Effects of initial stage of dam-break flows on sediment transport

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Abstract. Experimental and numerical studies of dam-break flows over sediment bed under dry and wet downstream conditions are investigated and their effects on sediment transport and bed change on flow are illustrated. Dam-break waves are generated by suddenly lifting a gate inside the flume for three different upstream reservoir heads. The flow characteristics are detected by employing simple and economical measuring technique. The numerical model solves the two-dimensional Reynolds-Averaged Navier–Stokes (RANS) equations with $k-\epsilon$ turbulence closure using the explicit finite volume method on adaptive, non-staggered grid. The model is validated with laboratory data and is extended for simulating non-equilibrium sediment transport and bed evolution process. The volume of fluid technique is used to track the evolution of the free surface, satisfying the advection equation. The comparison study reveals that the current model is capable of defining the dam-break flow and improves the accuracy of determining morphological changes at the initial stages of the dam-break flow. A good agreement between the model solutions and the experimental data is observed.

Keywords. Dam-break flow; flume experiment; RANS equation; finite volume method; sediment transport.

1. Introduction

The dam-break flow has been a topic of significant research interest due to both realistic and scholastic interests for several decades. The propagation of dam-break waves generated by the breaching of the dam can have a catastrophic effect on the downstream, because of the large volume of water released instantaneously from the reservoir. The initial dam-break flows are normally wave breaking and turbulence dominates induce active sediment transport and result in significant changes in river morphology. Therefore, the interactions among flow, sediment transport, and river morphology have raised a strong incentive to study dam-break flow over mobile beds during the initial stage.

In the past, extensive efforts have been made to understand and simulate the dam-break flow problems over non-mobile or fixed beds by conducting laboratory/field study, theoretical analysis and developing numerical models. In contrast with the non-mobile or fixed beds, the simulation of dam-break flow over mobile beds is really challenging owing to sediment particles liable to be entrained in motion and has appeared in recent years.

A limited number of experimental modeling of dam-break flow was involved in measurement of the velocity

field and measurement of the free surface variation by various techniques [1–6]. Stansby *et al* [1] conducted a series of experiments on the dam-break flows under dry and wet bed conditions at downstream. They confirmed that the wave breaking and turbulence lead the flow field during the initial stage of dam-break waves and hence the assumptions of the long wave and hydrostatic pressure used in the conventional theoretical models are not valid. Lauber and Hager [2] studied both experimentally and analytically the fronts of the positive and negative dam-break waves over dry bed condition in a horizontal rectangular channel. However, the obtained results were not in good agreement with the experimental observation during the initial stage of dam-break. Janosi *et al* [3] conducted experiments on the dam break driven waves with dry- and wet-bed conditions at downstream of the dam, and studied drag reduction due to the addition of polymer in their experiments. A series of experiments were conducted over the horizontal bed by [6–9] and obtained velocity profiles of 2D dam-break flows using a particle tracking velocimetry method. However, the near-bed velocity profiles were not well-established owing to detection of seeding in images. Flume experiments are normally constrained by the comparatively small spatial scales in laboratories and may not be sufficient for fully unravelling the complex mechanisms of dam-break flows. Thus, numerical study is an alternative for

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enhancing the understanding of mobile bed dam-break flows.

Numerous effort have been executed earlier in the development of 1D and 2D depth-averaged models to simulate dam-break flows [4, 5, 10–21]. Different approaches have been used to accommodate the wetting and drying process at the wave front with varied levels of success. However, the assumptions used to derive the governing equations such as hydrostatic pressure distribution, insignificant vertical acceleration, and free surface curvature, are inappropriate in the initial stages of dam-break wave front. In particular, flow conditions close to wave front just after the dam fail are strongly affected by vertical accelerations which are not considered in 1D and 2D Shallow Water Equation (SWE) models.

Dam break flows involve mixed flows with discontinuities usually propagate along rivers and floodplains, where the processes of fluid flow, sediment transport and bed evolutions are closely related [22]. However, the majority of existing 2D models used to simulate dam-break flows are applicable to fixed beds. Capart *et al* [22] pointed out that in some extreme cases, particularly those caused by a dike or dam failures, the volume of entrained material could reach the same order of magnitude as the volume of water initially released from the failed dike or dam. Therefore, research on sediment transport requires a unique method to understand the complex phenomena between the interface of different fluids [16, 22–24].

