Conclusions from the 1st CADAM meeting - Wallingford, UK

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ABSTRACT

The first CADAM (Concerted Action on Dam-Break Modelling) meeting was held in Wallingford, UK (March 2 and 3, 1998) in the continuity of the former IAHR Working Group on Dam-Break Modelling. The aim was to test the capabilities of the numerical models used by the participants to reproduce experimental data on still more elaborated test cases, bringing each a new feature closer to real life cases. The first one concerned a wave propagating in a rectangular channel with a 45° bend, and the second one focused on bottom slope effects. A brief summary of the models and methods used by the participant is given, and followed by conclusions drawn from the numerical results and the discussions held during the meeting.

1) Introduction

Continuing with the activities promoted by the Working Group on Dambreak Modeling, a fourth meeting was held in Wallingford, UK, during 1-3 March, 1998 and hosted by Wallingford Research Ltd. The event was the first of a series of four meetings supported by DG XII of the European Economic Community under a Concerted Action devoted, generally speaking, to promote understanding and sharing of skills in the domain of dam break modeling within the European Community and abroad. In view of these premises the meeting has been named the *First CADAM (Concerted Action on Dambreak Modeling) Meeting*.

Previous meetings of the group addressed the performance of different numerical schemes in use with respect to analytical test solutions (Lisbon, November 1996) and their ability to reproduce laboratory simple test cases (Brussels, June 1997). In the light of what was discussed there, it was judged necessary to further test the capabilities of the models used to reproduce somewhat more elaborated laboratory test cases. After group discussion two configurations were selected: The first one concerned a dambreak wave propagating in a rectangular channel with a 45 degrees bend and will be named hereafter Test Case No. 1. The experimental work was undertaken by the group of Prof. Zech at Université Catholique de Louvain, Belgium.

Test Case No. 2 addressed the propagation of a one dimensional dambreak wave over a rectangular cross section channel with a bottom obstacle of triangular shape. The experimental data were obtained by Prof. Hiver at Université Libre de Bruxelles and Laboratoire de Recherches Hydrauliques (Châtelet), Belgium. As regards modeling, some eleven groups undertook the challenge of simulating the first test case while other nine coped with the second one. A full report containing a thorough compilation of the experiments performed and the numerical results obtained by the different researchers, will be published soon by the experimental groups involved.

2) Description of Test Case No. 1: Channel with 45° bend

The experimental setup made by Prof. Zech and his coworkers is composed of a reservoir of almost square base (239cm x 244cm) as shown in Figure 1. A rectangular channel 50cm wide and about 8m long stems out of it. The entrance of the channel is asymmetrically located with respect to the side of the reservoir. Water level in the reservoir is 20cm over the bottom of the channel, and the bottom of the reservoir is 33cm below that of the channel. Therefore an abrupt step is present at the entrance of the channel what has some influence on the outflow and poses certain difficulties to numerical modeling. The experimenters group estimated the Manning friction coefficients of channel bed and walls respectively in 0.0095 and 0.0195 $s/m^{1/3}$. The gate separating the channel from the reservoir plays the role of a dam and is pulled up mechanically very fast to simulate an instantaneous failure. At the end of the channel a chute was allowed.

Two sets of runs were performed: One with initial dry bottom in the channel and the other with an initial film of water 1cm deep. Water stage was obtained at time intervals from a set of either water level or pressure gauges in several locations along the channel.

The measurements provided by the two different means were cross-checked and resulted in very good agreement with respect to each other.

