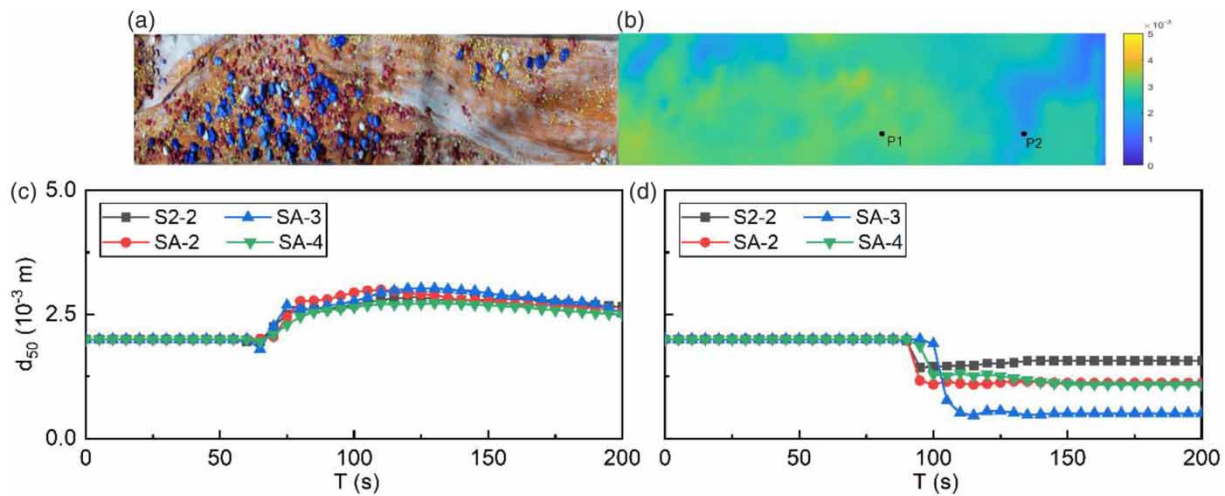


**Figure 19** | Final surface grain size distribution and the influence of sediment fractions and active layer depth on the variation of median diameter (take S2-2 as an example): (a) and (b) numerical and experimental final surface grain distribution; (c) and (d) variation of median diameter.

of active layer depth. However, the variation is relatively small (Figure 19(a)). In the zone where the median diameter decreases (around P2 in Figure 19), the median diameter first decreases and then remains constant. The sediment fractions and the active layer depth have a significant influence on the decrease of the median diameter. Specifically, a smaller final median diameter is observed with the increase of sediment fractions and decrease of active layer depth (Figure 19(b)).

## 5.2. Limitations

The numerical results in this paper are in good agreement with the experimental data, but there are still some limitations. First, the accuracy in simulating dam failure with fine sediments by using the present numerical model could be improved. Indeed, the model cannot well capture the steep slope induced by backward erosion and mass failures when the fine content is relatively large in the material composition. This would require further assumptions and the implementation of geomechanical failure mechanisms adapted to cohesive sediments in the present numerical model. Secondly, the model is unable to properly consider the remobilization and redeposition of the sediments. Indeed, in the numerical scheme related to the change of layer, only the information related to the sediments in the active layer is recorded, which is sufficient for dam failure simulation. However, when the sediments are remobilized or redeposited, this may cause inaccuracies in the simulation results related to morphological changes and surface grain size distribution. In addition, the deposition of sediments with different diameters during the simulation should be further optimized. Third, up to now, only two experimental tests are simulated to validate the numerical model. Further applications should be considered in the future, especially related to real dam failure cases.

## 6. CONCLUSION

In this paper, a two-dimensional numerical model considering multi-layered and non-uniform sediments is proposed to simulate landslide dam failures and the induced morphological changes. The model solves the shallow water equations, the Exner equation and Hirano's active layer equation based on a weakly coupled finite volume scheme. Two experimental tests are used to testify the applicability and accuracy of the present numerical model.

The first test considers a dam-break flow case with movable bed composed of uniform sediments. Three numerical scenarios were tested, with one-layer uniform sediments, multi-layer uniform sediments and multi-layer non-uniform sediments. The model can well simulate the water level and morphological changes related to uniform sediments, and the numerical results are in good agreement with the experimental data. In addition, the numerical model is able to simulate the surface coarsening and fining phenomena related to multi-layer and non-uniform sediments. The variation of the median diameter on the surface of the movable bed can be attributed to the change of sediment layers and the percentage content of each sediment fraction in each layer, among which, the change of sediment layers shows more obvious influence on surface grain size distribution.

The second series of tests aims at replicating a landslide dam failure with a wide grain size distribution and the induced morphological changes, and three scenarios with different sand content in the materials are analyzed. The numerical results of parameters change, dam failure process and the morphological changes are in good agreement with those of the experimental data when the material with smaller sand content are considered. With the increase of sand content in the material, the numerical model tends to inaccurately simulate the dam failure process, further leading to an underestimation of the peak discharge and of the decrease rate of the upstream water level, as well as to the inaccurate results related to the final bed elevation.

The influence of sediment fractions and active layer depth on numerical results is further analyzed based on the landslide dam failure test. Compared to the scenario with one sediment fraction, scenarios with two and four sediment fractions can better simulate the dam failure and the induced morphological changes. However, the difference in scenarios with two and four sediment fractions are relatively small. The active layer depth shows no influence on the dam failure and the morphological changes. The numerical surface grain size distribution is in good agreement with the experimental result. In addition, a larger median diameter in the zones where the median diameter increases and a smaller one in the zone where the median diameter decreases are observed with the increase of sediment fractions and the decrease of the active layer depth.

Future research work will focus on improving the simulations for finer sediments, the erosion and deposition of sediments and the real dam failure cases.

## ACKNOWLEDGEMENTS

The authors acknowledge the support from the Natural Science Foundation of China (No. 41731283 and No. 42307196) and the China Postdoctoral Science Foundation (2023M733029).

## DATA AVAILABILITY STATEMENT

Data cannot be made publicly available; readers should contact the corresponding author for details.

## CONFLICT OF INTEREST

The authors declare there is no conflict.

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