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Application of a coastal modelling code in fluvial environments

I.M. Hartanto [a], L. Beevers [a], I. Popescu [a,*], N.G. Wright [b]

[a] UNESCO-IHE Institute for Water Education, Hydroinformatics and Knowledge Management, P.O. Box 3015, 2601 DA, Delft, The Netherlands
[b] University of Leeds, Leeds LS2 9JT, United Kingdom

A R T I C L E I N F O

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A B S T R A C T

XBeach is an open source, freely available two dimensional code, developed to solve hydrodynamic and morphological processes in the coastal environment. In this paper the code is applied to ten different test cases specific to hydraulic problems encountered in the fluvial environment, with the purpose of proving the capability of XBeach in rivers. Results show that the performance of XBeach is acceptable, comparing well to other commercially available codes specifically developed for fluvial modelling. Some advantages and deficiencies of the codes are identified and recommendations for adaptation into the fluvial environment are made.

Software Availability

Name of software: XBeach
Developers: It is a public-domain model that has been developed with funding and support by the US Army Corps of Engineers, by a consortium of UNESCO-IHE, Deltares (Delft Hydraulics), Delft University of Technology and the University of Miami
Contact address: UNESCO-IHE Institute for Water Education, Westvest 7, 2611 AX, Delft The Netherlands
Availability and Online Documentation: Free download with manual and supporting material at: http://public.deltares.nl/display/XBEACH/Home
Year first available: 2004
Hardware required: IBM compatible PC
Software required: MS Windows (tested on Windows XP)
Programming language: FORTRAN 99
Program size: 4.9 MB

1. Introduction

Historically coastal modelling software has developed out of different constraints from fluvial software, due to the necessity of representing different characteristics of hydraulic behaviour. Parameters like wind and tidal forces, which have high influence in the coastal environment (de Vriend, 1991), have minor effects in fluvial environments. Conversely, a longitudinal slope and varying initial water level, which are very important in river modelling, are not considered important in coastal modelling. However, the hydraulic calculations are similar, hence coastal software can be applied in fluvial areas. The application of a code outside its original domain needs to be verified and tested comprehensively before wider application is attempted.

XBeach is open source coastal software developed to model coastal flooding, sediment transport and morphological changes in two dimensions. The software contains a number of sub-routines which solve the non-stationary two dimensional shallow water equations that are able to calculate a fluvial flood wave. Open source codes provide payment-free software (usually under the GNU Public License – http://www.gnu.org/licenses/gpl.html) to users, which is a key advantage in developing countries (Bitzer, 2004; Lanzi, 2009). This approach allows the user to modify the code to meet their specific requirements (Henley and Kemp, 2008) and can lead to a significant development and improvement of the code, whilst affording flexibility.

This research tests the validity of applying this freely available software in a cross-over domain (fluvial environments), which opens up its use to a greater number of professionals, and also permits use in coastal/river transition zones such as estuarine areas. Due to the morphological tools available within the software, specialists would also have the possibility of accessing free 2D

* Corresponding author. Tel.: +31 15 2151895; fax: +31 15 3122921.
E-mail addresses: isnaeni@gmail.com (I.M. Hartanto), l.beevers@unesco-ihe.org (L. Beevers), i.popescu@unesco-ihe.org (I. Popescu), n.g.wright@leeds.ac.uk (N.G. Wright).

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sediment transport capabilities. XBeach has generally been used as a stand-alone model for small scale coastal applications. It has many capabilities such as: depth-averaged shallow water equations including subcritical and supercritical flow, time-varying wave action balance, wave amplitude effect and the depth-averaged advection-diffusion equations (Roelvink et al., 2009). This paper focuses solely on the depth-averaged shallow water equations solver.

The main objective of the development of the XBeach was to provide modellers with a robust and flexible environment where the concepts of dune erosion, over washing and breaching can be tested (Roelvink et al., 2009). During the code development, the stability of the numerical method was considered as a top priority. Consequently, first order accuracy was accepted since the software concentrated on representing near shore and swash zone processes which have strong gradients in time and space (Roelvink et al., 2008). Such accuracy is the norm in river modelling software.

The objective of this paper is to test the applicability of this coastal software (XBeach) in the fluvial environment. This is completed through a number of tests which are designed to recreate particular hydraulic problems encountered in fluvial flooding scenarios. The tests include comparison to semi-analytical calculations, other modelling codes and laboratory experimental results. The aim is to demonstrate that an open source approach is also applicable in the fluvial environment.

2. Theoretical background

2.1. Numerical methods

The increased demand for improved safety against flooding, prompted the development of mathematical models which describe flow propagation in rivers. These mathematical models, in most cases, do not have an analytical solutions and are solved using numerical methods (Ferziger and Peric, 1999). Flow description in rivers, lakes and coasts are long waves, which can be described by means of the so-called Shallow Water Equations. These are a hyperbolic set of partial differential equations depending on the nature of the problem to be solved. These equations describe the mass conservation and momentum conservation.

Significant effort during the 1980's and 1990's was devoted to defining efficient and accurate numerical methods for hyperbolic systems. Mathematically the hyperbolic equations permit discontinuous solutions and their numerical integration should lead to the computation of such discontinuities sharply and without oscillations.

The differential form of the Shallow Water equations, in the reference framework of Fig. 1, are:

$$\frac{\partial u}{\partial t}+\frac{\partial f}{\partial x}+\frac{\partial g}{\partial y}=s,\ \text{or}\ \frac{\partial u}{\partial t}+\mathrm{B}\frac{\partial u}{\partial x}+\mathrm{C}\frac{\partial u}{\partial y}=s\quad \text{with B}=\frac{\partial f}{\partial u}\ \text{and C}=\frac{\partial g}{\partial u} \tag{1}$$

Where:

$\Omega \subseteq \mathbb{R}^2$ is the domain of computation; σ is any open subset of $\mathbb{R}^2$ with boundary Γ n is the outward unit normal.

The vectors included in the equation are:

$$\begin{aligned}
\mathrm{u} &= \left(h, q_x, q_y\right)^T \\
\mathrm{f} &= \left(q_x, \frac{q_x}{h}+\frac{g}{2}h^2, \frac{q_x q_y}{h}\right)^T, \quad \mathrm{g} = \left(q_x, \frac{q_x q_y}{h}+\frac{q_y^2}{h}\frac{g}{2}h^2\right)^T \\
\mathrm{s} &= \left(0, gh\frac{\partial h_o}{\partial x}, gh\frac{\partial h_o}{\partial y}\right)
\end{aligned} \tag{2}$$

With q(x,t) -the unit-width discharge,

h_o(x, y)-the depth under the reference plane in Fig. 1,

ζ(x, y, t)-the elevation over the same reference plane,

h(x, y, t) = ho + ζ

g − the gravitational acceleration

s − the source term which accounts for the bottom slope

Γ is the boundary of σ.

B and C the Jacobian matrices of the fluxes f and g respectively.

Equations (2) are the conservative form of the Shallow Water equations (all the spatial derivatives of the unknowns are in the form of a divergence operator). In the case of a flat bottom ($h_o = 0$) the right-hand side of the equation is 0 and the equation is the strong conservation form of the Shallow Water equations.