Recently, two approaches are commonly used for modeling the morphodynamic processes, i.e., uncoupled and coupled solutions [25]. In the beginning, numerical models for simulating dam-break flows over mobile beds, uncoupled solutions were adopted, without accounting for the effects of the sediment transport and the bed deformation on the movement of the flow [26–28]. One of the problems in movable-bed modeling is that under the dam-break flow condition the sediment concentration is so high and the bed varies so rapidly that their effects on the flow cannot be ignored. To predict suitably the consequences of a dam failure in a compound topography, the interface between the flow and the bed morphology must be considered in modeling. At present, several models for simulating dam-break flows over mobile beds based on the coupled solutions were developed because the rate of bed evolution being more comparable to the rate of water depth variation [15, 18, 29]. Capart and Young [30], and Spinewine and Zech [31] used two-layer 1D models to simulate dam-break flows over mobile beds. These models were applicable to morphological changes caused predominately by the non-equilibrium transport of bed load. But, the applicability of the models was limited because of the assumption of a constant sediment concentration in the lower layer. Cao *et al* [15] developed 1D model for the dam-break flow over movable beds considering the non-equilibrium sediment transport and the effects of sediment on the flow. Their approaches are relatively reasonable, and provided valuable conclusions for

dam-break fluvial processes. However, the model exaggerates the separate bore because of the unreasonable prediction of sediment entrainment. Wu and Wang [17] proposed a 1D model to simulate dam-break flows over mobile beds using the coupled approach, and applied the model to investigate the mechanisms of morphodynamic processes caused by dam-break flows. A more difficult method was used in this 1D model [17] to calculate the rates of sediment deposition and entrainment, which could account simultaneously for the process of bed evolution caused by the suspended and bed loads. Emmett and Moodie [32, 33] developed a shallow water model to investigate the transport of dilute sediment under dam-break flows over both horizontal and sloping dry beds, taking into account basal friction as well as the effects of particle concentrations on the flow dynamics. They concluded that the presence of a dilute suspension did not have a significant effect on the height or velocity profiles of the flows. However, the presence of drag has significantly altered the shape of the depth profile in the immediate vicinity of the leading edge as well as the velocity structure of the flow over a bed. Simpson and Castellort [34] extended an existing 1D coupled model of Cao *et al* [15] to a 2D model for the free surface flow, sediment transport and morphological evolution. The model used a Godunov-type method with a first-order approximate Riemann solver, and was verified by comparing the computed results with the documented solutions. Cao [35] stated that, the first-order numerical scheme in solving the governing equations may have limitations in modeling water levels and sediment concentrations with gradient discontinuities. Therefore, it is necessary to develop a morphodynamic model for simulating these complex flows over mobile beds, one must rely on vertical 2D or 3D models which solve the Reynolds-Averaged Navier–Stokes (RANS) equations. The numerical model was implemented and the RANS equations were solved using the finite-difference method to compute the initial stage of dam-break waves, in which the volume of fluid was employed to track the free surface [36, 37]. It was seen that the model application for the study of sediment transport processes was inadequate. Hsu *et al* [38] presented an experimental and numerical investigation of dam-break driven flow over a horizontal smooth bed for different downstream-to-upstream water depth ratio. The numerical model COBRAS [39, 40] based on the 2D vertical RANS equations, with a $k-\varepsilon$ turbulence closure was adopted to simulate the complex hydrodynamics induced by dam-breaking. It is shown that the numerical model suitably predicts the measured data, as well as the impingement location of the forward breaking jet. However, it is found that the larger discrepancies correspond to the smaller water depth ratio.

In the present work, a 2D numerical model of dam-break flow has been developed, which solves the RANS equations explicitly with $k-\varepsilon$ turbulence closure using a finite-volume method. The numerical model is extended to examine the mechanisms of sediment transport and pattern of sediment

erosion under varied dam-break flow conditions. In this work, the model is coupled with the non-equilibrium sediment transport and bed evolution modules to simulate dam-break flows over sediment beds.

The effects of sediment concentration on sediment settling and entrainment are considered in determining the sediment settling velocity and transport capacity. Further, models for simulating both suspended and bed load are implemented into the code. In order to underpin numerical modeling approaches, experiments have been conducted in a laboratory flume over sediment bed under dry- and wet-bed conditions. In this work, a more general model for the simulation of fluvial process under dam-break flow has been established and compared with experimental data. To the authors' knowledge, findings obtained from dam-break experiment and comparison with numerical simulation have not been reported in a systematic manner before. This work will certainly promote new significance and understanding of the dam break flow problem. In the following section, the governing equations, numerical methods, details of the test facility and procedures used in the experiments are described. Discussion of the results of especially dam-break flow is illustrated.