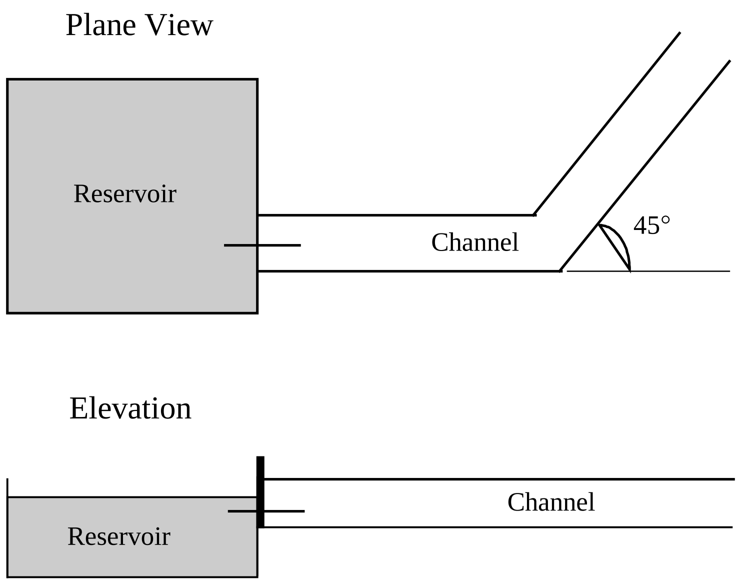


Fig. 1: Sketch of Test Case No. 1 experimental setup.

Also the reproducibility of the measurements was tested by checking the hydrographs at the different gauging points for several runs of the experiment. It can be concluded that measured results are reproducible as regards water level within five percent or less. Some pictures of the experiment were taken at selected times from a suspended camera in order to have a global view of the flow field. However no velocity measurements were taken.

3) Description of Test Case No. 2: Channel with triangular bump

This test case was adopted within the Dam Break Modeling group in order to test the ability of the numerical models to propagate a dam break wave upslope along a dry bed bump, overtop it and reproduce further wave interactions. Prof. Hiver and his group at Université Libre de Bruxelles and Laboratoire de Recherches Hydrauliques (Châtelet), Belgium carried out the experiment. The experimental setup, shown in Fig. 2, was made of a reservoir and a 22.5m long channel of rectangular cross section separated by a gate that plays the role of a dam and is suddenly removed to simulate an instantaneous failure. A bump of rectangular shape 40cm high and 6m long is located 10m downstream of the gate. The bump is symmetrical, thus composed of 3m uphill and 3m downhill. Although some of the computational groups used a 2-D model, the geometry is clearly one dimensional and expected 2-D effects are of minor importance as will be seen later on.

Three different forms of the experiment were run. In what Prof. Hiver called DRY test all the channel is dry at the opening of the gate and a chute imposed downstream. In test called WET1 the channel is dry upstream of the bump but the water depth is maintained at 0.15m downstream of it by means of a sharp crested weir. Finally in WET2 test the initial conditions are identical to those of WET1 but the weir is replaced by a wall high enough to prevent any spilling of water out of the channel. Therefore all waves attaining the end of the channel are completely reflected by the closing wall. Note that in all three cases the crest of the bump is dry at the initial instant since it is 40cm high. Also in all cases the initial water level in the reservoir attained 75cm.

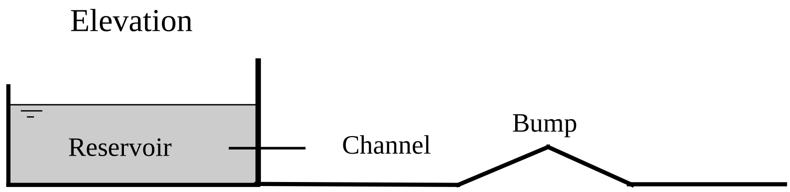


Fig. 2: Sketch of Test Case No. 2 experimental setup.

The estimated Manning coefficient was 0.0125 for the bed and 0.011 for the channel walls. A total number of nine gauges were located for the DRY case at 2, 4, 8, 10, 11, 12, 13, 16 and 20m downstream of the gate. Note that the cusp of the bump is located 13m downstream of it. For the WET1 and WET2 cases a total of twelve gauging points were installed at 2, 4, 8, 10, 11, 12, 13, 14, 15, 16, 18 and 20m downstream of the gate. The gauging devices were water level measuring probes (electrostatic) furnishing the water elevation at 0.1s time intervals during 40s. No water velocities were measured during the course of this experiment.