The Shallow Water equations have an infinite hierarchy of conservative forms (Ambrosi, 1995) expressing the conservation of mass, energy, discharge rate, velocity, etc. Any two of these equations

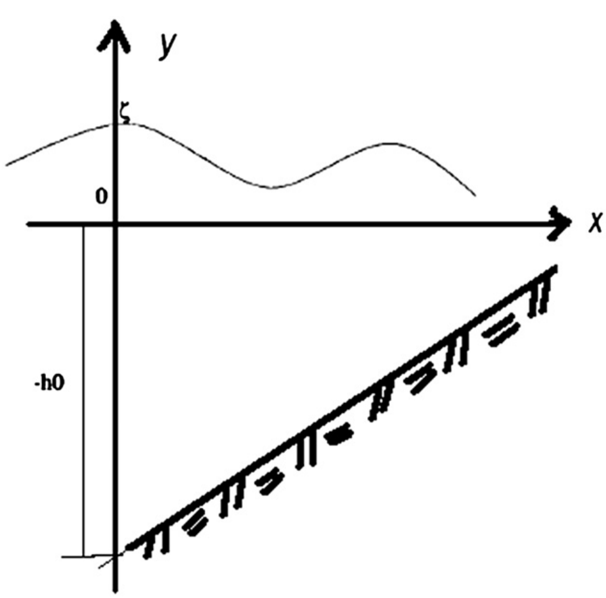

Fig. 1. Elevation and Depth for Shallow Water equations.

Table 1
Summary of tests completed.

Test no.	Name	Description	Figure no.
1a, 1b	Semi-analytical	Comparison of the model runs with semi-analytical solutions	Fig. 3
1c, 1d		M1, M2 curves (mild slope); S2, S3 curves (steep slope)	
2a	Idealised	Flow in a straight idealised channel	Fig. 4
2b		Flow in an embanked straight idealised channel	
2c		Flow in a meandering idealised channel	Fig. 5
3	EA case 1	Wetting and drying of a disconnected body	Fig. 6a & b
4	EA case 2	Low momentum flow	Fig. 7
5	EA case 3	Momentum conservation	Fig. 8
6	EA case 4	Flood propagation over a plain	Figs. 9 & 10
7	EA case 5	Dam break over a valley	Figs. 11 & 12
8a	EA case 6	IMPACT: Hydraulic jump and wake zone (laboratory scale)	Fig. 13
8b		IMPACT: Hydraulic jump and wake zone (realistic scale)	Fig. 14
9	Experimental	Dam break through an urban area	Fig. 15

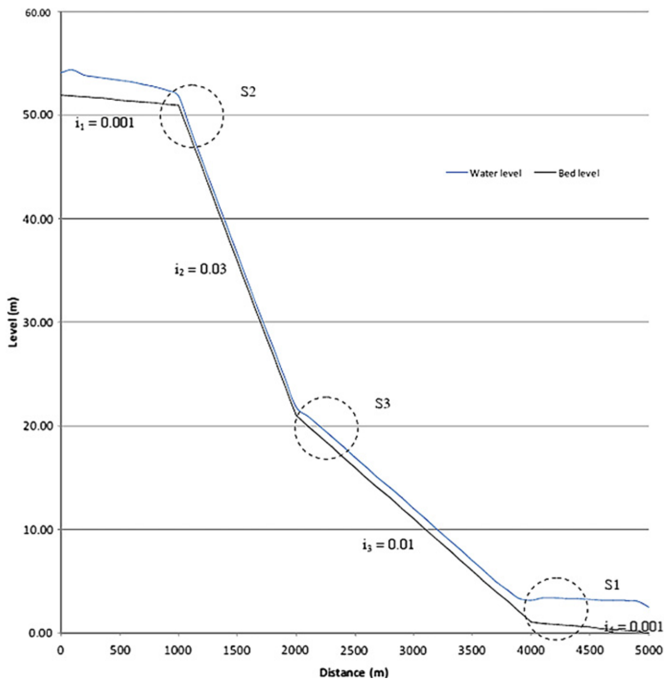

Fig. 2. Connections of the slopes on a river profile for the S1, S2 tests.

are equivalent to one another if the solution belongs to C. This equivalency is no longer valid when shocks (i.e. bores) are involved.

Current codes which solve the Shallow Water equations do so using different numerical methods. From the current literature, several numerical techniques for solving the Saint Venant Equations are available. These include the method of characteristics, explicit difference methods, semi-implicit methods (Casulli, 1990), fully implicit methods, and Godunov methods (Van Leer, 1979). The characteristic method transforms the Shallow Water partial differential equations into a set of ordinary differential equations, which are solved using finite difference methods. The explicit methods transforms the Shallow Water equations into a set of algebraic equations, which can be solved, in sequence, at each point of discretisation, at each time step, while implicit methods solve the equations simultaneously at all computational points at a given time. If, due to boundary conditions and assumptions, the set of Shallow Water equations are non-linear, iteration is needed in order to find the solution (Richtmeyer, 1957).

Numerical stability and convergence issues need to be addressed, while solving the Shallow Water equations numerically. In order to prevent error propagation in explicit methods, the Courant Frederichs Levy (C.F.L.) condition is imposed. This relates the time step to the spatial discretisation and the wave speed, i.e. the time step must be less than or equal to the ratio of the reach length to the minimum dynamic wave celerity $\Delta t \leq \Delta x/c$.

Godunov-type methods can be explicit or implicit. Generally, however, they are explicit in time and, accordingly, the allowed time step is restricted by the C.F.L. stability condition. These methods are in general based on non-staggered grids and can achieve first order accuracy. Godunov-type methods were originally developed for gas dynamics and were then later extended to hydrodynamics on the basis of the analogy between the equations for isentropic flow of a perfect gas with constant specific heat and the Shallow Water Equations (Toro, Leveque).

Semi-implicit methods can be unconditionally stable and still computationally efficient. A semi-implicit method that conserves the fluid volume, applied to channels with arbitrary cross-sections was introduced by Casulli and Zanolli, (1998). However, these methods have to be carefully considered, especially in the case when the physical conservation property of momentum is not satisfied, since incorrect results arise if the methods are applied to discontinuous problems. However, when a semi-implicit scheme using the efficiency of staggered grids is combined with the conservation of both fluid volume and momentum then problems addressing rapidly varying flow can be solved (Stelling and Duinmeijer, 2003).

There are a number of different numerical schemes embedded in different codes, for example: the weighted four point-Preissmann scheme (Preissmann, 1960), Godunov-based methods (LeVeque, 1992), the the weighted six-point Abbott-Ionescu scheme (Abbot and Ionescu, 1967), and TVD (Total Variation Diminishing) schemes (Toro, 1997). Each numerical scheme has its own advantages and disadvantages. Below schemes relevant to this paper are discussed.

The first schemes developed for hydrodynamic computational codes were the fully implicit schemes of Preissmann and Abott–Ionescu during the 1960s. These schemes have developed over time (most significantly in terms of graphical user interfaces GUI), but remain the most popular and widely used in commercially available software. Two examples of codes that use the Preissmann scheme are DAMBRK, which was developed by the US National Weather Service, and ISIS which was developed by Halcrow and HR Wallingford in the UK. An example of a code using Abbott-Ionescu scheme is Mike11, developed at Danish Hydraulic institute.

Table 2
Summary of model meshes, sizes and boundary condition types.