2. Experimental descriptions

The experiments are conducted in a straight rectangular plexiglas flume of 18 m long, 0.4 m wide and 0.6 m high over a horizontal sediment bed with a surface roughness of $k_s \sim 0.45$ mm. Here, k_s is set equal to the median diameter d_{50} of bed material because there is only sand-grain roughness on the bed. Schematic diagram of the experimental set-up are shown in figure 1. The dam site is located 8 m from the channel entrance and is made of a thin metal plate of 6 mm thickness, which could glide in small plastic channels mounted on a section around the flume bed and sides. A rubber seal is used to avoid water leakage. Actually, 1–3 mm

thin film of water depth due to leakage on the bed downstream of dam is termed as the dry bed. The gate was abruptly lifted up with a wire rope attached to the top of the plate and was guided by a pulley. A weight of 8.5 kg was attached to the other end of the cable and about 1 m above the floor. All tests were carried out using a weight of 8.5 kg and a drop height of 0.45 m. The initiating time, $t = 0$ is considered to be the instant at which the gate begins to move. According to Lauber and Hager [2], the gate opening can be considered as instantaneous, if the lift time $T_L \leq \sqrt{2h_u/g}$. Here, h_u is the initial upstream reservoir depth and g is the acceleration due to gravity. The time of the gate lift was found to be 8 s for $h_u = 0.25$ m. The experiments were made at three scales with upstream depths (h_u) of 0.25 m, 0.35 m and 0.45 m and the flood wave propagated over dry and wet beds. The rigid bed of the flume is covered with 100 mm thick horizontal layer of uniform, fully saturated and uncompacted sand with the median diameter of sediment is about 0.45 mm, specific density is 2.63 and initial porosity is 0.40 (porosity after deposition is 0.36). The downstream condition is a free outflow over the sediment bed and is maintained by a vertical plate at the initial elevation specifying the same height as that of the bed layer. Table 1 summarizes all experimental conditions. Flow velocity on the downstream of gate is measured by using Ultrasonic Velocity Profilers UVPs (Met-Flow, Switzerland). The UVP setting for all the measurements are shown in table 2. Ultrasound measuring devices G1, G2, G3, and G4 are placed on the center line of the channel at 1.15, 1.85, 2.15 and 2.5 m downstream from the gate, respectively, to record the time series of water surface. In the model, the horizontal grid spacing near the gate is 5 cm and increases gradually in downstream and upstream directions. The vertical grid consists of 16 layers with the grid spacing equal to about 2.5 cm. The experiment lasted for a period of 45 s for each test. At the end of the experiment the bed elevation was measured using a rail mounted point gauge with ± 0.1 mm accuracy over the entire flume width.

Figure 1. (a) Schematic diagram of the experimental set-up; (b) section showing pulley system ($h_d = 0$ for dry bed condition).

Table 1. Test conditions for present dam-break flow cases.

Test	h_u (m)	h_d (m)	$r = \frac{h_d}{h_u}$	Test	h_u (m)	h_d (m)	$r = \frac{h_d}{h_u}$	Test	h_u (m)	h_d (m)	$r = \frac{h_d}{h_u}$
1	0.25	0	0	7	0.35	0	0	13	0.45	0	0
2		0.013	0.05	8		0.018	0.05	14		0.023	0.05
3		0.025	0.1	9		0.035	0.1	15		0.045	0.1
4		0.05	0.2	10		0.07	0.2	16		0.09	0.2
5		0.075	0.3	11		0.105	0.3	17		0.135	0.3
6		0.1	0.4	12		0.14	0.4	18		0.18	0.4

Table 2. UVP inputs for the measurement locations.

Inputs	Upstream 1-2	Downstream 1-2
Sampling period (ms)	18	16
Sound speed	1480	1480
Max. and min. velocity (m/s)	0.768	3.26
Frequency (MHz)	4	2
Accuracy	± 1	± 3

3. Numerical model

To numerically reproduce the basic patterns of flow and sediment transport, the free and open source code TELEMAC-MASCARET 2D, and open-source CFD code OpenFOAM models are selected for this study. OpenFOAM is an open-source suite of C++ language designed for the development of numerical solvers for continuum mechanics problems, including CFD applications. This model uses the finite volume method to solve the RANS equations. The dam-break experiments were simulated using the OpenFOAM solver, InterFoam. This solver models an incompressible and immiscible two-phase (water–air) system, using the Volume-of-Fluid (VOF) method to track the free surface on the air–water interface. Here, to represent essential configuration of the sediment bed, the 2D model of the open source Telemac-Mascaret Modeling System is used. At each time step, the 2D models of the open source Telemac-Mascaret Modeling System comprises two steps. The first step calculates the flow variables in the channel, which is subsequently internally coupled with the sediment transport and bed evolution module (Sisyph module). Then, a sediment transport capacity formula is used to compute the bed load rate, and bed evolution is determined by solving the 2D sediment continuity equation. Telemac-Mascaret 2D is based on the solution of the Reynolds Averaged Navier-Stokes (RANS) equations with a non-hydrostatic pressure distribution.