Reproducibility of the data was checked by running several times the experiment and comparing the readings of the gauges. Again water levels are reproduced with an accuracy better than five percent. The sequence of events that can be seen for the DRY and WET1 cases is the same in the part of the channel upstream of the bump. Namely a dam break wave propagating towards the obstacle, climbing upslope and overtopping it at about 4s. A strong wave is reflected by the obstacle and propagates upstream towards the reservoir. In the part of the channel downstream of the bump the wave pattern is more complex for the WET1 case because of the reflections produced at the weir. For the WET2 case no water leaves the channel and multiple reflections and interactions of waves traveling up and downstream and eventually overtopping several times the bump take place. For long times the water eventually becomes still at a level close to the height of the bump.

4) Brief summary of the models and methods used by the participants

As regards the numerical computations, all the groups involved based their models on the de Saint-Venant (Shallow Water) equations either in one (1-D) or two dimensions (2-D). Bottom friction was estimated with Manning’s formula by most modellers but also Chezy coefficient was used in some cases since it is better suited for dry bed computations. It was recommended the use of a spatial step of 5cm for test case 2, and the mesh was free for test case 1, and some codes ran on triangular cells. The characteristics of the numerical methods used can be briefly summarized as follows.

- N. Goutal and F. Maurel from Laboratoire National d'Hydraulique, EDF, France computed the flood wave in 1-D and 2-D modes. The 2-D model is a new version of code TELEMAC: Finite volume, Roe solver, unstructured grid (triangles 5cm), upwinding of source terms á la Vazquez and explicit time integration (CFL<1). Overall first order accurate.
  The 1-D computations were performed with code MASCARET 4.1 with similar characteristics as those mentioned above. An interesting feature of the model is the local 2-D representation of junctions or branches thus, coupling 1-D and 2-D computations.

- M. Nujic from the Federal Armed Forces University at Munich, Germany, performed 2-D simulations with package FLOODSIM: Cell centered finite volume method on structured grid (5cm), TVD space discretisation and time integration. Splitting of hydrostatic and convective terms to improve performance over irregular beds. Semi-implicit treatment of friction. Globally second order accurate.

- A. Paquier from Cemagref, Lyon, France, computed the test case in 1-D with code RUBAR3 and in 2-D with RUBAR20. They are based on variable extrapolation and Roe Riemann solver, two step explicit time integration (save implicit treatment of friction). Special computation of pressure flux to enforce horizontal water level in absence of flow. Also special is the treatment of dry cells.

- M. Szydlowski from Technical University of Gdansk, Poland used 2-D Shallow Water equations discretised in finite volumes with MUSCL reconstruction and upwinding of source terms á la Vázquez-Cendón. Second order accuracy.

- S. Soares from Université Catholique de Louvain, Belgium, performed 1-D and 2-D computations with codes B1D and B2DT. Flux splitting of the Boltzmann type with lateralisation of the source terms arising from bed slope. No special treatment for dry beds. 2-D on triangular grids.

- B. Zhang and J.M. Tanguy from University of Technology of Compiègne and Ministry of Public Works respectively used code REFLUX 4.0, based on 2-D Finite elements with implicit Euler or explicit Lax-Wendroff integration schemes. It includes mixing length or constant viscosity turbulence models. Elements used of 4cm size. No shock capturing ability.

- T. Viseu, A. Bento-Franco and A. Betâmio de Almeida from Technical University of Lisbon and National Laboratory of Civil Engineering, Portugal performed the simulation with model BIPLAN: Second order explicit, mixed forward and backward finite differences, fractional steps scheme. TVD limited and Roe linearisation with Harten entropy condition and van Leer limiter. CFL 0.8. Mesh size of 0.5cm.

- E. Vázquez-Cendón from University of Santiago de Compostela, Spain, used the Q-scheme of van Leer in unstructured meshes and a specialized type of finite volume. Source terms are upwinded to cope with abrupt bathymetries in moderately fine meshes.