Test Case	Grid size	Grids counts	Roughness C(Chezy) N (manning)	Boundary conditions		Time step	Time span
				Upstream	Downstream		
1a	100 × 100 m	100 × 2 cells	$C = 50$	Constant Discharge	Constant Water Level	5 s	150 min
1b	100 × 100 m	100 × 2 cells	$C = 50$	Constant Discharge	Constant Water Level	5 s	150 min
1c	10 × 10 m	400 × 11 cells	$C = 50$	Constant Discharge	Constant Water Level	5 s	150 min
1d	10 × 10 m	400 × 11 cells	$C = 50$	Constant Discharge	Constant Water Level	5 s	150 min
2a	10 × 5 m	505 × 22 cells	$C = 50$	Varying Discharge	Constant Water Level	30 s	130 min
2b	10 × 5 m	505 × 22 cells	$C = 50$	Varying Discharge	Constant Water Level	30 s	130 min
2c	100 × 100 m	598 × 133 cells	$C = 50$	Varying Discharge	Constant Water Level	5 s	150 min
3	2 × 2 m (10 × 10 for other soft.)	350 × 50 cells	$n = 0.03$	Varying Water Level	N/A (wall)	60 s	20 h
4	20 × 20 m	100 × 100 cells	$n = 0.03$	Varying Discharge	N/A (wall)	60 s	48 h
5	5 × 5 m	60 × 20 cells	$n = 0.05$	Varying Water Level	N/A (wall)	2 s	15 min
6	5 × 5 m	200 × 400 cells	$n = 0.05$	Varying Discharge	N/A (wall)	60 s	5 h
7	50 × 50 m	160 × 340 cells	$n = 0.04$	Varying Discharge	N/A (wall)	60 s	30 h
8a	0.1 × 0.1 m	36 × 990 cells	$n = 0.01$	N/A (initial water level)		1 s	2 min
8b	2 × 2 m	36 × 990 cells	$n = 0.05$	N/A (initial water level)		15 s	30 min
9	0.05 × 0.05 m	72 × 716 cells	$n = 0.01$	N/A (initial water level)		1 s	2 min

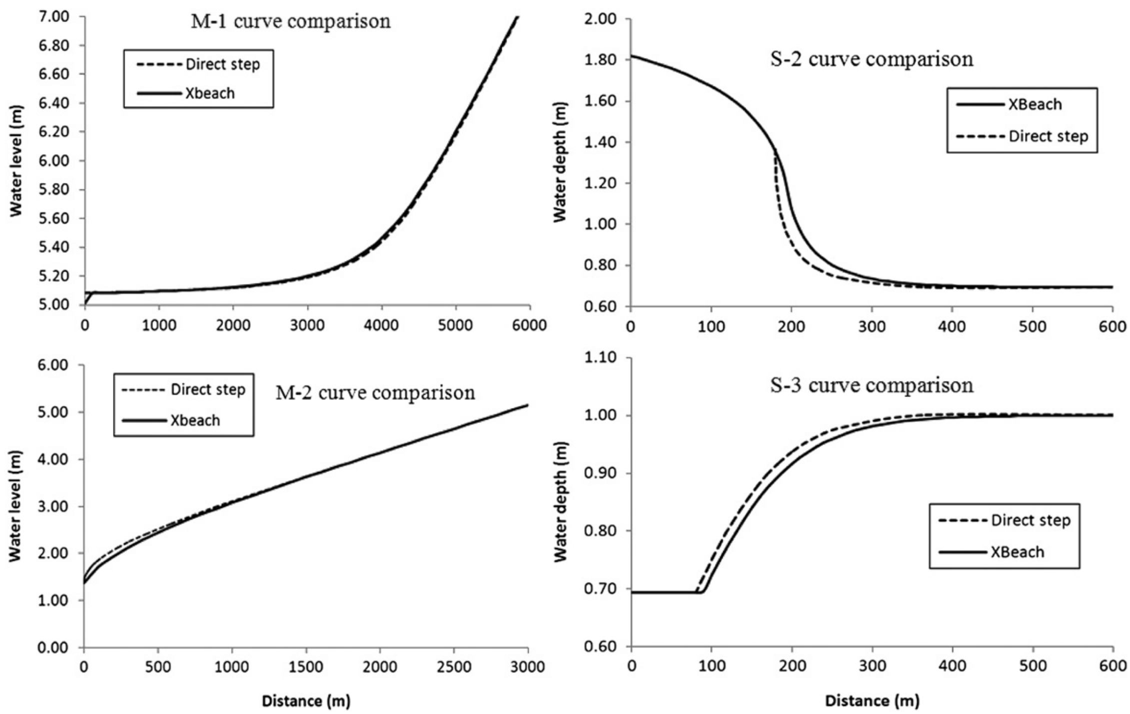

Fig. 3. Modelled and analytical solution of M1, M2, S2, and S3 types of flow (Test 1a, 1b, 1c, 1d).

Godunov developed a method to solve the non-linear systems of the hyperbolic conservation laws describing fluid flow. As a result, the scheme is able to solve the Riemann problem by including various approximate Riemann solvers (Toro, 1997). The Riemann problem is a discontinuity of the conservation law and of piecewise constant data (LeVeque, 1992). Godunov-based schemes with various Riemann solvers are used in river modelling software such as Infoworks RS 2D (Roca and Davison, 2009), TRENT (Villanueva and Wright, 2006) and BreZo (Begnudelli et al., 2008).

The TVD method is able to solve the competing requirement of high order of accuracy and the absence of unphysical oscillations in the vicinity of large gradients (Toro, 1997). A Riemann solver can also be integrated with this method to handle shock capturing. Furthermore, TVD upwind schemes, where the solution in space develops from left to right, are the extension of the Godunov first order upwind method. The TVD scheme has been implemented in the ISIS 2D software (Lin et al., 2006).

Finally, XBeach uses the Stelling and Duinmeijer scheme (Stelling and Duinmeijer, 2003), combining the efficiency of staggered grids with momentum conservation properties needed to ensure accurate results for rapidly varied flows and expansion and/or contractions. This method is very efficient in simulating large scale inundation (Stelling and Duinmeijer, 2003).

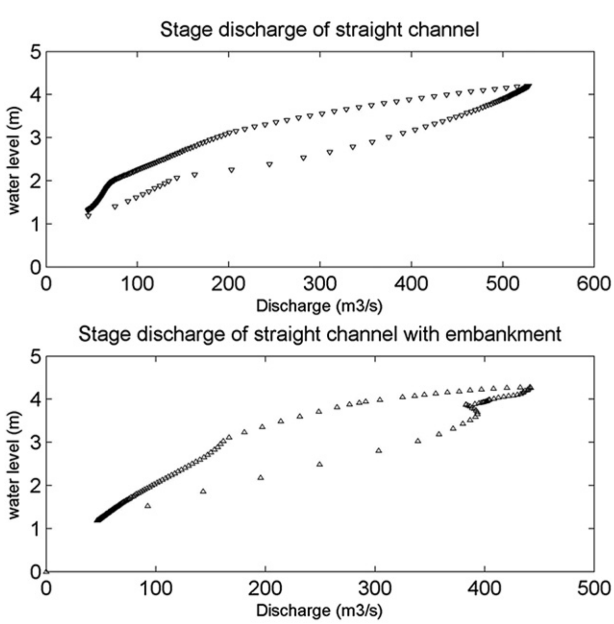

Fig. 4. Test 2a and 2b, Stage discharge in straight channel.