3.1 Model equations

The model equations used are based on 2D RANS equation with k - ε turbulence closure providing logarithmic velocity distribution in the boundary layer, and are solved by using

the explicit finite volume scheme on adaptive, non-staggered grid. The volume of fluid method with the compressive interface capturing scheme (CICS) is employed to track the evolution of the free surface, satisfying the advection equation. In the present study, the influence of sediment concentration (c) on the settling velocity is considered. The governing 2D continuity and RANS equations for an incompressible fluid flow, which can be written in Cartesian coordinates as follows:

$$\frac{\partial}{\partial x_i}(u_i) = 0 \quad (1)$$

$$\frac{\partial u_i}{\partial t} + \left(u_j \frac{\partial u_i}{\partial x_j}\right) = -\frac{1}{\rho} \frac{\partial p}{\partial x_i} + g_i + \frac{1}{\rho} \frac{\partial \tau_{ij}}{\partial x_j} - \frac{\partial (u'_i u'_j)}{\partial x_j} \quad (2)$$

where $i, j = 1, 2$ for two-dimensional flow; u_i = the ensemble averaged flow velocity; p = the ensemble-averaged fluid pressure; τ_{ij} = viscous stress; ρ = fluid density; u' instantaneous fluctuation velocity; and g_i = gravitational acceleration component. The density is excluded from the temporal and convective terms of the momentum equation because the density is continuous and constant in the domain containing water. Although the diffusion term in the momentum equation may be neglected due to the fact that the convection processes under dambreak flow conditions are much stronger, this term is retained in order to preserve the general form of the RANS model. The mass balance equation for k and ε are given by

$$\frac{\partial k}{\partial t} + \frac{\partial (ku_j)}{\partial x_j} = \frac{\tau_{ij}}{\rho} \frac{\partial u_i}{\partial x_j} + \frac{\partial}{\partial x_j} \left(v + \frac{v_t}{\sigma_k} \right) \frac{\partial k}{\partial x_j} + g \frac{v_t}{\sigma_c} (s - 1) \frac{\partial c}{\partial z} - \varepsilon \quad (3)$$

$$\frac{\partial \varepsilon}{\partial t} + \frac{\partial (\varepsilon u_j)}{\partial x_j} = c_{\varepsilon 1} \frac{k}{\varepsilon} \frac{\tau_{ij}}{\rho} \frac{\partial u_i}{\partial x_j} + \frac{\partial}{\partial x_j} \left(v + \frac{v_t}{\sigma_\varepsilon} \right) \frac{\partial \varepsilon}{\partial x_j} + c_{\varepsilon 3} \frac{k}{\varepsilon} g \frac{v_t}{\sigma_c} (s - 1) \frac{\partial c}{\partial z} - c_{\varepsilon 2} \frac{\varepsilon^2}{k} \quad (4)$$

Here, s representing the specific gravity of sediment. The standard values of empirical coefficients used in Eqs. (3)–(4) are: $C_{\varepsilon 1} = 1.44$, $C_{\varepsilon 2} = 1.92$, $C_{\varepsilon 3} = 0$, $\sigma_\varepsilon = 1.3$, and $\sigma_k = 1.0$. The turbulent viscosity is expressed as $v_t = C_\mu \frac{k^2}{\varepsilon}$, and

$C_\mu = 0.09$ is a closure coefficient. The sediment transport model solves the transport processes of graded suspended load and bed load as well as the bed change equations as per [41]

$$\begin{aligned} \frac{\partial c}{\partial t} + \frac{\partial}{\partial x_i}(cu_i) + \frac{\partial}{\partial x_j}(cu_j) &= \frac{\partial}{\partial x_j}\left(\frac{v_t}{\sigma_c} + v\right)\frac{\partial c}{\partial x_j} \\ &+ \frac{\partial}{\partial x_i}\left(\frac{v_t}{\sigma_c} + v\right)\frac{\partial c}{\partial x_i} \\ &+ \frac{\partial}{\partial x_j}(c\omega_s) \end{aligned} \quad (5)$$

$$\frac{\partial}{\partial t}\left(\frac{q_b}{u_b}\right) + \frac{\partial \alpha_x q}{\partial x} + \frac{\partial \alpha_y q}{\partial y} = \frac{1}{L}(q_b - q) \quad (6)$$

$$(1 - \eta)\frac{\partial z_b}{\partial t} = (D_b - E_b) + \frac{1}{L}(q - q_b) \quad (7)$$

