Comparison of hydrodynamic and sediment transport model for dam break analysis over mobile bed.

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Abstract. A dam failure causing uncontrolled release of impounded water can cause considerable damage to life and properties. The intensity of the failure of dams varies depending on the extent of the flood characteristics, the size of the flooded area, the size of the population at risk and the time available for warning. In addition to this, huge morphological changes also occur at the downstream of the dam due to the dam break flood. This leads to the need for an analysis of sediment transport, together with the hydrodynamic analysis of the dam break flow, to provide a realistic picture of its devastating impact. This study uses an open source CFD software named TELEMAC-MASCARET to prepare a numerical model to simulate a 2-D dam break flow over mobile bed. A hydrodynamic model and an integrated model that comprises of sediment transport analysis are prepared and the results are compared to determine the most effective model for a 2-D dam break analysis. The study indicates that the integrated model performs better than the hydrodynamic model and replicates the real scenario more adequately.

1. Introduction

Dam break analysis is an area of research that is of interest to many hydraulic modelers, researchers and engineers due to its massive destructive consequences. Numerous researches have been done to analyze dam break flows using 1-D, 2-D and 3-D models using a variety of numerical schemes. Earlier simulations were conducted on the assumption that the bed was fixed. Nonetheless, given the significance of sediment movement that occurs during a dam break flood and its subsequent morphological changes that occur, in recent studies, morphodynamic analysis along with hydrodynamic analysis of the dam break floods have also been considered.

Any computational model requires that it be calibrated and validated against some actual data to verify its modeling capability and reliability to forecast a natural phenomenon. Dam break study is one such domain that has very little observed data that can be used to validate the model. It is for this reason that experimental setups are used to reproduce a scaled version of dam break incidents, the results of which could be used to validate a numerical model, which in turn can be used to forecast a real-time scenario.

This study is an attempt to compare the performance of a hydrodynamic and an integrated model using experimental data provided by Frazao et al. (2012) and Wu et al. (2018). It has been done using the software TELEMAC-MASCARET and its modules. The results have been compared with that of the experimental results.
2. Literature Review

Various literatures on dam break analysis, sediment transport during a dam break flood and validation of dam break studies in mobile bed has been extensively reviewed. A dam break flood, taking into account only the hydrodynamic flow parameters, would tend to be less accurate compared to that which considers both hydrodynamic and morphodynamic parameters. Xia et al. (2010) and Biswal et al. (2018) have discussed details of 2D morpho-dynamic models for predicting dam-breaks over mobile beds. Biswal et al. (2018) conducted experimental and numerical studies of dam-break flows over sediment beds under dry and wet downstream conditions, and investigated their effects on sediment transport and flow change on beds. Both studies suggest the need to consider sediment transports for dam break studies.

In order to consider sediment transport along with a hydrodynamic analysis an extensive study was conducted to select a suitable model for simulation. This led to the decision to adopt the TELEMAC-MASACARET software with its built-in coupling mechanism to integrate hydrodynamic and sediment transport process. Various literatures such as the study by Jean-Charles et al. (1991) describing equations, advection step, spread step, solution method and boundary conditions used in TELEMAC 2D, have proven to be extremely useful in learning about the integrated model used in this study. Begam et al. (2018), Reisenbuchler et al. (2018) and Villaret et al. (2013) have also used integrated models formulated by coupling the TELEMAC 2-D and SISYPHE modules of the TELEMAC MASACARET programme to simulate complex hydrodynamic and morphodynamic processes. All of these studies have been very useful in understanding a great deal about the model adopted in this paper.

Any numerical model must be verified with certain existent and established conditions that can ascertain that the model will simulate accurate results. Likewise this work was performed as a pilot study to validate an integrated model that was formulated for conducting the 2-D dam break analysis of a masonry dam. The experimental results from the Frazao et al. (2012) were used for validation. Wu et al. (2018) has also used the experimental results in Frazao et al. (2012) to verify the results of a coupled hydrodynamic and non-equilibrium sediment transport model developed on non-uniform rectangular mesh to simulate dam break flow over movable beds by using Godunov-type Riemann-problem-solver-free central schemes. Certain information of the experimental data used in this analysis were also taken from Wu et al. (2018).

A comparison of hydrodynamic and integrated model results has been implemented in this study to analyse the need for the use of an integrated model by studying the difference in results between a hydrodynamic model and an integrated model.

3. Numerical Modelling Approach

The experimental flume and the elevation data obtained from the above referred publications are modelled and analyzed using ArcGIS and BlueKenue software. The required numerical parameters are input and processed using TELEMAC 2-D and SISYPHE. The results are then analyzed using BlueKenue. Sediments are assumed to be non-cohesive in nature and the transport of bed loads has only been considered for sediment transport. The integrated model solves the classical Meyer Peter Muller’s bed load transport equations. A hydrodynamic model solving the Saint Venant’s equations has also been prepared. The water levels of the two models prepared and the bed levels of the sediment transport models are compared with the experimental results provide by Frazao et al. (2012) and Wu et al. (2018). Wu et al. (2018) have used the same experimental setup to conduct their study.