2.1.1. XBeach formulation

XBeach uses a rectilinear, non-equidistant, staggered grid. This discretisation calculates bed level, water level, water depth and concentration of sediment at cell centres while velocities and sediment transport are calculated at the cell border. Velocities at

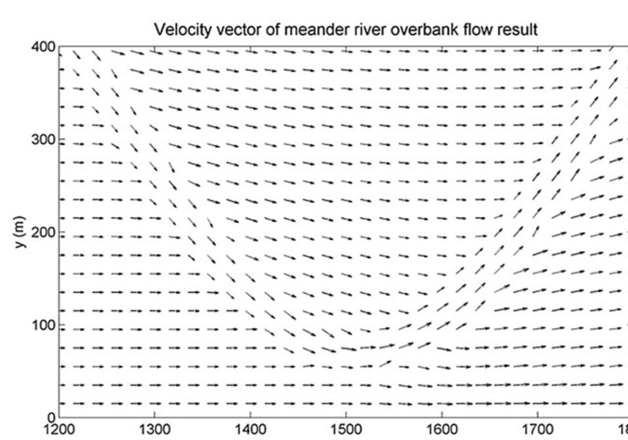

Fig. 5. Test 2c: Flow pattern in meandering channel.

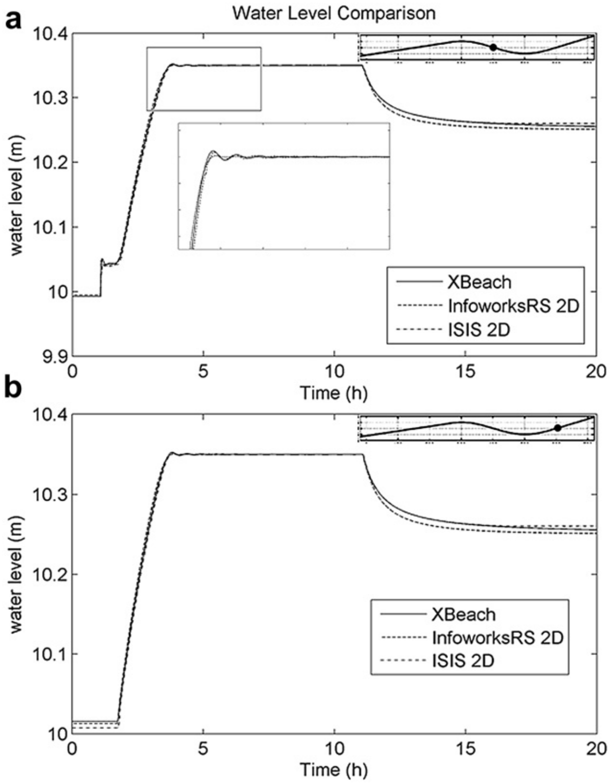

Fig. 6. a. Test 3 – wetting and drying of a disconnected body (results recorded on the downslope of the initial bump).

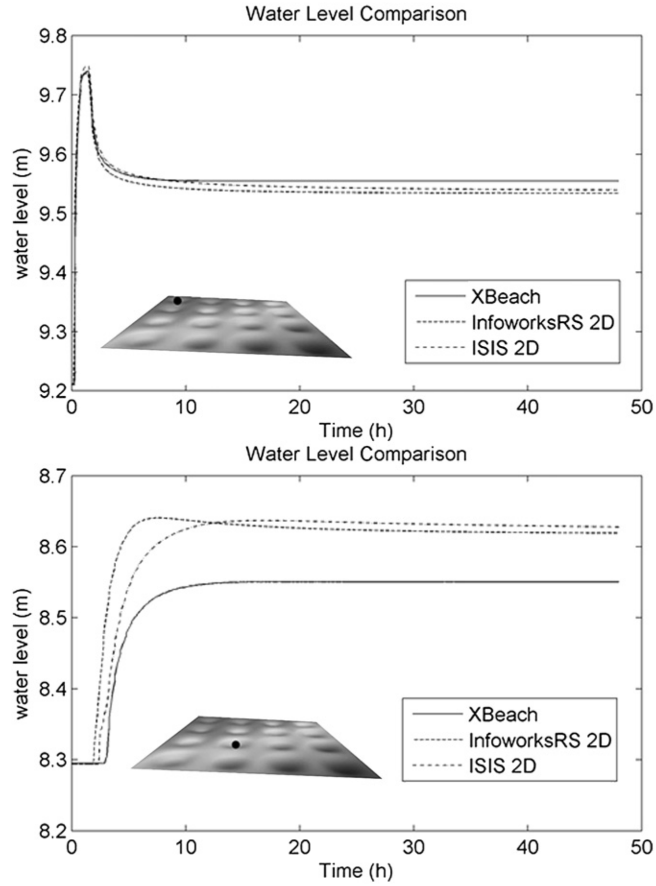

Fig. 7. Test 4 – Low momentum flow.

the cell centres are obtained by interpolating the results from the four surrounding points (Roelvink et al., 2009).

The shallow water equations that are used in XBeach are two dimensional, non-conservative, and are as follows:

Continuity:

$$\frac{\partial \eta}{\partial t}+\frac{\partial hu}{\partial x}+\frac{\partial hv}{\partial y} = 0 \tag{3}$$

X momentum:

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}-f\ v-v_h\left[\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right] = \frac{\tau_{sx}}{\rho h}-\frac{\tau_{bx}}{\rho h}-g\frac{\partial \eta}{\partial x}+\frac{F_x}{\rho h} \tag{4}$$

Y momentum:

$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}-f\ u-v_h\left[\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right] = +\frac{\tau_{sy}}{\rho h}-\frac{\tau_{by}}{\rho h}-g\frac{\partial \eta}{\partial y}+\frac{F_y}{\rho h} \tag{5}$$

Here τ_{bx}, τ_{by} are the bed shear stresses, η is the water level, F_x, F_y are the wave-induced stresses, ν_t is the horizontal viscosity and f is the Coriolis coefficient (Roelvink et al., 2009).

The other notations in the equations are:

η = water level
t = time
x, y = distance
u,v = water velocity
f = Coriolis coefficient
ρ = water density
g = gravity force per unit mass
νt = horizontal viscosity
h = water depth
τbx, τby = bed shear stresses
F_x, F_y = wave-induced stresses

A first order upwind explicit schematisation with an automatic time step is the preferred numerical method used in XBeach (Roelvink et al., 2008), due to the many shock-like characteristics which occur in hydrodynamic and morphodynamic behaviour (Stelling and Duinmeijer, 2003). The discretisation is similar to the one developed by Stelling and Duinmeijer in its momentum-conserving form, hence it is able to capture shocks and is very suitable for 'drying and flooding', allowing for combinations of sub- and supercritical flows.

The developers of XBeach selected upwind scheme in order to avoid numerical oscillations of many shock-like phenomena, which occur in coastal and flooding situations.

The scheme is able to avoid shock oscillations introduced by the additional dissipative term (Hibberd and Peregrine, 1979). As a result, the upwind scheme, together with a staggered grid, makes the model robust (Roelvink et al., 2009).

In this paper, XBeach is tested against a number of cases; firstly it is compared to the calculation results from semi-analytical solutions. Second, it is tested against the results from different fluvial codes based on various cases (Heriot Watt, 2009). The last comparison is against an experimental case in a laboratory environment (Soarez-Frazao and Zech, 2008). Table 1 provides an overview of the tests undertaken.