where, σ_c is the Schmidt number (usually $0.5 < \sigma_c < 1$), and is set to be 0.8 in this work; q and q_b being the actual and equilibrium transport rates of bed load, respectively; L is the non-equilibrium adaptation length of sediment particles that is a characteristic distance for sediment to adjust from a non-equilibrium state to the equilibrium state under given flow and sediment condition; η is the porosity of sediment deposit; z_b is the bed elevation; α_x and α_y being the direction cosines of bed-load movement that is assumed to be along the near-bed flow direction, which are obtained from the horizontal components of near-bed flow velocity vector, u and v , as $\alpha_x = \frac{u}{U}$ and $\alpha_y = \frac{v}{U}$, here, $U = \sqrt{u^2 + v^2}$. The bed-load velocity u_b and the equilibrium transport rate of graded bed load, q_b are computed using the modified van Rijn [42] formula and Wu *et al.* [23], respectively, as

$$u_b = 1.64T^{0.5}\sqrt{gd_{50}\left(\frac{\gamma_s}{\gamma} - 1\right)} \quad (8)$$

$$q_b = 0.0053\sqrt{\left(\frac{\gamma_s}{\gamma} - 1\right)gd_{50}^3T^{2.2}} \quad (9)$$

Here, T is the transport stage number (excess shear stress) defined as $T = (\tau_{be}/\tau_{cr}) - 1$; τ_{be} is the effective bed shear stress related to the grain; and τ_{cr} is the critical shear stress of incipient erosion. Details about determining the sediment entrainment at the bed shear stress can be found in [41]. The settling velocity is calculated by Wu and Wang [43] and can be expressed as

$$\omega_s = \frac{Mv}{Nd} \left[\sqrt{0.25 + \left(\frac{4N}{3M^2}D_*^3\right)^{\frac{1}{n}}} - 0.5 \right]^n \quad (10)$$

where, $D_* = d_{50} \left[\frac{(\rho_s/\rho) - 1}{g} \right]^{1/3}$ and d = nominal diameter of sediment particles. D_* = particle size parameter, d_{50} is the median diameter of bed material (0.45 mm in this case).

The Corey shape factor of the sediment is assumed to be 0.7, and the corresponding values of M , N and n are 33.9, 0.98 and 1.33, respectively, used in this study. The exchange of sediment between the layers is done through deposition (downward sediment flux) at rate D_b and entrainment from the bed-load layer (upward flux) at rate E_b . So, the net flux at any given instant is $D_b - E_b$. The suspension flux and deposition flux are evaluated relate to reference concentration C_r , and deposition concentration C_d respectively, as $E_b = C_r w_s$ and $D_b = C_d w_s$. In this work, deposition concentration is obtained from the first neighboring grid point above the bed. However, the reference concentration formula of Fredsoe and Deigaard [44] is used to state the reference concentration with the critical Shields parameter as 0.05.

$$C_r(\theta) = \begin{cases} 0, & \theta < \theta_c \\ C_b \frac{\theta - \theta_c}{0.75 - \theta_c}, & \theta_c < \theta < 0.75 \\ C_b, & \theta > 0.75 \end{cases} \quad (11)$$

Shields parameter is calculated using friction velocity as

$$\theta = \frac{\rho u_*^2}{gd_{50}(\rho_s - \rho)} \quad (12)$$

3.2 Surface-capturing method

In the present model, the water surface movement is traced using the VOF method with Compressive Interface Capturing Scheme (CICS) to solve the following advection equation, which is in the conservative form, to compute the time evolution of the F -function as

$$\frac{\partial F}{\partial t} + u_i \frac{\partial F}{\partial x_i} + u_j \frac{\partial F}{\partial x_j} = 0 \quad (13)$$

A step function $F(x, z, t)$ is defined to be unity at any cell occupied by fluid and zero at cells occupied by empty. The average value of F in a cell would represent the fractional volume of the cell occupied by fluid. Discrete values of the dependent variables, including the fractional volume of fluid (F) variable used in the VOF technique, are located at cell positions shown in figure 2.