- P. García-Navarro and P.Brufau, from University of Zaragoza, Spain, employed several methods to compute the 1-D test case, namely Roe´s first order method, Lax-Friedrichs and McCormack TVD with upwinding of source terms. For the 2-D computation a multidimensional splitting code of first order accuracy and pointwise integration of source terms was used . Time discretisation was Euler explicit.

- A. Sleigh from University of Leeds, UK, adopted an unstructured grid approach on triangles based on MUSCL variable extrapolation and explicit time integration with a flux form of the bed source term. Second order global accuracy in time and space. Special treatment of dry beds. A particularly coarse mesh in the 2-D case is to be remarked.

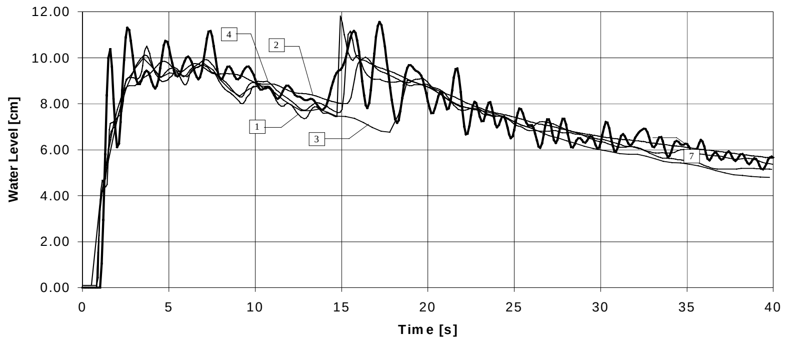
- F. Alcrudo from University of Zaragoza, Spain performed both 1-D and 2-D computations with code SW2D, based on finite volume Muscl discretisation in body fitted structured multiblock meshes. It uses Roe type Rieman solver and two step explicit time integration. Source terms are treated semi-implicitly and pointwise. For the 2-D test case a very fine grid of 2cm (more than twice as fine as the one proposed) was used. For the 1-D test case only one row of cells was used with the recommended spacing of 5cm.

- No accurate information is available as of this writing about the method used by L. Fraccarollo from Universita di Trento, Italy, who also presented results at the meeting.

As can be seen from the above paragraphs most researchers employed alike methods. Special references must be done to the Boltzmann type of Riemann solver used by S. Soares and to the special treatment of source terms proposed by M. Nujic and E. Vázquez-Cendón. It is interesting to note that some modellers included wall friction in their computations and some others did not, what as will be seen later, apparently does not influence much the quality of the results.

5) Results for Test Case No. 1: Channel with 45° bend

As regards this test case it must be firstly stated that no big differences can be observed on either the experimental or the computed water levels between the runs with dry and wet bottom. The most remarkable difference is that fronts propagate slightly faster in the wet case. Therefore all comments made for the dry bed run in what follows are valid also for the wet bed situation and vice-versa. Analyzing the hydrographs at the nine different gauging points one can see general good agreement between experimental and the different computational results.



— experimental
1- Alcrudo (UZ)
2- Soares (UCL)
3- Sleigh (UL)
4- Fraccarollo (UT)

Figure 3 : Test Case 1, G3

Gauge No. 1 is located inside the reservoir and it is to be noticed that methods not taking into account the bottom mismatch between reservoir and channel follow closer the experimental hydrograph (for instance results by Sleigh and Alcrudo). Usually results from those participants taking into account the bottom mismatch show a faster emptying of the reservoir than actually measured (for instance Goutal and Maurel and Nujic). This is somewhat unfortunate because the right way to model the problem is taking into account the entrance step. Usually those who have dropped it out of their simulation have done it to avoid spurious velocities generated at abrupt slopes. It must be said, however that differences between both possibilities are small. In relation with this, it is interesting also to note that the velocity of small amplitude waves generated inside the reservoir are better modeled by those who consider the right depth inside the tank, as could be expected.