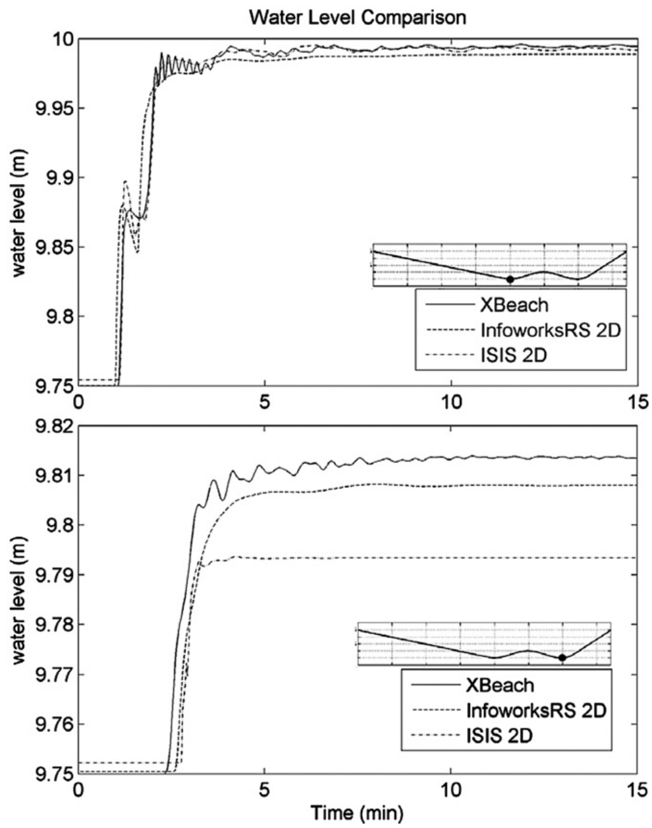

Fig. 8. Test 5 – momentum conservation.

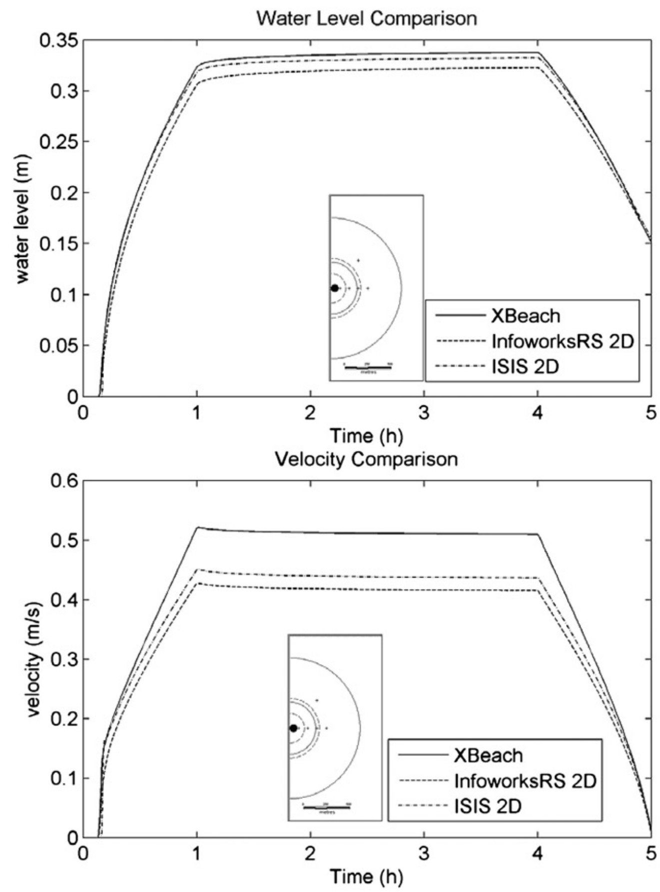

Fig. 9. Test 6 – flood propagation over a plain.

3. Test cases

3.1. Semi-analytical solution comparison

There are certain fluvial hydraulic scenarios which can be solved using semi-analytical methods. These cases provide the starting point for testing the capability of XBeach in the fluvial environment. In addition to the cases that have semi-analytical solutions, there are also cases where known fluvial behaviour can be tested.

The first test examines XBeach's capability to model simple cases such as gradually varied flow and backwater effects, Test 1a-d. The code was tested to model mild slope types (M1 and M2) and steep slope types (S2 and S3) (Chow, 1959; Cunge et al., 1980).

In the mild slope case (1a and 1b), a fragment of very wide river is modelled of a 100 m width with frictionless walls on both sides. A gentle slope of 0.001 is created in the model bathymetry over a 10 km distance. Uniform roughness with a Chézy coefficient of $C = 50$ is applied. For the M1 case (Test 1a), a constant discharge of 2 $m^3/s/m$ is introduced at the upstream boundary and a constant 5 m water level boundary condition in downstream. These conditions at the boundaries generate an M1 flow curve which varies from a water level of 1.17 m–5 m.

The M2 flow curve (Test 1b) was set up using a 5 $m^3/s/m$ discharge at the upstream boundary and a critical depth of 1.37 m at the downstream boundary of the model. These boundary conditions generate a normal water depth of 2.15 m at the upstream boundary.

The cases 1c and 1d are the steep slope cases, here the S2 and S3 flow curves are generated using a 5 km long channel with various bed slopes (i.e. 0.03, 0.01 and 0.001), as shown in Fig. 2. The upstream boundary condition is set as a constant discharge of 5 $m^3/s/m$, and the downstream boundary condition of 2.15 m water level. The S2 flow curve occurs after the transition from the 0.001 to the 0.03 slope and the S3 flow curve at the transition from 0.03 to 0.01 slope.

The second theoretical case (Test 2a) is a straight trapezoidal channel with a flat floodplain on both sides. The model uses a uniform value of Chézy roughness coefficient for both the main channel and floodplain area. The test investigates the two dimensional flow calculations of the software. Furthermore, Test 2b investigates the case of an embanked floodplain to examine the hydraulic representation of a disconnected waterbody.

Case 2a was modelled using a 5 km long, straight channel, with a 0.001 slope and a river cross-section of 30 m width (bottom), 2 m deep and floodplain of 30 m each side. The upstream boundary condition was a varying discharge from 50 to 700 m^3/s with the peak discharge occurring after 43.3 min and a minimum value after 60 min. The dimensions of the channel were set so as to allow an overflow at the peak flow. In Case 2b dikes are constructed on both banks of the channel.

More complex, and realistic, hydraulic behaviours throughout the domain are found in meandering channels. A perfect sinusoidal meander with no slope was modelled (Test 2c). This test investigates the water flows from the main channel to floodplain and vice versa, secondary flow in the curved channel and velocity distributions as well as flood wave behaviour.

For Case 2c the modelled reach uses a rectangular channel 50 m wide and 5 m depth, with a length of 4.50 km. The actual model domain was only 3 km as the meanders were introduced to create the extra length in the main channel. The total width of the floodplain was 600 m on both river banks. A zero bed slope was applied in this case, in order to model flow in the channel only as

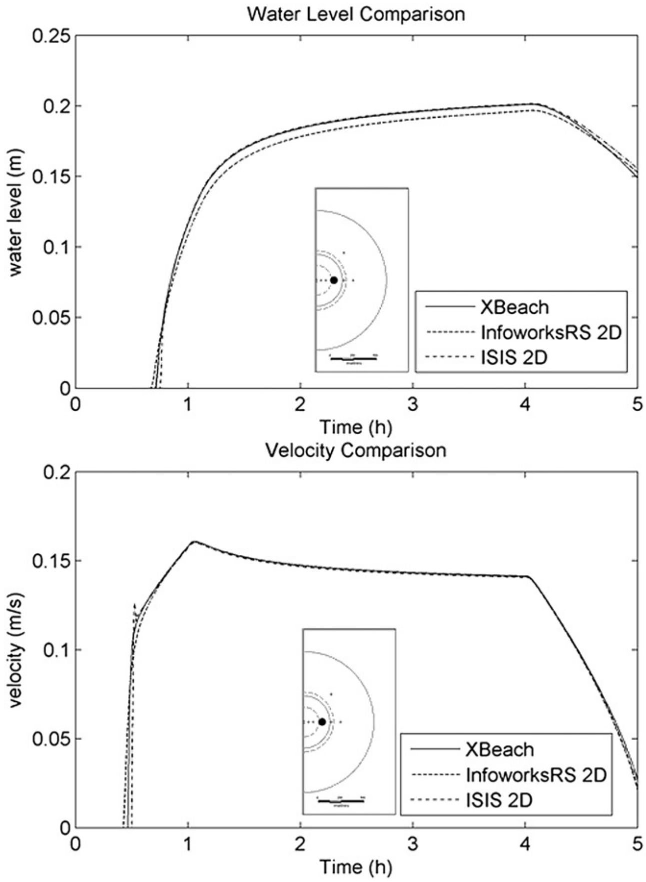

Fig. 10. Test 6 – flood propagation over a plain (cont).