3.3 Boundary conditions

The boundary conditions are presented for the volumes adjacent to the water surface and the river bed. The initial condition is defined by designating areas of water in the upstream reservoir to a specified level with air filling the rest of the grid. The upstream boundary was set as wall due to the absence of flow into the reservoir and constant reservoir length. The downstream boundary was set as outflow for the dry bed test. For wet bed tests, it was set as

Figure 2. Position of variables in a typical mesh cell.

wall because the downstream end was closed by a vertical steel plate. At the free surface, the influence of wind shear is ignored and the pressure is assumed to be the atmospheric value. Since the water surface is defined by VOF,

zero shear stress and constant atmospheric pressure are applied as boundary conditions over the air–water interface. The wall function is adopted within the near-wall region, where the velocity is described by the logarithmic law. For sediment transport, logarithm law for rough bed is applied as

$$\frac{u}{u^*} = \frac{1}{k} \ln \frac{30z}{k_s} \quad (14)$$

For smooth bed, $k_s = 0$ and for stationary flat beds in laboratory experiments, k_s is usually set to the median diameter d_{50} of bed material, but in practice usually somewhat higher values are assumed as $k_s = 2d_{50}$ proposed by [45]. In this study, the net fluxes of horizontal momentum and turbulent kinetic energy, and the velocity normal to the surface are set to be zero. The dissipation rate ε is calculated from the relation given by [46] as $\varepsilon = \frac{k^{1.5}}{(0.43h)}$ at $z = z_s$, where h is local flow depth. Standard near-wall modeling is implemented to compute bed friction velocity u_* based on the model results of velocity, (u) obtained at the first grid point above the bed.

Figure 3. (a) Water surface profiles at different time intervals for upstream reservoir head $h_u = 0.25$ m with various flow depth ratios. (b) Water surface profiles at different time intervals for upstream reservoir head $h_u = 0.35$ m with various flow depth ratios.

Figure 3. continued

4. Results and discussion

In this section, the experimental and numerical results on the initial stage of dam-break flow are presented. The numerical model is first validated with the laboratory observations and further extended to simulate the sediment erosion in dam-break problem. The comparison results are illustrated in the following sequence: free-surface profiles, the wavefront velocities, as well as the impingement location of the forward breaking jet, velocity profile and bed erosion.

4.1 Free-surface elevation

The spatio-temporal evolution of free-surface along the length of the flume during the initial stage of dam-break is examined for six different water depth ratios ($r = 0, 0.05, 0.1, 0.2, 0.3$ and 0.4). The time evolution of water level are

recorded by means of ultrasonic probes at four gages located downstream of the gate. Here, the numerical results compared with the measured data are illustrated in figures 3(a) and (b) merely for two initial reservoir head (h_u) of 0.25 m and 0.35 m with three water depth ratios, $r = 0, 0.1$ and 0.2 . It is observed that the behavior of the free surface is quite different in the presence of larger water depth ratio. While the surface profiles are originally parabolic, the pressure is lower than hydrostatic. As time passes, the pressure becomes higher than hydrostatic due to bottom friction, causing a convex free surface. However, such difference gradually disappears and wave breaking is deferred with increasing the depth ratio (i.e., $r = 0.2$), as a result, the maximum turbulence and air entrainment are formed. Laboratory images are not presented here to identify regimes where air-bubble entrainment is significant. According to Shigematsu *et al* [36] the turbulence closure and bottom boundary condition may be inappropriate for almost dry-bed conditions. It is noticed that the

Figure 4. Non-dimensional impingement location of the forward breaking jet for three initial upstream cases: $h_u = 0.25$ m, 0.35 m and 0.45 m, with different water depth ratios.

numerical results do not agree with experimental data in the early stages ($t < 0.25s$) for dry-bed condition (i.e., $r = 0$), due to the violently breaking waves and substantial amount of entrapped air-bubbles during the breaking process. It is stated that for very small downstream water depth ($h_d = 0$) the energy of the upstream flow has not been fully released, owing to breaking jets that occur earlier. The breaking jet causes significant vertical flow structure and turbulence after impinging into the main flow, and has important implication to sediment erosion. In the wet-bed, the initial reservoir water pulled the tailwater downstream once the gate is lifted. At that moment, the still tailwater resisted to pull, and thus the wave front is broken and jet is formed in forward and backward directions. Stansby *et al* [1] observed jet like phenomenon after dam-break initiation. A similar trend is obtained in the present study over wet bed configuration. It is shown that the present computational results compare reasonably with measurements as the tailwater depth increases for the wet-bed pattern. Both measured and simulated data suggest that the peak value of the forward breaking jet velocity occurs at the water depth ratio $r = 0$, and starts to decrease at $r > 0.2$ due to the development of backward breaking jet. It is obvious that the present numerical method can successfully capture the complex topological changes of the turbulent free surface during the dam breaking.

Figure 4 shows the dimensionless impingement location of the forward breaking jet. It is noted that the normalized impingement locations [i.e., $x/(h_u - h_d)$] as a function of water depth ratio [$r = h_d/h_u$] for different h_u fall down into one curve. Indeed, there is an approximately linear relationship for $r \leq 0.3$. The numerical results are in agreement with measured data. However, further numerical

Figure 5. Comparisons of the average wave front velocity between the observed data and simulated results for various flow depth ratios.

simulations for a larger r value ($r = 0.4$) suggest the existence of a nonlinear relationship.