Considering gauge No. 2 located a few centimeters after the entrance to the channel again good matching between experimental and computed results is found. Usually the front height is better reproduced by those taking the entrance step into account but mean water level is then slightly over predicted. The arrival time of the front reflected at the bend is also usually well predicted. This ideas can be extended on to the next gauges, No.3 and 4, located half way between the channel entrance and the bend and just before the bend respectively. The arrival time of the reflected front at gauge No. 4 is over predicted by almost all methods in the wet bed case and not so much on the dry bed case.

Gauges No. 5 , 6 and 7 are located along a line across the channel section just after the bend. The flow there is quite complicated since a curved shock is formed after the arrival of the dam break front. This curved shocks progresses slowly across the channel until it impinges obliquely onto the inner wall and is reflected several times downstream. Meanwhile the bend has been drawn with the incoming water and a reflected front starts traveling upstream. In this situation a small inaccuracy in the location of the *numerical probes* can provide quite a different hydrograph. Despite this fact good agreement is found at this probes too as the envelope of the whole set of numerical results show.

Finally Gauges No. 8 and 9 correspond to points a few cm downstream of the bend and halfway between the bend and the exit respectively. Usually arrival times of the first front are very well predicted by most methods although some inaccuracies affect the water level.

It can be said that the Shallow Water representation and the numerical solution techniques adopted are well suited to predict this problem with errors not exceeding twenty percent save in very particular situations. For instance, there is no possibility to take into account splashing within this mathematical framework and this is a phenomenon that occurs during main front reflection at the bend. Also 1-D models are not capable of reproducing the complex front reflection phenomena present and this effect can clearly be seen in their computed hydrographs at the bend.

6) Results for Test Case No. 2: Channel with triangular bump

This set of experiments is of 1-D nature and therefore it must be firstly stated that no big differences appear between 1-D and 2-D models. More differences can be found when comparing computed hydrographs with models taking into account wall friction and those not considering it. In any case the dispersion of data is even less pronounced than that of Test Case No. 1 what gives confidence on the models used.

As regards the DRY case it must be said that all methods produce similar hydrographs to those obtained from the experiment. There is a clear tendency towards overestimating the intensity (height) of the arriving front and a minor dispersion about its arrival time. Somewhat more pronounced differences are found for the arrival time of the front reflected at the bump. Remarkable are the results by A. Paquier as regards prediction of the arrival times of the incoming and reflected fronts which are computed exactly. The other participants are in any case very close to it.

In the WET1 case no differences appear with respect to the previous one save for the gauges located downstream of the bump. In the WET1 case more front reflections and different water depths are produced because of the presence of the weir. Discrepancies appear mostly concerning the arrival time of the front reflection at the weir. Usually the computation predicts an earlier arrival. In some extreme cases this front is not even computed but this fact is probably due to an erroneous boundary condition.

Finally WET2 case is the most complicated due to the multiple front reflections and overtopping of the bump. Surprisingly a good deal of methods reproduce quite accurately intensity and arrival time of the multiply reflected fronts. The highest errors are found at the gauge located at the cusp of the bump, where although fronts are accurately located in time, their intensity is mispredicted by about thirty percent. Specially striking is the complete agreement between computed and measured water elevation at the farther gauge located 20m downstream of the gate.

7) Conclusions

A general overview of the work presented at the First CADAM Meeting has been given to the reader. It has been intended to give an impression on what can be expected from the methods presently in use by some researchers in the field. As has been put forward, most of them rely upon the simple Shallow Water theory but, nonetheless, they are able to predict important features of very complex flow phenomena that, at present, cannot be handled by more elaborated theories or more complete systems of equations such as Navier-Stokes ones. The test cases discussed here still represent strong simplifications with respect to real life dambreak flows but share with them some common characteristics, like front propagation, dry beds and abrupt bathymetries. The purpose of CADAM project is to pursue the testing and enhance the modeling capabilities of present methods, and in order to do this actual dambreak cases in reduced laboratory models as well as real life events will be modeled and the results presented and discussed in forthcoming meetings.

8) References

First CADAM (Dambreak Modeling Concerted Action) Meeting - Fourth Meeting of the Working Group on Dam Break Modeling. Wallingford, UK, March 1998. Technical Report. Publication under way.