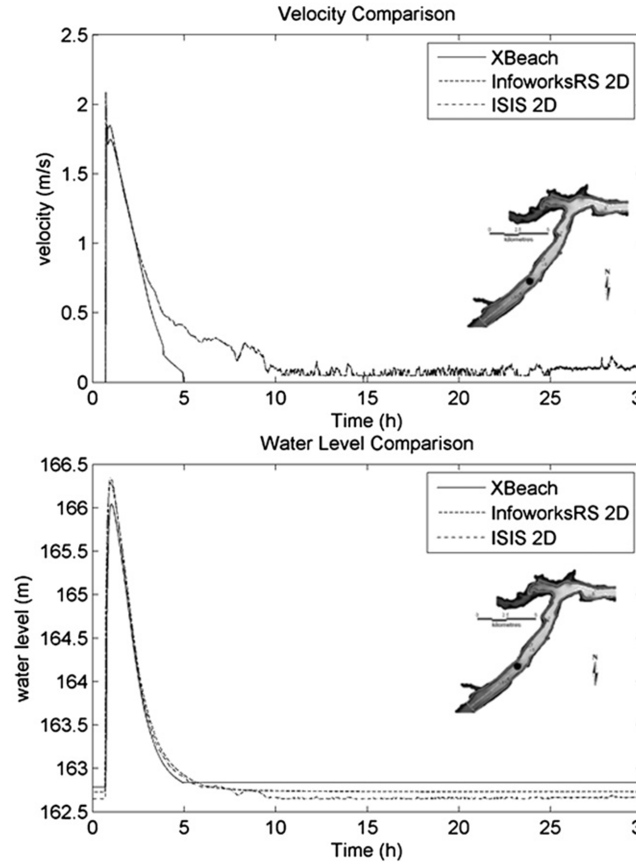

Fig. 11. Test 7 – dam break over a valley (head of the valley).

a result of the upstream boundary condition. This was set as a varying discharge (from 200 m^3/s to 1000 m^3/s),

3.2. The Environment Agency 2D benchmarking study (EA2D)

In 2009, the Environment Agency of England and Wales carried out a benchmarking study for 2D software. The benchmarking was undertaken to ensure that codes used for fluvial studies commissioned by the Agency were appropriate for use in assessing flood risk. The project was led by Heriot Watt University (Heriot Watt, 2009) and the simulations were set up and carried out by various model developers throughout the world. The final report of this study is available to the public (Neelz and Pender, 2010). Six out of eight EA2D tests were chosen to test the ability of XBeach in fluvial modelling situations. For this study comparisons were made to InfoWorks RS 2D and ISIS 2D.

Test 3 (the first test of the EA2D project) investigates the code's capability to handle wetting and drying of a disconnected waterbody. A water level fluctuation is introduced in the model with two low points disconnected by a bump. The modelled domain is 100 m $\times$ 700 m with a large bump in the middle of the bathymetry of 10.25 m elevation, in order to disconnect the water bodies once the water level is lower than the elevation of the bump. The boundary conditions imposed were varying water levels at the left side of the model (between 9.70 m and 10.35 m at time $t = 1$ h–11 h and than decreasing water levels back to the value of 9.7 m). The water level and velocity at two locations are compared.

Test 4 determines inundation extent with low momentum flows in a complex topography. Furthermore, it also examines the disconnected waterbody, wetting and drying of a floodplain, inundation extent and looks at final depth rather than maximum depth. The model domain is a sloped plain in two directions with 16 depressions included in the terrain to retain a portion of the water that flows in from upper corner of the domain. The model covers an area of 2000 m $\times$ 2000 m with 16 depression each of 0.5 depth. There is an overall slope of 1:1500 in the north direction and 1:3000 towards the east, resulting in a $\sim$2 m drop of elevation between top left corners to bottom right corner.

The inflow boundary condition was located at the top left side of the domain over a length of 100 m, with a discharge value of 20 m^3/s for a period of 75 min starting at time $t = 10$ min. All other boundaries of the domain are closed boundaries.

The fifth test simulates momentum flow over a barrier. This capability is important in sewer or pluvial flood modelling in urban floodplain areas. The domain consists of a steep slope to accelerate the inflow and a bump to disconnect it from another depression. The boundary condition is a discharge of 65.5 m^3/s for 10 s starting at time $t = 5$ s with a peak at time $t = 15$. The model is 300 m long with a bump of 25 cm height. The domain consists of a steep slope to accelerate the inflow and a bump to disconnect it from another depression. The volume of the inflow is just enough to fill the depression. Water is expected to overtop the bump due to the force of momentum and settle in the depression behind the bump. This test differentiates codes which incorporate the full momentum terms and those that do not.

Case 6 tests the simulation of flood propagation over a wide floodplain following a dike failure. A high burst inflow is applied at the breach point, and a wide flat floodplain is modelled to test the propagation of a flood wave and velocities at the leading edge of the flood wave. The modelled area is a 1000 m $\times$ 2000 m of flat

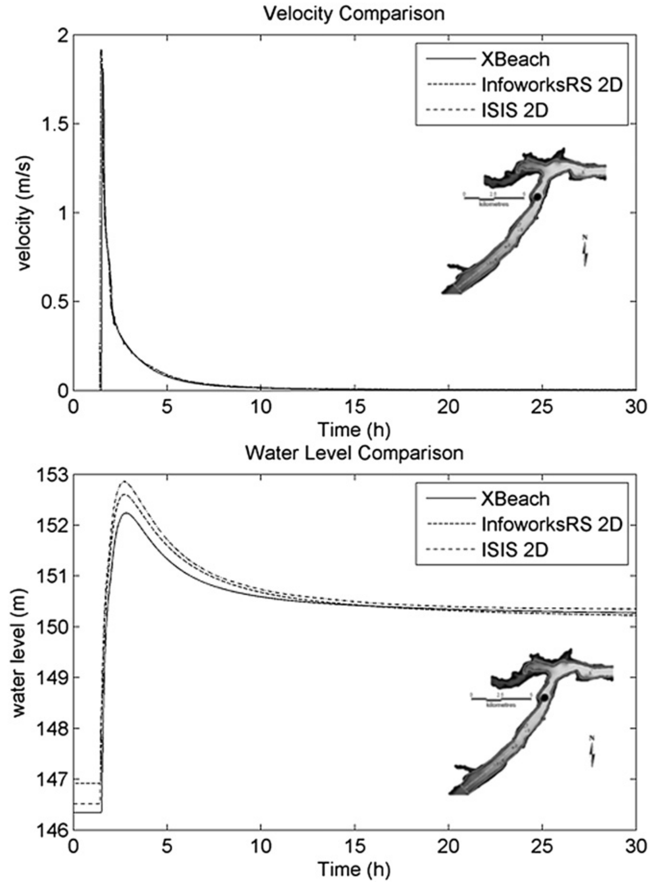

Fig. 12. Test 7 – dam break over a valley.