3.1. Experiment Data Used

The experimental data provided by Frazao et al. (2012) have been used to perform this study. Their study provides the modeler with the data required to simulate the physical process numerically, as well as the measurements of the water level and the bed level that can be used to compare the numerical results with the experimental results. The Laboratory experiments of dam-break flow over moveable beds were conducted at the Hydraulics Unit of the LEMSC (Mechanical and Civil Engineering Laboratory, Université Catholique de Louvain, Belgium). The experiments were carried out in a 3.6 m
wide and 36 m long flume with a useful length of 27 m (Fig 1). The dam was represented by a 1 m wide
gate located between two impervious blocks. The gate located at 12 m from the upstream end of the
flume was pulled up rapidly. The flume was covered with a 0.085 m thick layer of coarse sand extending
over 9 m downstream of the gate and 1.5 m upstream of the gate. The initial water depth was 0.47 m
upstream of the gate, while at the downstream there is no water. The characteristics of the bed material
and Manning friction coefficients have also been provided as shown in Table 1.

These are the data available for numerical modeling. They have also suggested the free selection for
Downstream Boundary Condition (DBC), to be simulated either as a free outflow or a closed wall. This
degree of freedom was given because it was assumed that the DBC would not affect the morphological
bed evolution during the limited test duration. The experiment was conducted for a time period of 20
seconds.

Figure 1. Flume dimensions (a) Plan (b) Elevation (c) Cross sections (source: Frazao et al. (2012))

Table 1. Data Available for numerical modelling

| Description                  | Value  |
|------------------------------|--------|
| d50                         | 1.61 mm|
| Specific Gravity \(\rho_s/\rho_w\) | 2.63   |
| Porosity \(\varepsilon_0\)   | 0.42   |
| Manning’s Coefficient n for fixed bed | 0.01   |
| Manning’s Coefficient n for sand bed | 0.0165 |

3.2. Numerical Model Used

The TELEMAC 2-D module and SISYPHE module that deals with the hydrodynamic and
morphodynamic analysis respectively of the TELEMAC suite has been used in this study for numerical
simulation. The Geometry and the bottom elevation data of the flume was created using a combination
of ArcGIS and BlueKenue (Pre/Post processing software available for TELEMAC). The initial
condition was assigned as an initial water depth of 0.47 m at the upstream of the gate and no water at
the downstream. Manning’s coefficient was also assigned as provided in the experimental data available.
The upstream boundary condition provided was a closed solid boundary and downstream free outflow
boundary. The total duration of the experiment was also assigned as 20 sec with a time step of 0.1 sec. The k-epsilon turbulence model has also been used.

3.3. Governing Equations

3.3.1. Hydrodynamic Equations. The governing equations used in this model are non-conservative form of the shallow water continuity and momentum equation with h (depth) and u and v (velocity components along x and y respectively) as the unknowns. The equations are as follows:

\[
\frac{\partial h}{\partial t} + \vec{u} \cdot \nabla h + h \nabla \vec{u} = 0 \quad (1)
\]

\[
\frac{\partial u}{\partial t} + \vec{u} \cdot \nabla u + g \frac{\partial h}{\partial x} = D_x + S_x - g \frac{\partial Z_f}{\partial x} \quad (2)
\]

\[
\frac{\partial v}{\partial t} + \vec{u} \cdot \nabla v + g \frac{\partial h}{\partial y} = D_y + S_y - g \frac{\partial Z_f}{\partial y} \quad (3)
\]

With h: water depth, u and v: components of the velocity field, g: gravity, \(D_x\) and \(D_y\): diffusion terms, \(S_x\) and \(S_y\): Source terms (bottom friction, Coriolis force, wind stress, etc.)

The diffusion terms are written as follows:

\[
D_x = div(v \nabla u) \quad (4)
\]

\[
D_y = div(v \nabla v) \quad (5)
\]

Where \(v\) may be a constant as well as a turbulent viscosity depending on space and time, and given by a turbulence model.

3.3.2. Bed Load Transport. The non-dimensional current-induced sand transport rate is expressed as

\[
\Phi_s = \frac{Q_b}{\rho_s s^3 - 1 d_{ch}^3} \quad (6)
\]

With \(\rho_s\) the sediment density; \(s = \rho_s / \rho\) the relative density; \(d_{ch}\) characteristic sand grain diameter for uniform grains; and \(g\) the gravity. As presented next, the non-dimensional sand transport rate \(\Phi_s\) is, in general, expressed as a function of the non-dimensional skin friction or Shields parameter \(\theta'\) defined by:

\[
\theta' = \frac{\mu \tau_0}{(\rho_s - \rho) g d_{ch}} \quad (7)
\]

with the correction factor for skin friction \(\mu\) and the bottom shear stress \(\tau_0\).