4.2 Wave front velocity

The propagation of the dam-break wave is a transient and non-uniform free-surface flow with large spatial and temporal gradients, especially at the initial stage of the movement. The behavior of the wave front depends on the water depth ratios $r = h_d/h_u$. Figure 5 shows the comparison of average wave front velocity between the experimental and numerical results as a function of flow depth ratio. The average velocity is computed over two fixed intervals of 3.5 m along the flume length. Propagation of the wave front is recorded in unsteady flow condition. It is found that a heavily concentrated and eroding wave front develops earlier and then depresses gradually as it propagates downstream. Hydraulic jump is formed in the early stage of the dam-break around the dam site, which diminishes gradually as it propagates downstream and ultimately disappears. It is observed that the velocity of the wave front is supercritical and larger than the reference wave speed $\sqrt{gh_u}$ for $r < 0.15$. For the depth ratio, $r > 0.25$ the wave front velocity is smaller than the reference wave speed and approaches $0.9\sqrt{gh_u}$. The experimental results demonstrate that the bed erosion and friction significantly affect the wavefront celerity. In all cases, the wave front is highly aerated and short-lived, but intensely chaotic with a strong splash especially for larger flow rates. It is also observed that the wave front velocity decreases with increase in water depth ratios, however, increases with increase in bed concentration. This aspect of the wave front is reasonably

Figure 6. (a) Dimensionless velocity profiles at upstream locations 1 and 2 for initial reservoir heads of 0.25 m and 0.35 m. (b) Simulated versus measured velocity profiles downstream of the lifted gate for initial reservoir heads of 0.25 m and 0.35 m at two different locations 3 and 4.

similar to the surge wave reported by [1, 3]. The numerical results are consistent with this trend, and are in fairly good agreement with the laboratory data.

4.3 Velocity profile

Figure 6(a) shows non-dimensional plots of velocity profiles $u(x, t)$ at two upstream locations for two initial reservoir depths (0.25 m; and 0.35 m). The velocity profiles at location 1 and 2 on the upstream reservoir, which are marked as $x = -0.80\text{ m}$ and -1.14 m , respectively far from the gate position. The time instance of this plot is set as 2.45 s. The velocity profiles in the upstream reservoir could be predicted satisfactorily with or without turbulence modeling. The results demonstrate that the vertical distribution of velocity is relatively uniform, and turbulence is

not significant. The velocity profiles closer to the free surface are uniform and a thin shear layer approximately 3% of the initial reservoir head can be observed in the near-bed velocity profiles in figure 6(a). During the early stages of the flow development following the lifting of the gate, the velocity magnitude is relatively small at the far upstream in the reservoir as compared with the value near the gate. Near the lifted gate, the velocity increased to its maximum value, and then started to decrease as the reservoir head dropped. The figure depicts that a difference of 0.1 m in initial reservoir head resulted in a difference of 0.28 m/s in velocity magnitude. Figure 6(a) shows an excellent collapse of the data confirming that the upstream velocity profiles are self-similar under different initial reservoir head.

Figure 6(b) shows a comparison of the relative root mean square error (RRMSE) between measured and simulated velocities at two downstream locations 3 ($x = 2.35\text{ m}$) and 4 ($x = 3.34\text{ m}$), respectively far from the gate position. The velocity profiles shown in this plot correspond to a time of 15.4 s after the gate is removed, and flow velocities are obtained using the UVP placed 0.045 m above the bottom of the flume. But, the vertical profiles of velocity could not be obtained in the downstream side due to the shallow depth. Initially, flow velocity increases with gate raise heights and gradually decreases along the flow direction with increased bed roughness.

The plot shows that velocity is small farther downstream of the gate, and is relatively uniform over the measurement distance during the later stages of the flow. The measurement versus simulation plot and the RRMSE value shown in figure 6(b) show satisfactory agreement.

4.4 Bed profile

Figures 7 and 8 display a comparison between the measured and calculated longitudinal bed profile at centerline i.e., $y = 0$ and cross-sectional bed profiles after 45 second for one adaptation length $L = 0.05$ and two different values of bed friction coefficient. The bed is eroded significantly immediately after gate opens. Once the gate is removed the flow starts to collapse due to gravity and strong vertical flow velocity is generated which results in a high level of turbulence at both the free surface and the bed. These two regimes of turbulence merge into one due to shallow downstream flow depth. Near the bed, the vertical component of the flow velocity is converted into a large horizontal component, which induces the bed stresses. More significantly, suspended sediment due to the high level of turbulence and local bed stresses is entrained into the breaking wave bore. Therefore, surface generated turbulent motion is interacting with bottom sediment suspension, causing high level of sediment suspended throughout the downstream water column. The plot shows significant erosion close to the initial gate location as a result of high bed shear stresses

Figure 7. Comparison of bed profiles along the flume length at $y = 0$ for two different upstream water depth: (a) $h_u = 0.25$ m and (b) $h_u = 0.35$ m.