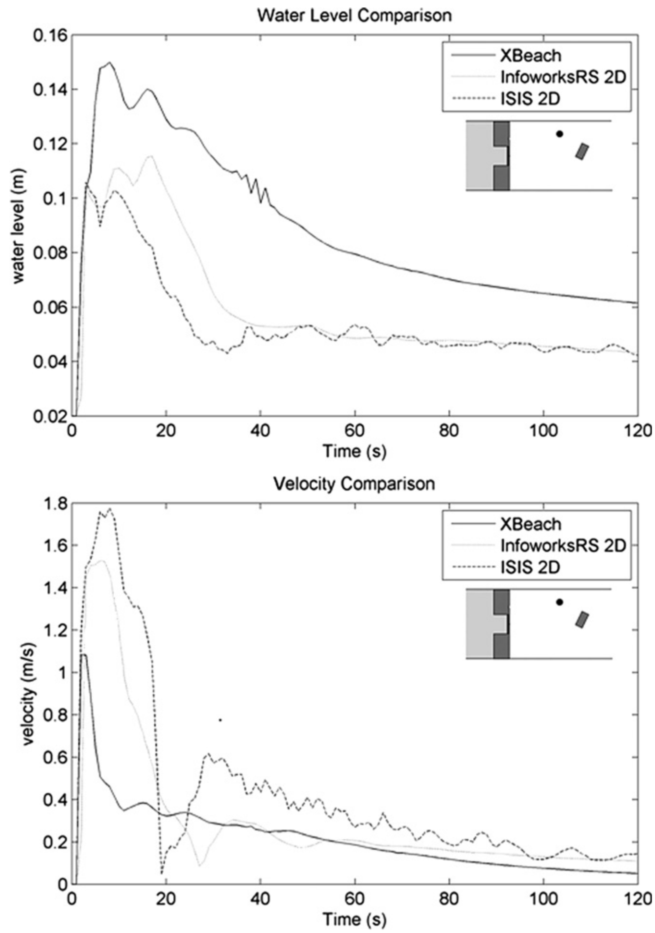

Fig. 13. Test 8a – dam break over a building, laboratory scale.

topography. The inflow boundary condition reaches a peak discharge of 20 m^3/s at time $t = 60$ min and continues constantly with this value for a further 180 min. The inflow is located in the centre of the left boundary of the model. The objective of the test is to examine the capability of XBeach to simulate the speed of flood wave propagation and predict transient velocities and depths. The test is applicable for fluvial and coastal flooding caused by a dike breach.

The penultimate EA2D test (Test 7) models the simulation of a flood wave propagation following a dam failure that flows through a river valley. The case tests the software's capability in simulating major flood inundation and flood hazard prediction that arises from a dam break scenario. The software was expected to be able to model a high burst discharge over steep and mild bed slopes involving both subcritical and supercritical flow. The test has a skewed discharge boundary with peak flow of 3000 m^3/s at time $t = 10$ min for 10 min and slowly decreases thereafter for a further 80 min.

Test 8 is adapted from a benchmark test case from the IMPACT project (IMPACT, 2005; Soares-Frazao and Zech, 2002). This test examines the capability of the software to simulate hydraulic jumps and the wake zone behind a building. The test consists of two cases, the laboratory scale (1:20) (Test 8a) and the realistic scale (Test 8b). The scale of the scaled model is 1:20. The dam size is 3.6 m $\times$ 99 m with a breach of 1 m wide in the middle of the dam (6.75 m from the left side of the dam). The initial water level in the reservoir behind the dam is 0.20 m, while the water level in the floodplain area is set to a value of 0.02 m (a wet bed domain). A model of a building is set in the floodplain in line with the dam breach location. Test 8b is a dam break case at real scale. The size of the computer model is obtained by multiplication with 20. Therefore the initial water level at the dam is 8.00 m and in the floodplain area is 0.4 m.

3.3. Experimental case comparison

The experimental test is based on the paper by Sandra Soares-Frazao and Zech (2008). The physical model represents an urban area with 25 buildings blocks flooded by a dam break simulation of a reservoir (Soarez-Frazao and Zech, 2008). The main difference between this test (Test 9) and the last test of EA2D project is the complexity of the obstacle. In this case there are many buildings and simulated streets. The capability to model hydraulic jump and complex flow through urban areas is therefore investigated.

Details on the models meshes, sizes and boundary condition types are given in Table 2. The Courant number used is not included in the table, but remains the same for all tests. The number used is 0.9.

4. Results and discussion

For the backwater cases comparing the modelled results with the semi-analytical results shows deviations (Fig. 3). In the M1 and M2 cases, a difference is observed at the boundary while for the S2 and S3 cases, the deviation occurs along the profiles, from the point of disturbance until the solution reaches normal depth. These anomalies occur at the transition from mild to a steep slope (M2, S2), while smaller differences are observed at the transition from

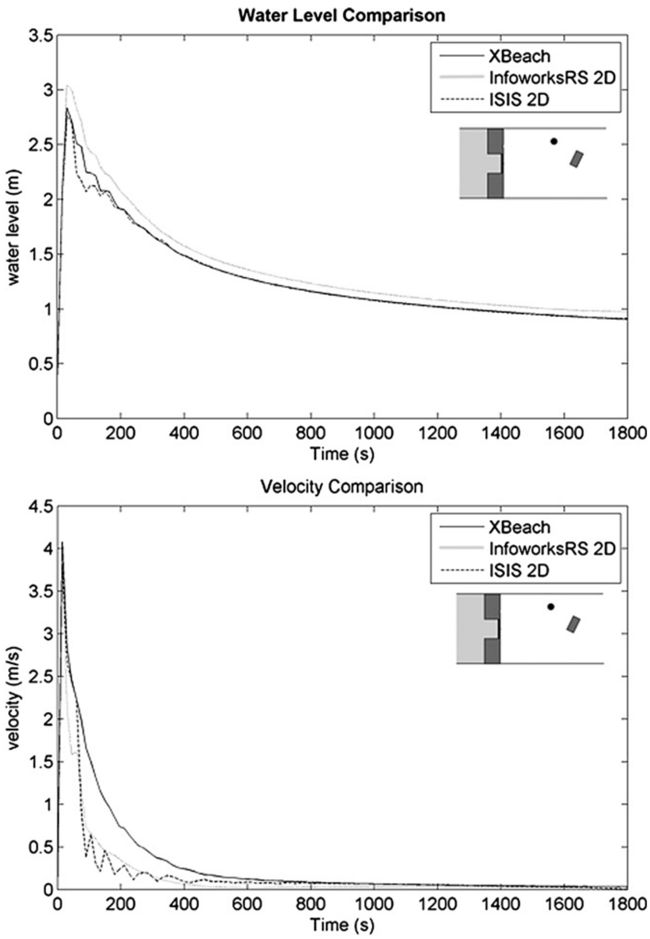

Fig. 14. Test 8b – dam break over a building, realistic scale.

steep to mild slope (M1, S3). This is due to the transition from subcritical to supercritical flow and shows the ability of the code to capture shocks. Boundary conditions also show inconsistencies. The inconsistency in results observed at the boundary is due to the implementation of a flow boundary condition, which is currently represented using velocity vectors. These are implemented at the centre of each cell situated in the boundary.