The classical Meyer Peters Muller’s bed-load formula validated for coarse sediments in the range \((0.4 \text{ mm} < d_{50} < 29 \text{ mm})\) has been used.

\[
\Phi_s = \begin{cases} 0 & \text{if } \theta' < \theta_c \\ \alpha_{mpm} (\theta' - \theta_c)^{3/2} & \text{otherwise} \end{cases} \quad (8)
\]

With \(\alpha_{mpm}\) a coefficient = 8, \(\theta_c\) the critical Shields parameter = 0.047.

3.3.3. Bed Evolution Equation. To calculate the bed evolution, SISYPHE solves the Exner equation:

\[
(1 - n) \frac{\partial Z_f}{\partial t} + \nabla Q_b = 0 \quad (9)
\]

Where \(n\) is the non-cohesive bed porosity, \(Z_f\) is the bottom elevation, and \(Q_b\) (m$^3$/s) is the solid volume transport (bedload) per unit width.
4. Results and Discussion
For the purpose of comparison of the hydrodynamic and sediment transport models the water levels at eight gauge points (Figure 2) and bed levels at 3 longitudinal axis (Figure 4) that had been provided in the study has been used to compare the simulated results as shown in Figure 3 and Figure 5. The simulated results for water levels comprises of results of the hydrodynamic model and Integrated model solved for MPM (Meyer peter Muller’s) bed load equation.

**Figure 2.** Gauge points located downstream of dam for water level measurements (Source: Frazao et al. (2012))
Figure 3. Water level comparisons at eight Gauge points

Table 2. R² and NSE values of water levels

| Gauge Points | R² (Integrated) | NSE (Integrated) | R² (Hydro) | NSE (Hydro) |
|--------------|-----------------|------------------|-----------|------------|
| US1          | 0.79            | -4.67            | 0.32      | -12.4      |
| US2          | 0.71            | 0.03             | 0.39      | -3.71      |
| US3          | 0.84            | 0.04             | 0.43      | -0.2       |
| US4          | 0.81            | -0.6             | 0.11      | -1.1       |
| US6          | 0.91            | 0.68             | 0.91      | 0.99       |
| US7          | 0.59            | 0.33             | 0.64      | 0.75       |
| US8          | 0.77            | 0.16             | 0.28      | -0.6       |

The visual interpretation of the graphs and the statistical comparison of the results show a clear variation in the water levels of the hydrodynamic and sediment transport model at the immediate downstream of the dam, although further downstream at the gauge points US6 to US8 the variations are not so significant.

The experimental results at US5 show an ambiguous trend different from the remaining points. This can be inferred as a likelihood of human error during measurement. Owing to the uncertainty, the comparison of results for this point has been omitted. The water levels predicted by the integrated model is lesser than that of the hydrodynamic model indicating the water level reduction due to the presence of sediments. However, according to the statistical comparison, the water levels predicted by the integrated model indicate a better agreement with the experimental results, with the exception of US1 and US4, which may be due to the inability of the model to reproduce the water reflection from the side walls. The values of R² and NSE clearly indicate that the hydrodynamic model is inadequate to predict a dam break flow. Although the integrated model has a fair agreement with the experimental results, there are several limitations that need to be addressed in order to improve its performance.

Figure 4. Longitudinal axis for bed levels
The MPM equation was applied to solve the bed load transport in the integrated model considering the range of particle size. The bed level comparison are shown in Figure 5 and Table 2 gives the $R^2$ and NSE values between the experimental and simulated results. The bed levels predicted by the sediment transport model give results in fair agreement with the experimental results.

5. Conclusion
This study attempts to prepare a 2-D dam break model using the TELEMAC – MASCARET software. A hydrodynamic model using TELEMAC 2-D and an integrated model coupling TELEMAC 2-D and SISYPHE module was prepared. The integrated model assuming bed load transport was run using the MPM bed load transport equation. The hydrodynamic as well as the integrated model are able to simulate unsteady flow. However the comparison of results indicates poor prediction by the hydrodynamic model. The integrated model is also able to predict the morphological changes occurring in the bed depicting scouring and deposition. The integrated model results are showing fair agreement with the experimental results. Most of the discrepancies may be attributed to the rough calibration of the friction coefficient, as the only available information consisted of an indicative Manning’s value and the mean size of the bed material (Frazao et al. 2012). This work has been used as a preliminary step for the validation study of a 2-D dam break analysis using an integrated model.
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