Figure 8. Comparison of lateral bed profiles for two different upstream water depth: (a) $h_u = 0.25$ m and (b) $h_u = 0.35$ m at various locations: (i) $x = 10.5$ m, (ii) $x = 10.75$ m and (iii) $x = 11.25$ m.

and some of the eroded sediment deposited along the sidewalls due to decrease in the flow strength. While the dam-break wave moves towards the sides of the flume and

hits the side walls; hydraulic jump is formed and decreased in the flow strength. Since, the bed is made up of uniform sediment, the value of sediment transport capacity is

proportional to the velocity. This led to the largest bed erosion occurring close to the centerline and its value decreased away from the centerline. Significant bed erosion occurs as the dam-break wave front moves downstream. After the wave front passes, the erosion becomes much weaker. Erosion and deposition processes in the near-field are strongly influenced by vertical momentum fluxes because of the curvatures of the breaking waves. Flow acceleration and deceleration are a proxy for rate of change of bed shear stress and thus for the rates of sediment entrainment and deposition. The deposition configuration is controlled by the cross waves generated by the dam-break wave reflected from the side walls. The results of Wu and Wang [17] show a similar trend that closely matches with the present acquired results over sediment bed. Wu and Wang [17] confirmed that the water surface profile is significantly modified in movable bed than the rigid bed and the backward wave propagates basically at the same speed as over a rigid bed. Whereas the forward wave propagates more slowly over a movable bed than over a rigid bed at the initial stage but speeds up later.

It can be seen from figure 8 that the predicted bed levels for ($C_{bf} = 1$) are different from those for ($C_{bf} = 2$). The change in bed friction coefficient from 0.2 to 0.1 results in considerably less bed erosion and deposition. The smaller value of bed friction coefficient leads to underestimating the effective bed shear stresses, which is a driving force to erode the bed, and consequently generates less deposition at downstream of the flume. A discrepancy between the simulated and the measured profiles exists near the gate opening at $x = 8$ m because of the effects of erosion on the flow hydrodynamics. Considering the complicated aspects of sediment transport under dam-break flow, the model reproduces the erosion and deposition patterns usually well. The maximum erosion depth is about 85 mm which is located close to the gate location (i.e., $x = 8.2$ m). This is because further upstream, the flow weakens and cannot suspend sufficient amount of sediment to fill in the erosion hole under the initial gate location. The main characteristics of the turbulent and sediment suspension pattern for dry case are the intense interaction between the boundary layer and the breaking wave turbulence due to shallow flow depth. Thus, it is also critical to examine such interaction for cases of larger r value.

5. Conclusions

In this study, experimental measurements and numerical simulation of dam-break flow were conducted. According to the Lauber and Hager [2], the dam removal was instantaneous. An explicit finite volume method based on adaptive, non-staggered meshes was adopted to solve the RANS equations. A coupled approach has been used simultaneously to solve the flow and sediment transport processes and morphological changes induced by dam-

breaks. Model sensitivity analyses show that the bed friction coefficient, and sediment adaptation length are two important parameters in the developed model. The volume-of-fluid technique with the compressive interface capturing scheme was employed to track the water surface boundary. The developed model was tested using several laboratory experiments of dam-break flow over a sediment bed with dry and wet-bed downstream conditions. Time evolution of velocity profiles in the upstream reservoir and the flooded downstream region was obtained using UVP probes, and change of the water surface level was recorded using an ultrasonic measurement device. The velocity profiles in the upstream reservoir could be predicted satisfactorily with or without turbulence modeling. The velocity profiles are self-similar at different distances upstream of the dam and under different initial head.

Numerical results indicate that the maximum bed-erosion occurs at the gate location and it moves farther downstream depending on the ratio of the downstream and to the upstream water depth. The turbulence generated in bottom boundary layer dominates when dam-break wave propagate onto a dry bed. The results suggest that the complexity of dam-break flows and local erosion process near the wave front can significantly affect the morphodynamic. A fairly good agreement between model results and the measured data is obtained. It is demonstrated that the model is capable of simulating the interactive processes of the water flow, sediment transport and morphological changes caused by dam-break flow for different initial upstream reservoir heads with various water depth ratios. The present model can be extended for studying real-life problem.

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