For Test 2a and 2b, the straight trapezoidal channel, good results are obtained in the case of overland flow prediction and Fig. 4 indicates that the hydraulic behaviour of flow on the floodplain is reasonably modelled. The hysteresis effects can clearly be seen in both figures (a and b), indicating the behaviour of the fluid as it flows out of bank. In the case of the embanked river, a distinctive transition can be observed in the hysteresis as the water overtops the levees and accommodates the available volume below. For Test 2c, Fig. 5 shows the patterns of flow in a meandering channel. Known behaviours such as water flows in to the main channel, and flow distribution characteristics in a channel with floodplain are observed (Muto et al., 1999). The flood wave progressing down the main channel ahead of the flow distribution on the flood plains can be observed. Tests 2a, 2b and 2c perform well and demonstrates that the celerity of propagation of a fluvial flood wave is represented well in the numerical scheme in a variety of scenarios.

For Test 3, the incoming water is expected to fill the depressions in the domain. Fig. 6a and b shows that XBeach compares well with ISIS 2D and Infoworks RS 2D for this test. A small instability can be observed following maximum depth, on the downslope of the first bump (Fig. 6a). Each computational code compared gives marginally different results. Additionally, differences are observed at initial and final water depths, which are of the order of a few millimetres. In general however, the results of Test 3 show a good comparison to other fluvial computational codes.

On the other hand, Fig. 7 shows the results of Test 4, where significant differences between codes can be observed. Due to the wet/dry threshold value that was set in XBeach, a greater number of dry depressions are observed than anticipated. The closer to the inflow location that the result is sampled the more favourable the XBeach comparison with other codes. However XBeach performs poorly in this test. The reason for this behaviour is that the threshold value of the wetting and drying algorithm is high. This means the domain retains 2 cm of water when it should actually be dry. This is due to the mathematical formulation of the problem whereby for Manning's equation the depth (d) calculation is completed with the Manning coefficient located in the denominator and hence the expression generates errors. The problem could be avoided if the Chézy equation is used instead of Manning. The use of the Manning equation was due to the test requirements where the Manning model is the preferred roughness representation in the fluvial environment for the Environment Agency. This issue can be neglected when modelling real rivers, since the 2 cm threshold does not impact significantly at the larger scale.

Fig. 8 shows the results of Test 5, momentum conservation, and indicates that there are minor differences between the codes. Firstly an instability occurred in the XBeach solution, close to maximum depth. Secondly, different final water levels for each code were observed, although all codes calculate water levels close to the expected 10 m mark. These differences are considered to be marginal for this case, and all codes demonstrate that momentum conservation is captured in the numerical formulation. The results of Test 6 are shown in Figs. 9 and 10. The results of the three codes give different values of velocity and water levels at the tested nodes. All show the general behaviour of half circle flood extents. From the results in Fig. 10 different values recorded at test nodes are observed. However, the arrival times for all nodes are approximately the same for all codes. After the arrival of the peak at the nodes differences in the predictions are observed. In general flood propagation over a plain and the momentum conservation tests (Tests 5 and 6) report good results. XBeach is able to model the

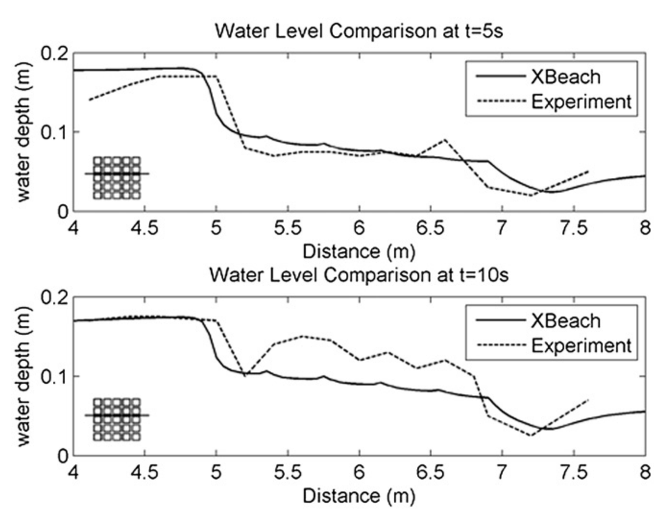

Fig. 15. Test 9 – Laboratory experiments.

required scenarios, and reproduce expected flow behaviour in a comparable manner to the other two river codes.

For the realistic scale dam break test (Test 7), XBeach gives a higher final water level result than the other tested codes and a lower value for flow velocities for nodes located at the head of the valley (Fig. 11). This trend is less evident further down the valley (Fig. 12). This is explained by the fact that Xeach is using a first order scheme, which is dissipative. However, for Test 8a (the EA2D laboratory small scale model), where the code's ability to reproduce hydraulic jumps at a laboratory scale is investigated, noticeable differences between the three codes for computed water depth and velocities can be seen (Fig. 13). When this is translated into the realistic scale (Test 8b) however, a similar result is observed for all three codes (Fig. 14).

The result from the dam break case through a valley (Test 7), and dam break over a building (Test 8b) gives comparable results to InfoWorks RS 2D and ISIS 2D. However, the small scale results show poor agreement (Test 8a), which is to be expected. Nevertheless, if Froude scaling is applied between the small scale measurement and simulation real scale computations for XBeach, a good match is found, which cannot be said for the other codes.

Finally, for Test 9, the comparison between XBeach and measured laboratory results of a dam break experiment over an urban area shows large differences between the two, especially at the street level. Improved results are likely if the computational mesh were to be refined significantly, however computational time would increase. This modification on the scale required however would be difficult. The structured rectangular grid that is used in XBeach can lead to a large number of cells, if it is applied to a detailed complex system at real scale such as a meandering channel, bifurcation or an urban area, however it gives a quick solution when modelling low complexity fluvial system. Highly complex river systems, which are solved only by using large domains, detailed meshes and long computational times, can be addressed by running such cases in a parallel manner, on multiple processor computers or by making use of grid computing.

5. Conclusions

The Shallow Water equation solver in XBeach has been seen to work well for river modelling scenarios, when compared to other codes developed for the fluvial environments. Furthermore, with an open source licence, any user may improve the software or add flexibility. However, some deficiencies are acknowledged. The existing representation of the boundary conditions, while adequate for coastal environments, was not always applicable for fluvial modelling purposes, especially in the case of upstream flow conditions. A small sub-routine was implemented, however further refinement is warranted.

There are several cases for which the software can be further tested, such as spillways, chute blocks, etc, which were not tested in this study. The main objective of this benchmarking study was to see the capability of the software to represent floods in rivers using this coastal software. The assumption under which the Shallow Water equations are solved using Xbeach restricts application of these equations for flow problems with a steep bed slope, particularly for the supercritical cases. Consequently testing spillways with this code would imply changes were implemented so that supercritical cases can be tested as well. Pipe flow was not tested in this study.

Codes treating coastal problems address the wetting and drying of computational cells differently from 2D river modelling which result in different inundation patterns. This can be overcome by imposing a different threshold value than that used in coastal applications of XBeach. Similarly in the coastal environment Chézy is the roughness representation of choice. Transferring to the fluvial environment for this study requires the implementation of a sub-routine to change the roughness coefficient, in this case to Manning's. Further modification should be implemented.

XBeach uses a structured staggered grid which can be inflexible for representing complex fluvial geometries. As a recommendation for further development of XBeach, it is suggested that an unstructured grid option is investigated so as to avoid very fine grids.

Finally, the conclusion of this research opens up the possibility to use this model for both hydraulic and potentially morphological problems in fluvial, coastal and hence transition areas.

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